MOTT MACDONALD

Development and Evaluation of Alternative Controls

Bergen County Utilities Authority
CSO Long Term Control Plan

NJPDES Permit No. 0020028

July 1, 2019

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List of Abbreviations and Acronyms

BCMP – Baseline Compliance Monitoring program
BF – Ballasted Flocculation
BSF – Base Sanitary Flow
BCUA – Bergen County Utilities Authority
CCI – Construction Cost Index
CEPT – Chemically Enhanced Primary Treatment
CIWEM – Chartered Institute of Water and Environmental Management
CMP – Compliance Monitoring Program
CSM – Combined Sewer Management
CSO – Combined Sewer Overflow
CSS – Combined Sewer System
CWA – Clean Water Act
DEFM – Demographic and Employment Forecast Model
DPW – Department of Public Works
DSW – Discharge to Surface Water
DWF – Dry Weather Flow
EDP – Effective Date of Permit
ENR – Engineering News Record
EPA – Environmental Protection Agency
FW – Freshwater
GWI – Ground Water Infiltration
I/I – Inflow/Infiltration
JC – Junction Chamber
LTCP – Long Term Control Plan
MGD – Million Gallons per Day
MHW – Mean High Water
NJDEP – New Jersey Department of Environmental Protection
NJHDG – New Jersey harbor Discharge Group
NJPDES – New Jersey Pollution Discharge Elimination System
NJTPA – North Jersey Transportation Authority
NOAA – National Oceanic and Atmospheric Administration
NT – Non-trout
OVRS – Overpeck Valley Relief Sewer
OVTS – Overpeck Valley Trunk Sewer
POTW – Publicly Owned Treatment Works
PVSC – Passaic Valley Sewerage Commission
PWQM – Pathogen Water Quality Model
RDII – Rainfall Derived Infiltration and Inflow
RTC – Real Time Control
RTK – Initials of hydrograph parameters for RDII unit hydrographs
SE – Saline Estuary
STP – Sewage Treatment Plant
TAZ – Traffic Analysis Zone
TGM – Technical Guidance Manual
TMDL – Total Maximum Daily Load
USACE – United States Army Corps of Engineers
USCG – United States Coast Guard

USFWS – United States Fish and Wildlife Service
WaPUG – Wastewater Planning Users Group
WMP – Wastewater Management Plan
WPCF – Water Pollution Control Facility
WQS – Water Quality Standards
WWF – Wet Weather Flow

Certification(s)

BCUA Development and Evaluation of Alternatives Report

Submitted on behalf of the following participating Permitee by

Bergen County Utilities Authority on behalf of the BCUA CSO Group

NJPDES Number NJ0020028 (Bergen County Utilities Authority)

Approval of Report

Permitee: _____ 6/26/15

Robert E. Laux — Date

Executive Director, Bergen County Utilities Authority

NJPDES Certification:

"Without prejudice to any objections timely made to permit conditions, I certify under penalty of law that this document and all attachments were prepared as part of a cooperation performed by members of the BCUA CSO Group effort in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons who developed this report, or those persons directly responsible for gathering the information, the information submitted is, to the best of my knowledge and belief, true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for purposely, knowingly, recklessly, or negligently submitting false information."

Permitee: _____ 6/26/15

Robert E. Laux — Date

Executive Director, Bergen County Utilities Authority

Certification

Title: **BCUA Development and Evaluation of Alternatives Report**

NJPDES Number NJ0020028

"I certify under penalty of law that this document and all attachments were prepared under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons who manage the system, or those persons directly responsible for gathering the information, the information submitted is, to the best of my knowledge and belief, true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for purposely, knowingly, recklessly, or negligently submitting false information."

Permitee: John Dening 6/25/2019

John R. Dening, NJ P.E. #45517 Date

Senior Project Engineer

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BCUA Development and Evaluation of Alternatives Report

Submitted on behalf of the following participating Permitee by

Bergen County Utilities Authority on behalf of the BCUA CSO Group

NJPDES Number NJ0034517 (Borough of Fort Lee)

Approval of Report

Permitee: 6/25/2019

Alfred R. Restaino Date

Borough Administrator

NJPDES Certification:

"Without prejudice to any objections timely made to permit conditions, I certify under penalty of law that this document and all attachments were prepared as part of a cooperation performed by members of the BCUA CSO Group effort in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons who developed this report, or those persons directly responsible for gathering the information, the information submitted is, to the best of my knowledge and belief, true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for purposely, knowingly, recklessly, or negligently submitting false information."

Permitee: 6/25/2019

Alfred R. Restaino Date

Borough Administrator

BCUA Development and Evaluation of Alternatives Report

Submitted on behalf of the following participating Permitee by

Bergen County Utilities Authority on behalf of the BCUA CSO Group

NJPDES Number NJ0108766 (City of Hackensack)

Approval of Report

Permitee: 6/20/2019

Susan Banzon — Date

Project Manager, City of Hackensack

NJPDES Certification:

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Permitee: 6/20/2019

Susan Banzon — Date

Project Manager, City of Hackensack

BCUA Development and Evaluation of Alternatives Report

Submitted on behalf of the following participating Permitee by

Bergen County Utilities Authority on behalf of the BCUA CSO Group

NJPDES Number NJ0109118 (Village of Ridgefield Park)

Approval of Report

Permitee: Alan O'Grady 6/19/19

Alan O'Grady Date

Superintendent, Ridgefield Park Village

NJPDES Certification:

"Without prejudice to any objections timely made to permit conditions, I certify under penalty of law that this document and all attachments were prepared as part of a cooperation performed by members of the BCUA CSO Group effort in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons who developed this report, or those persons directly responsible for gathering the information, the information submitted is, to the best of my knowledge and belief, true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for purposely, knowingly, recklessly, or negligently submitting false information."

Permitee: Alan O'Grady 6/19/19

Alan O'Grady Date

Superintendent, Ridgefield Park Village

Executive Summary

The Bergen County Utilities Authority (BCUA) currently owns and operates the Water Pollution Control Facility (WPCF) located in Little Ferry and provides wastewater transportation and treatment services for forty-seven (47) municipalities, serving a population of about 565,000 people. The BCUA services municipalities that are primarily located in the Hackensack River and Overpeck Creek drainage areas and provides wastewater treatment and transportation services to three (3) municipalities with combined sewer systems: the Borough of Fort Lee, the City of Hackensack, and the Village of Ridgefield Park. The BCUA Service Area extends approximately five (5) miles south and fifteen (15) miles north of the Little Ferry WPCF and is bounded by the Hudson River on the east, by New York State to the north, by the remainder of Bergen County to the west, and by Hudson County to the south. The Authority's service district covers approximately 135 square miles and is located primarily in the Hackensack River drainage basin.

The New Jersey Department of Environmental Protection issued New Jersey Pollutant Discharge Elimination Permits (NJPDES) to all municipalities/authorities that own or operate combined sewer systems and authorities that provide wastewater transport and/or treatment services to municipalities with combined sewer systems. The BCUA owns and operates the trunk/intercepting sewer systems that transport flows to the WPCF, including wastewater flows from combined sewer systems. The collection and conveyance of wastewater (both dry and wet weather) from municipal combined sewer systems into the BCUA Trunk Sewer, including the Combined Sewer Overflow (CSO) discharge pipes, are owned and operated by the individual municipalities. These facilities are permitted under Individual NJPDES Permits provided to the BCUA and each combined sewer municipality with an effective date of July 1, 2015 as follows:

- Bergen County Utilities Authority (BCUA) – NJPDES Permit No. 0020028
- Borough of Fort Lee – NJPDES Permit No. 0034517
- City of Hackensack – NJPDES Permit No. 0108766
- Village of Ridgefield Park – NJPDES Permit No. 0109118

These permits require that the permittees prepare and submit a Development and Evaluation of Alternative Controls Report (Report) and ultimately a CSO Long Term Control Plan (LTCP). The permit provided the option for these to be undertaken on a regional basis for all hydraulically connected municipalities to BCUA. The BCUA, Fort Lee, Hackensack, and Ridgefield Park agreed to undertake a Regional approach to the all required reports and the CSO LTCP. Work being undertaken together is being completed by the BCUA CSO Group, which is made up of all four individual permittees within the District.

While the members of the BCUA CSO Group have agreed to work cooperatively on the Development and Evaluation of Alternative Control Report. Most of the work for the Selection and Implementation of Alternatives Report will be completed separately and then coordinated and integrated through group meetings into a regional submission through the BCUA. Three different consultants were engaged in the development of Regional Report. The Borough of Fort Lee retained HDR to complete its individual Report, the City of Hackensack retained Arcadis to complete its individual Report, while the Village of Ridgefield Park and BCUA both retained Mott MacDonald to complete their Reports. As part of the Regional Sewer System Characterization Report (June 2018), Mott MacDonald developed a comprehensive model of the entire BCUA District using the InfoWorksICM modeling software covering the

forty-four separately sewered municipalities, with special emphasis on the combined sewer systems within the Borough of Fort Lee, the City of Hackensack, and the Village of Ridgefield Park. The models for the combined sewer areas were developed by the respective municipal permittees and imported into the overall BCUA model.

The CSO Long Term Control Plan (LTCP) for the BCUA Service District is due to the NJDEP no later than June 1, 2020. While the Development and Evaluation of Alternatives Report provide preliminary data on the alternatives being considered, it is not the LTCP. Additional investigations and coordination between the members of the BCUA CSO Group will take place over the next eleven months to work towards a Regional LTCP that is acceptable to all members and which complies with the individual NJPDES permits issued to individual members. The information provided herein is a summary of the investigations and cost analyses conducted by each individual member of the BCUA CSO Group. More detailed information is available within the individual reports from each member that has been referenced within this report. Copies of the municipal permitees reports have been appended to this report as Sections 13 through 15. The results will be summarized by each BCUA CSO Group member.

A. BCUA

Transport Facilities

The BCUA does not own nor operate any combined sewer overflow outfalls, and thus, is not responsible for directly reducing the number of CSO events, which is a responsibility of the other members of the group. As the owner and operator of the transport and treatment facilities the BCUA has responsibility for maximizing wet weather flows to their Water Pollution Control Facility and, by its permit is tasked with evaluating the feasibility and cost of treatment plant expansion and bypassing secondary treatment and blending of the effluent flows if the secondary treatment units are the limiting factor in the capacity of the plant.

The investigation of the BCUA Transport Facilities concentrated on that segment of the transport system that receives wet weather flows from the combined sewer municipalities of Fort Lee, Hackensack, and Ridgefield Park. Theoretical determination of trunk sewer capacity was made through the use of Manning's equation. Nevertheless, these are dynamic systems where the hydraulics are impacted by conditions that cannot be analyzed entirely through simple calculations. The BCUA developed an InfoWorksICM model of their entire transport system as part of the characterization of its transport system for evaluating the impact of wet weather flows from both combined and sanitary sewer systems. The InfoWorks model was exercised using rainfall from the typical year (2004) that showed that, except for those sewers directly impacted by the hydraulic grades within the influent wet well, the entire system operates under gravity flow conditions and does not surcharge. The model was also used in a theoretical manner to determine the peak flows generated and transported within the district using a set rainfall analysis, wherein the rainfall intensity was increased in steps, to evaluate what segment of the system surcharges first. That analysis showed that flows in the Overpeck Creek Trunk and Relief Sewers are controlled by the pipe capacity in the upper reaches, which reach a surcharged condition first.

In summary, the performance of the BCUA interceptors is complex and depends on many interrelated factors such as level control on wet well, what the critical points in the system are defined to be, and the distribution of flow entering the interceptor. The theoretical flows as determined through Manning's Equation indicate that the Main Trunk Sewer have a capacity of approximately 120 MGD, and that the joint Overpeck Valley Trunk and Relief Sewers have a combined capacity of approximately 143 MGD.

Together these sewers should theoretically be able to transport approximately 265 MGD to the WPCF. Notwithstanding the theoretical calculation, the InfoWorksICM model step rainfall analysis indicates that

the maximum flow capacity transfer through the Overpeck Valley Trunk Sewers is controlled by the upstream components of these sewers. Accordingly, the flows introduced downstream must be such that they do not negatively impact the upstream hydraulic grades to an extent that surcharging and potential backups are created upstream. Overall the modeling conducted indicates that approximately 210 MGD can be transported to the WPCF safely without hydraulically impacting upstream sewer systems as indicated in the previous table. When flows to the WPCF exceed 210 MGD it is likely that upstream in the system surcharging conditions exist.

Summary of Trunk Sewer Capacities

Trunk Sewer Description	Max Flow (MGD)
Main Trunk Sewer	128
Overpeck Valley Trunk Sewer	62
Overpeck Valley Relief Sewer	17
Total Flow to WPCF	207

BCUA Treatment Facilities

The BCUA retained Arcadis, its Authority Consulting Engineer, to evaluate the hydraulic and treatment capacity of its Little Ferry WPCF of the primary and secondary treatment units, and to then determine that facilities and costs that expected to increase the overall blended capacity of the WPCF to match the 210 MGD capacity of the its transport system. While the hydraulic capacity of the plant appears to be around 325 MGD, the process capacity of the plan is currently 120 MGD. A memorandum was prepared by Arcadis outlining the investigation undertaken and the results of those. The conclusions reached by Arcadis are noted in the extracts from the report as provided below.

CONCLUSIONS

Based on a hydraulic capacity assessment of the existing conveyance system to BCUA LF WPCF, as much as 210 MGD can flow through the plant under hydraulic control. If hydraulic control is not necessary, as in a major storm event, as much as 325 MGD can be accepted by the plant, however existing influent trunk capacity is limited to 210 MGD. Based on process modeling and a review of New Jersey and 10 States design standards, the process capacity of the WPCF is estimated at 120 MGD. In order to accommodate flow beyond 120 MGD (up to 210 MGD) during wet weather events, it is proposed that either CEPT or BF be used. The table below summarizes the estimated capital and operational expenditures associated with each of these options.

Table 16. Summary of EPOCC and OPEX (2018 dollars)

Option	EPOCC	OPEX
chemically-enhanced primary treatment	$32MM – $127MM	$0.80MM
ballasted flocculation	$55MM - $220MM	$1.20MM

Irrespective of which option is decided, negotiations with the NJDEP must be had in order to be authorized to blend effluent from the WWS with secondary effluent from the main WPCF. Blending will require a relaxation of current permit limits for cBOD and TSS, so the NJDEP will need to determine when the use of the WWS will be authorized. Further, an evaluation of the sludge management/conveyance systems should be performed to confirm they can handle the additional loadings during wet weather events.

B. Borough of Fort Lee

The following is a copy of the Executive Summary as included in the Borough of Fort Lee Report:

This report is being provided as one of several reports required for the Fort Lee Combined Sewer Overflow (CSO) Long Term Control Plan (LTCP) under NJPDES Permit No. NJ0034517. It evaluates control alternatives but makes no claims as to the ultimate control that will be selected. Costs are also developed for some control alternatives, but these are comparative costs developed solely for making comparisons of the capital and operating costs of the alternatives. Fort Lee has not yet committed to providing for the costs associated with control of their CSOs.

The Borough of Fort Lee was issued a Combined Sewer Overflow (CSO) permit (NJPDES Permit No. NJ0034517) by the New Jersey Department of Protection in 2015. The permit requires the development of a Long-Term Control Plan for reducing CSOs by June 1, 2020. There are several requirements for reports and other actions. This report, Development and Evaluation of Alternatives Report, discusses all the alternatives available for CSO reduction and selects alternatives that could be used to reduce CSOs in Fort Lee.

The Borough of Fort Lee comprises approximately 1,600 acres which is serviced by combined and separately sewered areas. The combined sewer system consists of approximately 640 acres discharging to three pump stations and two CSO outfalls. The CSO outfalls are activated in rain storms.

One significant improvement was made in 2016 to the combined collection system that impacts CSOs. In 2016 a sewer infrastructure project servicing the new Hudson Lights project revised the sewer collection system. Before 2016, the Lower Main Pump Station sent pumped flow to a 12-inch pipe by gravity to the Palisade Terrace Pump Station which would then pump to the interceptor. After 2016, the flow from the Palisade Terrace Pump Station was rerouted to a new 12-inch pipe that discharges directly to the BCUA interceptor. In addition to the rerouting the flow, the pump station capacity was upgraded from 2 MGD to 5 MGD. This change in the combined sewer system reduced annual CSOs at the Palisade Terrace outfall from 11.73 MGD to 4.17 MGD. This reduction in CSOs corresponds to 84.7% CSO capture which almost achieves EPA's CSO control policy objective of 85% capture.

The CSO outfalls are in a neighboring town, Edgewater, on the Hudson River. The Hudson River is an SE2 water body in the vicinity of Fort Lee with a fecal coliform criteria (geometric mean) of 770 cfu/100 mL. Currently, sampling programs show the water quality to be in compliance with this criteria; therefore, water quality is not a driver for CSO control.

If Fort Lee is required to reduce CSOs further, the alternatives that they could use to reduce or eliminate CSOs are gray infrastructure alternatives such as disinfection, high rate filtration with disinfection and storage tanks. Peracetic Acid (PAA) would be the likely disinfection process selected because of the long shelf life of PAA and the non-toxic nature of any residual concentration. Testing of the PAA disinfection process with and without high rate filtration would likely be performed to gather design data.

Control alternatives that could reduce but not eliminate CSOs are sewer separation and green infrastructure. Fort Lee may use sewer separation or green infrastructure to increase CSO capture to 85%. If more CSO capture is required by NJDEP then we will consider the gray infrastructure alternatives of disinfection, high rate filtration with disinfection and storage tanks.

The lifecycle cost for achieving 85% capture by sewer separation or GI ranges from $6,250,000 to $10,000,000. For tank storage, the most expensive CSO control alternative considered, the range is $47,000,000 to $167,000,000. For filtration with disinfection the cost range from $36,000,000 to $85,000,000. If disinfection alone proves to be a viable option then costs may range from $3,720,000 to $7,270,000. These evaluations of alternatives will serve as a base for the consideration and development of final selected CSO control plan in Fort Lee. We believe the most cost effective solution for meeting water quality objectives and complying with the EPA CSO control policy will be GI, sewer separation or disinfection with PAA.

C. City of Hackensack

The following is a copy of the Executive Summary as included in the City of Hackensack Report:

Pursuant to the City of Hackensack's (City) New Jersey Pollution Discharge Elimination System (NJPDES) Permit No. NJ0108766 (Permit), this Development and Evaluation of Alternatives Report (Alternatives Report) evaluates a reasonable range of combined sewer overflow (CSO) control alternatives that will meet the water quality-based requirements of the Clean Water Act (CWA) using either the Presumption Approach or the Demonstration Approach in accordance with the City's Long Term Control Plan (LTCP) requirements.

The City's combined sewer system (CSS) consists of two subdrainage areas, which are referred to as Anderson and Court Street. The Anderson Street subdrainage area serves an area of approximately 470 acres and conveys sanitary and combined sewage to a regulator facility at Anderson Street. The Court Street subdrainage area serves an area of approximately 440 acres and conveys sewage to a regulator

facility at Court Street. The regulators at Court Street and Anderson Street both overflow to the Hackensack River when the sewers have reached their wet weather capacities. These overflows are referred to as combined sewer overflows, or CSO. The dry weather flow from the City's CSS is conveyed to the Bergen County Utilities Authority (BCUA) Water Pollution Control Facility (WPCF) in Little Ferry, NJ.

The Permit specifies seven CSO control alternatives that should be evaluated by each permittee. Five of those alternatives are evaluated in this Alternatives Report: green infrastructure, increased storage capacity in the collection system, inflow and infiltration (I/I), sewer separation, and treatment of the CSO discharge. The two other alternatives are sewage treatment plant (STP) expansion and/or storage at the plant and CSO related bypass of the secondary treatment portion of the STP. As discussed in this Alternatives Report, the two other alternatives pertain to improvements that may be undertaken by the BCUA at the STP and that may impact the sizing of the five alternatives evaluated herein. The CSO control alternatives were prescreened to determine if certain CSO control alternatives were economical and feasible for the City to further evaluate. The prescreening process utilized the 2007 LTCP Cost and Performance Analysis Report (2007 Report), prepared by Malcolm Pirnie, Inc. (now Arcadis U.S., Inc.) that was required per the City's previous NJPDES Permit No. NJ0105023. City-wide sewer separation, STP expansion, CSO related bypass of secondary treatment at the STP, and maximizing the storage within the City's existing CSS network were prescreened out of further consideration due to outside factors, extensive costs, and limited capacity within the existing CSS.

Green infrastructure, satellite storage tanks, a regional storage tank, a tunnel, I/I reduction, and treatment of CSO discharge were further evaluated in this Alternatives Report. In the City's evaluation of the CSO control alternatives, the City used NJDEP approved hydrologic, hydraulic, and water quality models where applicable. The City utilized the hydrologic/hydraulic model to simulate the existing conditions and conditions as they are expected to exist after construction and operation of the chosen alternative(s). The City evaluated the practical and technical feasibility of the proposed CSO control alternative(s) and the water quality benefits of constructing and implementing various CSO controls or a combination of such controls. The CSO control alternatives were evaluated for 4, 8, 12, and 20 overflows per year as well as elimination or the capture for treatment of no less than 85% by volume of the combined sewage collected in the CSS during precipitation events for the 2004 typical year.

After further evaluation, the most effective CSO control alternatives to minimize overflows were satellite storage tanks or tunnel. Treatment of the CSO discharge was evaluated for disinfection alone and disinfection with pretreatment. If disinfection alone is determined to be an adequate CSO control alternative, it is a candidate to be selected as a preliminary CSO control alternative for the City. However, because of the uncertainty of future pretreatment requirements, it is not known if this alternative will satisfy the water quality requirements for future permits. The green infrastructure and I/I reduction alternatives are beneficial for the City for a variety of reasons; however, they are not solely adequate to reach the water quality goals. Therefore, a combination of storage tanks or tunnel with green infrastructure and I/I reduction may be the most effective and economical CSO control alternative for the City to incrementally implement in order to reach the required water quality goals. As future conditions change for the City and additional CSO technologies become available or improve, the CSO control alternatives in this Alternatives Report may be revisited in order to suit the City's best interests and needs to meet the water quality goals.

The following table was provided by Arcadis, the City of Hackensack Consulting Engineer, to summarize the results of their investigations:

Table 8-1: Summary

Name of Alternative	Percent of Capture	No. of Overflows	Reduction of Overflow Volume from Baseline (%)	Total Estimated Cost	Cost per Gallon of CSO Removed	Key Constraints
Baseline conditions for 2004	68%	56	N/A	-	-	-
Disinfection	68%	56	N/A	$ 15,800,000	N/A	Uncertain if this alternative satifies water quality goals, number of overflows not reduced, no pretreatment.
Pretreatment & Disinfection	68%	56	N/A	$ 49,930,000	N/A	Uncertain if this alternative satifies water quality goals, number of overflows not reduced.
GI - 5% Impervious Area	70%	51	13.0%	$ 31,980,000	$0.96	Does not reach performance & water quality goals, number of overflows not reduced.
GI - 10% Impervious Area	70%	51	14.8%	$ 42,640,000	$1.12	Does not reach performance & water quality goals, number of overflows not reduced.
Removal of Inflow and Infiltration (I&I)	68%	56	0.1%	$ 11,310,000	$59.84	Does not reach performance & water quality goals, number of overflows not reduced.
Tunnel Storage - 17.8ft Diameter Tunnel (4 Overflows)	96%	4	89.6%	$ 97,300,000	$0.43	Constructability of a deep tunnel has risks, high cost.
Tunnel Storage - 17ft Diameter Tunnel (8 Overflows)	95%	8	87.2%	$ 94,160,000	$0.43	Constructability of a deep tunnel has risks, high cost.
Tunnel Storage - 14ft Diameter Tunnel (12 Overflows)	93%	12	79.7%	$ 84,740,000	$0.43	Constructability of a deep tunnel has risks, high cost.
Tunnel Storage - 10.5 ft Diameter Tunnel (20 Overflows)	86%	20	60.9%	$ 73,750,000	$0.49	Constructability of a deep tunnel has risks, high cost.
Tunnel Storage - 10.5 ft Diameter Tunnel (85% Capture)	86%	20	60.9%	$ 73,750,000	$0.49	Constructability of a deep tunnel has risks, high cost.
Satellite Tanks - Two tanks, 115 ft dia., 100 ft deep (4 Overflows)	98%	4	93.0%	$ 140,000,000	$0.59	Siting issues for tank locations, high cost.
Satellite Tanks - Two tanks, 105 ft dia., 100 ft deep (8 Overflows)	96%	8	89.7%	$ 123,060,000	$0.53	Siting issues for tank locations, high cost.
Satellite Tanks - Two tanks, 87 ft dia., 100 ft deep (12 Overflows)	94%	12	81.6%	$ 96,420,000	$0.46	Siting issues for tank locations, high cost.
Satellite Tanks - Two tanks, 73 ft dia., 100 ft deep (20 Overflows)	89%	20	66.9%	$ 79,140,000	$0.46	Siting issues for tank locations, high cost.
Satellite Tanks - Two tanks, 60 ft dia., 100 ft deep (85% Capture)	85%	25	52.7%	$ 65,780,000	$0.49	Siting issues for tank locations, high cost.
Regional Tank - One tank, 120 ft dia., 130 ft deep (4 Overflows)	97%	4	91.8%	$ 118,600,000	$0.50	Siting issues for tank location, large diameter sewer construction, high cost.
Regional Tank - One tank, 120 ft dia., 110 ft deep (8 Overflows)	96%	8	87.9%	$ 107,730,000	$0.48	Siting issues for tank location, large diameter sewer construction, high cost.
Regional Tank - One tank, 100 ft dia., 105 ft deep (12 Overflows)	92%	12	77.3%	$ 87,250,000	$0.44	Siting issues for tank location, large diameter sewer construction, high cost.
Regional Tank - One tank, 80 ft dia., 100 ft deep (20 Overflows)	88%	20	63.5%	$ 71,480,000	$0.44	Siting issues for tank location, large diameter sewer construction, high cost.
Regional Tank - One tank, 65 ft dia., 100 ft deep (85% Capture)	85%	21	53.8%	$ 63,080,000	$0.46	Siting issues for tank location, large diameter sewer construction, high cost.
Control Program Alternative - I&I Removal and GI 5% Impervious Area	70%	50	13.0%	$ 43,290,000	$1.29	Does not reach performance & water quality goals, number of overflows not reduced.
Control Program Alternative - I&I Removal and GI 10% Impervious Area	70%	51	14.8%	$ 53,950,000	$1.43	Does not reach performance & water quality goals, number of overflows not reduced.

D. Village of Ridgefield Park

The following is a copy of the Executive Summary as included in the Village of Ridgefield Park Report:

This Development and Evaluation of Alternatives Report (DEAR) has been prepared in partial fulfilment of the Village of Ridgefield Park's obligations under their individual Combined Sewer Management Permit, permit number NJ0109118, issued March 12, 2015, with an effective date of July 1, 2015, and minor revisions issued October 9, 2015. Specifically, this report addresses the requirement of Part IV.D.3.b.v, as per the detail provided in Part IV.G.4. This report is being developed cooperatively with the Bergen County Utilities Authority (BCUA) Combined Sewer Overflow (CSO) Group. Reference are also made to prior reports submitted by Ridgefield Park or submitted by the NJ CSO Group on behalf of Ridgefield Park including:

- Combined Sewer System (CSS) Characterization Report
- Public Participation Process Report
- Sensitive Areas Report
- Baseline Compliance Monitoring Program (BCMP) Report

These documents have been approved by the New Jersey Department of Environmental Protection (NJDEP).

Permittee Background

The Village of Ridgefield Park (Village), located in south central Bergen County New Jersey, is bounded by the Overpeck Creek on the east and south, the Hackensack River on the west and Interstate Highway 80 on the north. The Village has a population of about 12,980 people within its municipal boundaries, which encompasses approximately 2 square miles.

The Village has a total of ten (10) Combined Sewer Overflow (CSO) regulating facilities, three of which are owned by BCUA. The regulators are connected to six (6) CSO discharge points, four of these discharge points flow into the Hackensack River, while the remaining two discharge to the estuary portion of Overpeck Creek.

Public Outreach

Public outreach and input are an important component of the LTCP progress, and the project team has endeavored to provide opportunities for public education and awareness, as well as to gain feedback on the CSO control alternatives. Efforts by the overall BCUA CSO Group, of which the Village is a part, are documented in the BCUA Report. This section only covers activities since the Public Participation Process report was last submitted on January 07, 2019, prior activities are documented in that report. Below id a summary of activies specific to The Village, undertaken since the approves Public Participation Progress Report.

- Village of Ridgefield Park Supplemental CSO (SCSO) Team Meetings
- Village Caucus Meeting
- Earth Day Public Outreach Event
- Additional Outreach Efforts

Sensitive Areas

Consistent with the requirements of the National CSO Control Policy, the NJPDES CSO Permits stipulates that the highest priority must be given to controlling overflows to sensitive areas. A thorough assessment was conducted as described in the Identification of Sensitive Areas Report, dated June 2018, submitted

by PVSC to the NJDEP, and approval was received on April 8, 2019. The approved report concluded there were no sensitive areas within Ridgefield Park. Accordingly, the Village's outfalls will be added uniformly with consideration to the overall reductions in systemwide volume and frequency of overflows.

Future Baseline Condition

The Permit requires the permittee to simulate "conditions as they are expected to exist after construction and operation of the chosen alternative(s)" (Part IV.G.4.e). The intent is to reduce the risk that foreseeable changes in the community and sewer system will reduce the effectiveness of the proposed LTCP facilities. To address this, an evaluation of anticipated changes to the Village's population and potential changes to sewer flows was undertaken. Discussions were also held with the Village to document planned changes to the sewer system. It has been assumed that the alternatives that are selected through the LTCP process will be constructed and implemented over a 30-year period. As such, the year 2050 has been selected as the future baseline condition. A comparison of the 2015 and 2050 baselines is provided below in Table 1, there was no meaningful change to overflow volume or frequency.

Table 1: Comparison of 2015 Baseline and 2050 Future Baseline

Outfall	Baseline 2015			Baseline 2050			Change		
	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)
001A	19	6.3	74	19	6.4	74	0	0.0	0
002A	13	0.4	11	13	0.4	11	0	0.0	0
003A	45	15.3	300	45	15.3	300	0	0.0	0
004A	53	24.1	614	53	24.1	614	0	0.0	0
005A	25	3.6	121	25	3.6	121	0	0.0	0
006A	12	0.5	39	12	0.5	39	0	0.0	0
Total	53	50.3	--	53	50.3	--	0	0.0	--

Screening of CSO Technologies

A two-tiered approach was applied to the development of alternatives, starting with a screening analysis followed by an evaluation of the remaining CSO control alternatives. The intent is to give adequate attention to the breadth of alternatives available, but to limit the list of alternatives evaluated to a reasonable amount. This is consistent with Chapter 3 of EPA's Guidance for Long Term Control Plans.

The first step of the screening process was to identify the breadth of alternatives which can then narrowed down to alternatives appropriate for the evaluation process. If necessary, a representative technology to apply to the evaluation is identified. A comprehensive list of CSO control alternatives was prepared by the NJCSO Group.

The screening took place on several levels. In some cases, a general category was screened in or out based on its applicability to the Village. If the general category of technologies was applicable as were many sub-categories, the screening reduced the sub-categories to a reasonable number of representative sub-categories.

The screening was based on the requirements to "evaluate the practical and technical feasibility of the proposed CSO control alternative(s)" (Part IV.G.4.e) to determine if the alternative will proceed to a more detailed evaluation in Section D. This above requirement introduced three concepts that were addressed for each technology:

- Evaluate
- Practical

- Technical Feasibility

Siting of CSO Facilities

Preliminary siting issues is listed in USEPA's Combined Sewer Overflow – Guidance for Long Term Control plans (EPA 832-B-95-002 September 1995) as a screening mechanism and recommends the evaluation of the following:

- Availability of sufficient space for the facility on the site
- Distance of the site from CSO regulator(s) or outfall(s) that will eb controlled
- Environmental, political, or institutional issues related to locating the facility on the site

The Village of Ridgefield Park was first analyzed using the following publicly available GIS information:

- Aerial photography
- Land Use / Land Cover
- Parcel data, including vacant land, land ownership, and property value information
- Open Space / Green Acres
- Soil Type
- Topography
- Known Contaminated Sites
- Brownfields

Potential sites were identified as were the constraints on each site. Some sites were eliminated from consideration due to the suitability for siting CSO control facilities.

Performance Objectives

The magnitude of the facilities in terms of CSO volume managed is the primary driver of both its cost and effectiveness. Accordingly, a procedure was developed to achieve the desired control objectives, in this case limiting the overflows to 0, 4, 8, 12 or 20 during the Typical Year. Since the permit requires the levels of control to be established based on the hydraulically connected system it is not adequate merely to achieve the desired number of overflows at each individual outfall, or within Ridgefield Park. Prior to the evaluation it was necessary to determine for the BCUA system what storm events must be controlled for each level of control. Since the LTCP may incorporate a mix of volume-based controls (storage) as well as peak flow-based control (treatment) the same sets of storms were established for either control methodology.

Control Programs

Six (6) Control Programs were developed, each is discussed in greater detail in the individual report.

- Control Program 1 – Elimination of CSO 006A
- Control Program 2 – Consolidated Tank Storage
- Control Program 3 – Consolidated Tunnel Storage
- Control Program 4 – Consolidated End of Pipe Treatment
- Control Program 5 – Sewer Separation
- Control Program 6 – Green Infrastructure

Evaluation

Each alternative was implemented in the approved InfoWorksICM 2050 baseline model and the modeled facilities scaled to achieve each of the performance objectives for the Typical Year rainfall. The exception was green infrastructure which was implemented to address 2.5%, 5%, 7.5%, 10% and 15% of the modeled directly connected impervious areas. 20-year net present work costs were generated for each alternative using capital costs and operations and maintenance costs. For comparison purposes each alternative was normalized by the cost to remove on gallon of CSO during the Typical Year. Results are summarized below in Table 2 through Table 7.

Table 2: 20-Year net present worth for all control programs

Control Program	NPW Summary - Overflows per Year ($M)				
Level of Control	0	4	8	12	20
1) Eliminate Outfall 006	NA	NA	NA	NA	NA
2) Storage (Consolidated)	$84	$54	$52	$47	$34
3) Tunnel	$118	$99	$99	$92	$86
4) Treatment (Consolidated)	$87	$77	$77	$77	$60
5) Sewer Separation	$193	NA	NA	NA	NA
	NPW Summary - % of Impervious Area Managed ($M)				
	2.50%	5%	7.50%	10%	
6) Green Infrastructure	$2.7	$6	$9	$12	

Table 3: Summary of CSO Volumes for Typical Year

Control Program	2015 Baseline (MG)	Level of Control - Overflows during Typical Year (MG)				
		0	4	8	12	20
1. Eliminate CSO-006A	50.3	NA	NA	NA	NA	NA
2. Consolidated Tank Storage	50.3	0.0	5.7	5.8	9.7	21.5
3. Tunnel	50.3	0.0	4.7	4.7	7.9	11.4
4. Consoldiated End of Pipe Treatment	50.3	0.0	0.2	0.2	0.2	3.0
5. Sewer Separation	50.3	0.0	NA	NA	NA	NA
% Impervious to GI		2.5%	5%	7.5%	10%	
6. Green Infrastructure	50.3	49.9	49.4	48.9	48.3	

Table 4: Summary of CSO Volume Reductions for Typical Year

Control Program	2015 Baseline (MG)	Level of Control - Overflows during Typical Year (MG)				
		0	4	8	12	20
1. Eliminate CSO-006A	50.3	NA	NA	NA	NA	NA
2. Consolidated Tank Storage	50.3	50.3	44.5	44.4	40.6	28.7
3. Tunnel	50.3	50.3	45.6	45.6	42.4	38.8
4. Consoldiated End of Pipe Treatment	50.3	50.3	50.0	50.0	50.0	47.2
5. Sewer Separation	50.3	50.3	NA	NA	NA	NA
% Impervious to GI		2.5%	5%	7.5%	10%	
6. Green Infrastructure	50.3	0.3	0.8	1.4	2.0	

Table 5: Summary of Frequency of Overflows for Typical Year

Control Program	2015 Baseline	Level of Control - Overflows during Typical Year				
		0	4	8	12	20
1. Eliminate CSO-006A	53	NA	NA	NA	NA	NA
2. Consolidated Tank Storage	53	0	4	4	10	20
3. Tunnel	53	0	4	4	7	10
4. Consoldiated End of Pipe Treatment	53	0	1	1	2	10
5. Sewer Separation	53	0	NA	NA	NA	NA
% Impervious to GI		2.5%	5%	7.5%	10%	
6. Green Infrastructure	53	53	53	53	53	

Table 6: Summary of Percent Capture Achieved by Each Control Program

Control Program	2015 Baseline	Level of Control - Overflows during Typical Year				
		0	4	8	12	20
1. Eliminate CSO-006A	69.5%	NA	NA	NA	NA	NA
2. Consolidated Tank Storage	69.5%	100.0%	96.5%	96.5%	94.1%	86.9%
3. Tunnel	69.5%	100.0%	97.2%	97.2%	95.2%	93.1%
4. Consoldiated End of Pipe Treatment	69.5%	100.0%	99.9%	99.9%	99.9%	98.2%
5. Sewer Separation	69.5%	100.0%	NA	NA	NA	NA
% Impervious to GI		2.5%	5%	7.5%	10%	
6. Green Infrastructure	69.5%	69.7%	70.0%	70.3%	70.7%	

Table 7: Net present worth costs normalized by gallon of CSO reduction

Control Program	Cost per Gallon Volume CSO Reduction ($/gal)				
Level of Control	0	4	8	12	20
1) Eliminate Outfall 006	NA	NA	NA	NA	NA
2) Storage (Consolidated)	$1.7	$1.2	$1.2	$1.1	$1.2
3) Tunnel	$2.4	$2.2	$2.2	$2.2	$2.2
4) Treatment (Consolidated)	$1.7	$1.5	$1.5	$1.5	$1.3
5) Sewer Separation	$3.8	NA	NA	NA	NA
	Volume Reduction for Impervious Area Managed (MG)				
	2.50%	5%	7.50%	10%	
6) Green Infrastructure	$9.1	$7.2	$6.3	$5.8	

Each alternative was ranked on the six categories below, and the results are summarized in Table 8:

- Cost
- CSO Reduction
- CSO Frequency Reduction
- Institutional Issues
- Implementability
- Public Acceptance

Table 8: Summary of Control Program Rankings

Control Program	Cost	CSO Volume Reduction	CSO Frequency Reduction	Institutional Issues	Implement-ability	Public Acceptance	Weighted Score
1. Eliminate CSO-006A	NA	NA	NA	NA	NA	NA	NA
2. Consolidated Tank Storage	4	5	5	4	3	3	4.0
3. Tunnel	3	5	5	4	2	2	3.5
4. Consoldiated End of Pipe Treatment	4	5	5	2	3	2	3.6
5. Sewer Separation	2	5	5	3	2	2	3.1
6. Green Infrastructure	1	1	1	5	4	5	2.7
Weighting	25%	15%	15%	15%	15%	15%	100%

1 Introduction

The Bergen County Sewage Authority (now the Bergen County Utilities Authority, or BCUA) undertook the construction of a trunk sewer and sewage treatment plant in the late 1940s to relieve the pollution in Overpeck Creek. The plant, which went into operation in 1951, provided secondary treatment for a design flow of 20 MGD to serve ten municipalities and industries along the Overpeck Valley (Cliffside Park, Englewood, Fairview, Fort Lee, Leonia, Palisades Park, Ridgefield, Ridgefield Park, Teaneck, and Tenafly) several of which had combined sewer systems. In the early 1960's, the Sewage Treatment Plant service was extended to sixteen (16) additional municipalities including Hackensack City, which also has a combined sewer system. The last municipality to be added to the District was Wood-Ridge in 1992. Plant capacity was increased periodically over the years to extend service to other municipalities in eastern Bergen County to its present permitted capacity of 94 MGD due to load restrictions in the receiving water, it has a design capacity of 109 MGD (10 States Standard) and a wet weather capacity of 120 MGD. In 2014 the average daily flow treated averaged 77.3 MGD.

The BCUA currently owns and operates the Water Pollution Control Facility (WPCF) located in Little Ferry and provides wastewater transportation and treatment services for forty-seven (47) municipalities, see Figure 1-1, serving a population of about 565,000 people. The BCUA service area covers approximately 135 square miles, primarily located in the Hackensack River and Overpeck Creek drainage basins. This service area is bounded by the Hudson River on the east, by New York State to the north, by the remainder of Bergen County to the west, and by Hudson County to the south. The areas serviced by the BCUA are primarily residential with isolated sections that service industrial and commercial facilities. It is estimated that approximately 8% – 10% of the dry weather flow to the BCUA Water Pollution Control Facility (WPCF) is contributed by these industrial and commercial facilities.

The BCUA provides wastewater treatment and transportation services for forty-four (44) municipalities with separate sewer systems, and three (3) municipalities with combined sewer systems: the Borough of Fort Lee, the City of Hackensack, and the Village of Ridgefield Park. While the BCUA owns and operates the trunk / intercepting sewer systems (trunk sewers) that transport flows to the WPCF, it does not own or operate any of local collector sewers, which are owned and operated by each individual municipality. The BCUA also does not own any CSO outfalls.

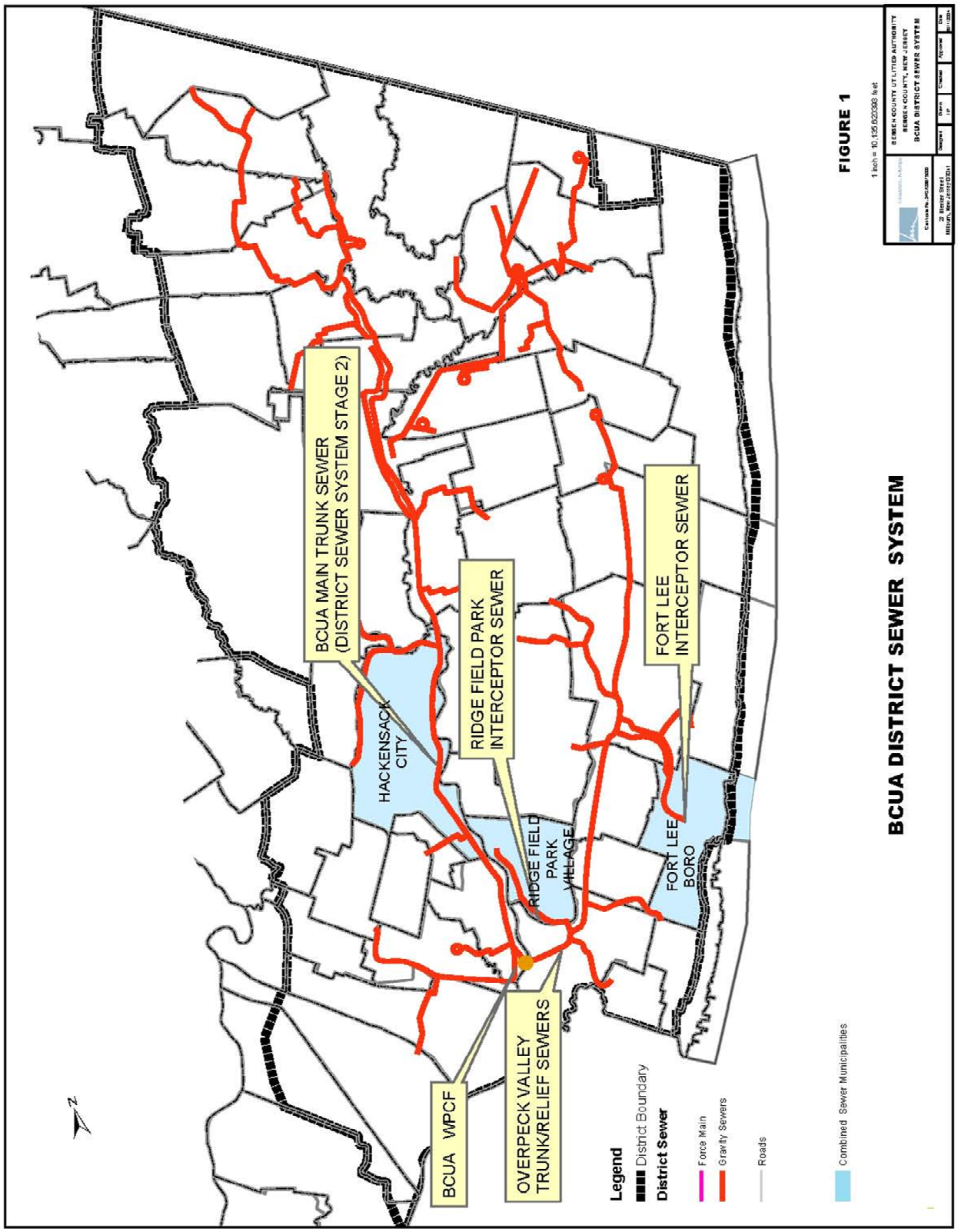

Figure 1-1: BCUA District Trunk Sewers

All CSO Control Facilities in the City of Hackensack and the Borough of Fort Lee, and four of the seven CSO Control Facilities within the Village of Ridgefield Park are owned and operated by their respective municipality. BCUA owns and operates three CSO Control Facilities within the Village of Ridgefield Park. These CSO facilities are permitted under Individual New Jersey Pollutant Discharge Elimination Permits (NJPDES) provided to the BCUA and each combined sewer municipality with an effective date of July 1, 2015 as follows:

- Bergen County Utilities Authority (BCUA) – NJPDES Permit No. 0020028
- Borough of Fort Lee – NJPDES Permit No. 0034517
- City of Hackensack – NJPDES Permit No. 0108766
- Village of Ridgefield Park – NJPDES Permit No. 0109118

These permits require that the permittees prepare and submit a Development and Evaluation of Alternative Controls Report and ultimately a regional CSO Long Term Control Plan for all hydraulically connected municipalities to BCUA. The information collected under the Sewer System Characterization Report was used to evaluate all CSO control alternatives as per Part IV – Combined Sewer Management (CSM), G.4 of the NJPDES permit.

The BCUA's Trunk Sewer System is depicted in Figure 1-1 with the three tributary combined sewer municipalities highlighted in blue located at the downstream end of the system. The combined sewer municipalities are serviced by either the Overpeck Valley Trunk Sewer/Overpeck Valley Relief Sewer (Fort Lee Borough and the Village of Ridgefield Park) or the BCUA Main Trunk Sewer constructed as the District Sewer System Stage 2 (Hackensack), which services municipalities along the Hackensack River and extends from the Little Ferry WPCF north toward New York State. Accordingly, the BCUA has studied its trunk sewer system to characterize the wet weather components of its transport and treatment facilities. This study, in conjunction with characterization studies performed with the other members of the BCUA Group, will be used to characterize and evaluate the maximization of wet weather flows along with other LTCP alternatives required under CSM Part IV.G.4 to the BCUA WPCF as part of the regional CSO LTCP.

1.1 Regulatory Background and Project History

In 2015, the New Jersey Department of Environmental Protection revoked prior authorizations related to combined sewer overflows under NJPDES Master General Permit No. NJ0105023 and issued individual permits to individual municipalities and commissions/authorities that own or operate facilities that control, transport, or treat wastewater flows from combined sewer systems. CSO Discharges from the Borough of Fort Lee, City of Hackensack, and the Village of Ridgefield Park designated CSO outfalls are now authorized and regulated by individual NJPDES Permits. While the BCUA does not own or operate CSO outfalls, the downstream portion of the BCUA trunk sewer system receives and conveys combined sewage from the Village of Ridgefield Park, the Borough of Fort Lee, and the City of Hackensack, whose systems are hydraulically connected. As such, the New Jersey Department of Environmental Protection (NJDEP) revoked and reissued the BCUA individual Category "A" Permit No. NJ0020028 to incorporate and update CSO NJPDES permit requirements as part of the permit actions.

In the current NJPDES Individual Permits, the NJDEP has mandated that the permittees undertake steps, as needed, for the development of a CSO Long Term Control Plan incorporating permit conditions that reflect the control standards and goals of the CSO Control Policy established by United States Environmental Protection Agency (EPA). The individual permits also encourage permittees to cooperate in the development of a plan that incorporates all permittees within a hydraulically connected system into a Regional CSO LTCP. The BCUA, Fort Lee, Hackensack, and Village of Ridgefield Park have joined

together to form the BCUA CSO Group in a cooperative manner to develop a regional plan. While the individual CSO municipalities must develop a plan for the reduction or elimination of CSO discharges, the BCUA needs to evaluate its transport and treatment facilities to ensure that it is, to the extent practical, maximizing flows to the WPCF during wet weather events.

A CSO LTCP involves a comprehensive study of the hydraulically connected sewer system and the evaluation of alternatives for reducing CSO impacts to receiving waters. It investigates the hydrologic and hydraulic relationships between precipitation, conveyance, treatment capacity, and overflows. It includes a feasibility study to evaluate the scope, costs, and performance of possible control alternatives for reducing the frequency and volume of CSO discharges.

The EPA CSO Control Policy and the individual NJPDES CSO Permits describe nine elements or requirements for the development of a CSO Long Term Control Plan:

1. Characterization, monitoring, and modeling of the combined sewer systems to provide a thorough understanding of the hydraulically connected system, its response to various precipitation events, the characteristics of the overflows, and the water quality impacts that result from the CSOs;
2. A public participation process that actively involves the affected public in the decision-making to select long term CSO controls;
3. Consideration of sensitive areas in identifying the highest priority for controlling overflows;
4. Evaluation of alternatives that considers a reasonable range of CSO control options that provide a level of control presumed (per the criteria given in the Policy and Permit) or demonstrated to meet the water quality-based requirements of the Clean Water Act (CWA);
5. Cost/performance considerations to demonstrate the relationships among a comprehensive set of reasonable control alternatives;
6. An Operational plan that incorporates revisions to the operation and maintenance program necessary after approval of the LTCP to incorporate its associated CSO controls;
7. Maximizing treatment at the existing publicly owned treatment works (POTW) treatment plant during and after each precipitation event so that such flows receive treatment to the greatest extent practicable utilizing existing tankage for storage, while still meeting permit limits;
8. An implementation schedule addressing the construction and financing of proposed CSO controls; and
9. A post-construction compliance monitoring program adequate to verify compliance with water quality-based CWA requirements and designated uses as well as to ascertain the effectiveness of implemented CSO controls.

The NJPDES Individual Permit divided the above requirements into three sequential steps, providing an orderly progression for the development of the LTCP. The tasks undertaken, and the documents submitted under each step, per the specified schedule, are as follows:

- *Step 1 incorporates the characterization, monitoring, and modeling element and components of the public participation process, consideration of sensitive areas, and compliance monitoring program. It is further divided into the following submittal requirements and schedule:*
  - Permittees were required to submit a System Characterization Work Plan within 6 months from the effective date of the permit (EDP), which corresponded to a due date of January 1, 2016. These documents were submitted on time and the approved work plan served as the basis for the subsequent characterization, monitoring, and modeling efforts.

- Permittees were required to submit a System Characterization Report within 36 months of the EDP, which equates to a due date of July 1, 2018. These documents were submitted on time and served as the basis for the subsequent development and evaluation of alternatives efforts.
- Permittees were required to submit a Public Participation Process Report and a Consideration of Sensitive Areas Information document within 36 months from the EDP (i.e., July 1, 2018). These documents were also submitted on time and contributed to the development and evaluation of alternatives efforts.
- Although listed separately from the steps in the permit under the LTCP Submittal Requirements, permittees were also required to submit a baseline Compliance Monitoring Program (CMP) Work Plan by January 1, 2016 and are then required to submit a baseline CMP Report and data within 36 months of the EDP, or again July 1, 2018. The members of the BCUA CSO Group collaborated with the NJ CSO Group, a coalition of New Jersey CSO permittees, to satisfy these permit conditions through a regional ambient water quality sampling and testing program and pathogen water quality modeling.

- *Under Step 2, permittees are required to submit a Development and Evaluation of Alternatives Report within 48 months from the EDP, or a due date of July 1, 2019. This step involves evaluating a broad range of control alternatives to meet CWA requirements and water quality standards (WQS) using either the "Presumption" Approach or "Demonstration" Approach and the corresponding conditions prescribed in the permit. Maximizing treatment at the existing POTW treatment plant and cost and performance considerations also need to be addressed in Step 2. This report is being submitted in fulfillment of this permit condition.*

- *Under Step 3, permittees are required to submit a Selection and Implementation of Alternatives Report that incorporates the final plan selection and implementation schedule for the construction and financing of proposed CSO controls. A proposed operational plan revision schedule and a post-construction compliance monitoring program also should be addressed. This submittal is due within 59 months from the EDP, which corresponds to a due date of June 1, 2020.*

This report has been prepared to fulfill the requirement for the Development and Evaluation of Alternatives Report as outlined in the individual NJPDES Permit No. NJ0109118 for the BCUA as well as members of the BCUA CSO Group.

The NJPDES Individual Permit further stipulates that each permittee must work cooperatively with other permittees and municipalities in a hydraulically connected system such that the LTCP is developed consistently based on the same data, characterization, analysis methods, and results. The members of the BCUA CSO Group have actively collaborated on the previous elements of the LTCP, such as the System Characterization Report, and continue to do so with this Development and Evaluation of Alternatives Report.

The BCUA CSO Group members are also involved in the regional assembly of NJPDES Individual permittees, referred to as the NJ CSO Group. Members of this group consist of municipalities, sewerage authorities, and other entities that have been issued a NJPDES Individual permit and have agreed to work jointly in addressing common and overlapping areas for permit compliance. On behalf of the member permittees, the NJ CSO Group, with the Passaic Valley Sewerage Commission (PVSC) serving as the managing member, is providing shared services for common elements of the permits, to avoid costly and inefficient duplication of efforts. These shared services encompass certain water quality monitoring and

modeling work, such as ambient in-stream sampling and testing, baseline compliance monitoring reporting, and pathogen receiving water quality modeling; production of CSO outfall identification signs; setup and operation of a CSO notification system website; and coordination with regulatory agencies on common issues for permit compliance. This regional partnership also assists in sharing information and gathering feedback on techniques for public involvement and CSO controls.

1.2 Summary of System Characterization and Sensitive Area Reports

In the CSO LTCP process, an extensive understanding of the combined sewer system is critical to the development of a computer model capable of accurately modeling the collection system and characterizing its response to precipitation events. As such, a thorough record review and updated sewer system description, inventory, assessment, and hydrologic and hydraulic model were developed and compiled in previous steps of the CSO long term control planning process. The following provides a summary of the final System Characterization report that was re-submitted on behalf of the BCUA CSO Group to NJDEP on February 18, 2019 and subsequently approved. Characterization reports were also submitted by the municipal permittee; these reports have also all been approved.

As part of the Regional Sewer System Characterization Report (June 2018) Mott MacDonald developed a comprehensive sewer system modeling of the entire BCUA District using the InfoWorksICM model covering the forty-four separate sewered areas, with special emphasis on the combined sewer systems within the Borough of Fort Lee, the City of Hackensack, and the Village of Ridgefield Park. The models for the combined sewer communities were prepared by the municipalities and imported into the overall BCUA model. This model will be used to in the development of the CSO LTCP. The extent of the BCUA Service District and the combined sewer areas within the study area are illustrated in Figure 1-1.

The primary goal for developing the BCUA collection system model was to build a reliable tool for characterizing the performance of the existing collection system and potential future improvements developed as part of an LTCP. The primary extents of the BCUA model include the BCUA's interceptor system assets. However, accurately simulating the contributions from the tributary communities (both combined and separate) was also critical. Although point of connection flow meters are available where municipalities contribute their flow to the BCUA interceptor, the BCUA model had to be able to estimate the flows that would be contributed from these communities under arbitrary rainfall conditions. The BCUA services forty-seven individual municipalities within the County and uses over 150 point of connection flow meters for billing. Average daily flows from the point of connection meters range from less than ten thousand gallons to several million gallons of wastewater flow per day. A review of average daily flows recorded during the period of analysis showed that forty-five (45) point of connection flow meters accounted for roughly 85% of the total flow contribution to the BCUA. Accordingly, it was determined that meter analysis of dry weather flow (DWF), infiltration and inflow, and development of drainage basin modeling parameters and flow distribution needed to improve the accuracy of the BCUA model, would be restricted to these drainage basins and points of entry. Restricting the analysis allowed the wastewater and inflow/infiltration (I/I) parameter to be established efficiently for a representative portion of the meters and then to extend the results to the other basins, without performing analysis on the over 150 meters.

In addition to the simulation of wastewater flows from the forty-four separated sewered communities, a more detailed modeling effort was required for the tributary communities with combined sewer systems (Fort Lee, Hackensack, & Ridgefield Park). All three municipalities developed and shared their calibrated and verified collection system computer models with BCUA. These models were incorporated fully into the BCUA model and then reviewed and checked against the flows as predicted during their individual calibration and/or verification to assure that the imported model was consistent with the original.

1.2.1 Service Area Land Use Data

The BCUA transports wastewater flows from combined sewer systems through its Trunk Sewers however, the Authority does not own nor operate any of the collector sewers nor the CSO outfalls. These are owned and operated by the individual municipality (Fort Lee, Hackensack, and Ridgefield Park).

1.2.1.1 Borough of Fort Lee

The wastewater flows from the Borough of Fort Lee are primarily residential and are pumped into a branch truck sewer at two locations, which then flow to the Overpeck Trunk that transports wastewater to the WPCF located in Little Ferry. The BCUA Trunk Sewers servicing the Borough of Fort Lee are illustrated in Figure 1-2. Additional information on the combined sewer systems within the Borough of Fort Lee can be found in the Borough of Fort Lee Sewer System Characterization Report as submitted to NJDEP.

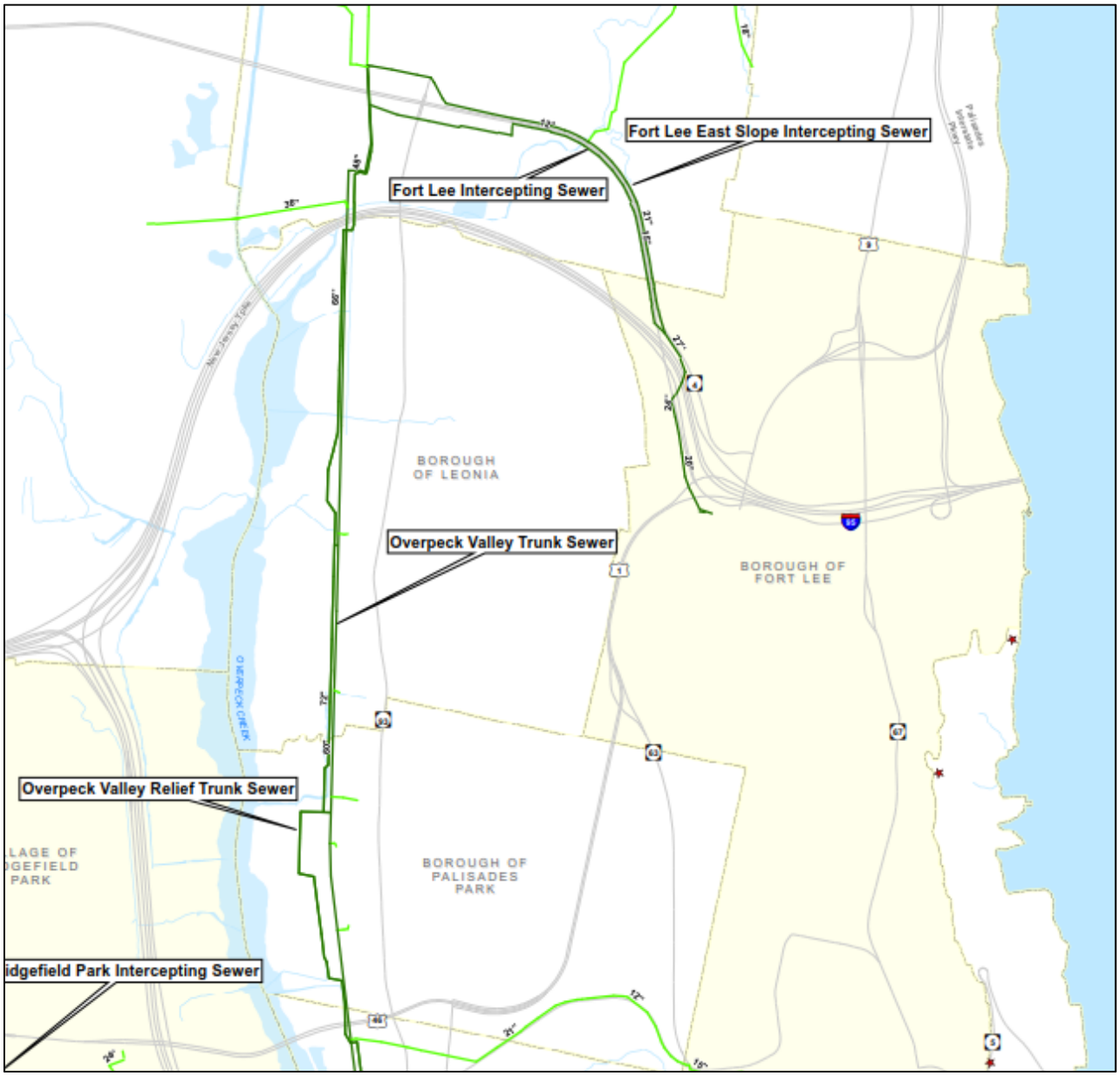

Figure 1-2: Trunk / Intercepting Sewers Servicing Fort Lee

1.2.1.2 City of Hackensack

Wastewater flows from the City of Hackensack are primarily residential and flow by gravity to the BCUA Main (Hackensack Valley) Trunk Sewer, which transports flows to the WPCF in Little Ferry. The BCUA Trunk Sewers servicing the City of Hackensack are illustrated in Figure 1-3. Additional information on the combined sewer systems within the City of Hackensack can be found in the City of Hackensack Sewer System Characterization Report as submitted to the NJDEP.

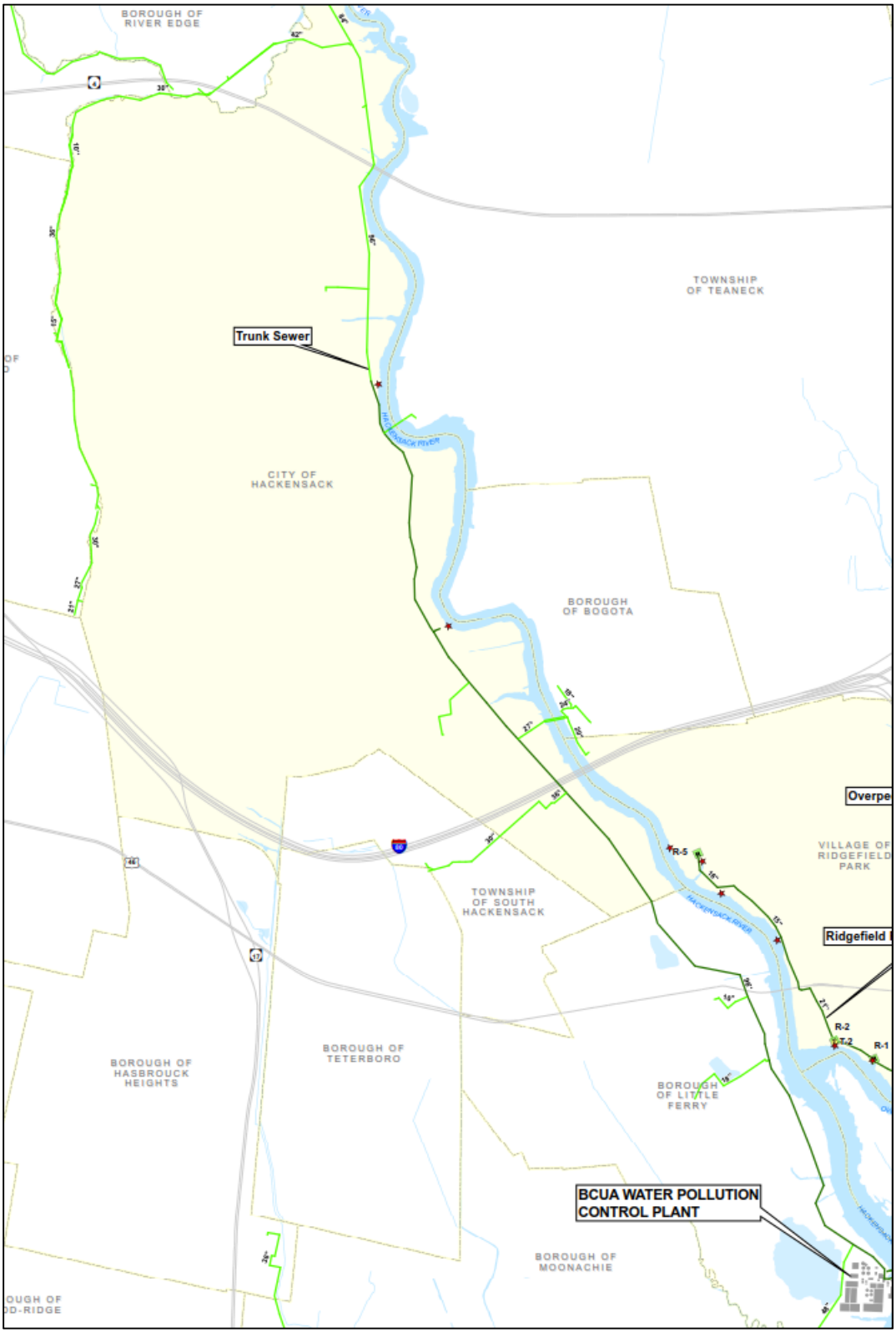

Figure 1-3: Trunk Sewers Servicing the City of Hackensack

1.2.1.3 Village of Ridgefield Park

Wastewater flows from the Village of Ridgefield Park are primarily residential and flow by gravity to a branch trunk sewer and then the BCUA Overpeck Trunk/Relief Sewers, which transport flows to the WPCF in Little Ferry. The BCUA Trunk Sewers servicing the Village of Ridgefield Park are illustrated in Figure 1-4. Additional information on the combined sewer systems within the Village of Ridgefield Park can be found in the Village of Ridgefield Park Sewer System Characterization Report as submitted to the NJDEP.

It should be noted that while the BCUA does not own nor operate any CSO Outfalls it does own and operate three of the CSO Control Facilities within the Village of Ridgefield Park as illustrated below. These are Regulators R-1, R-2, and R-5.

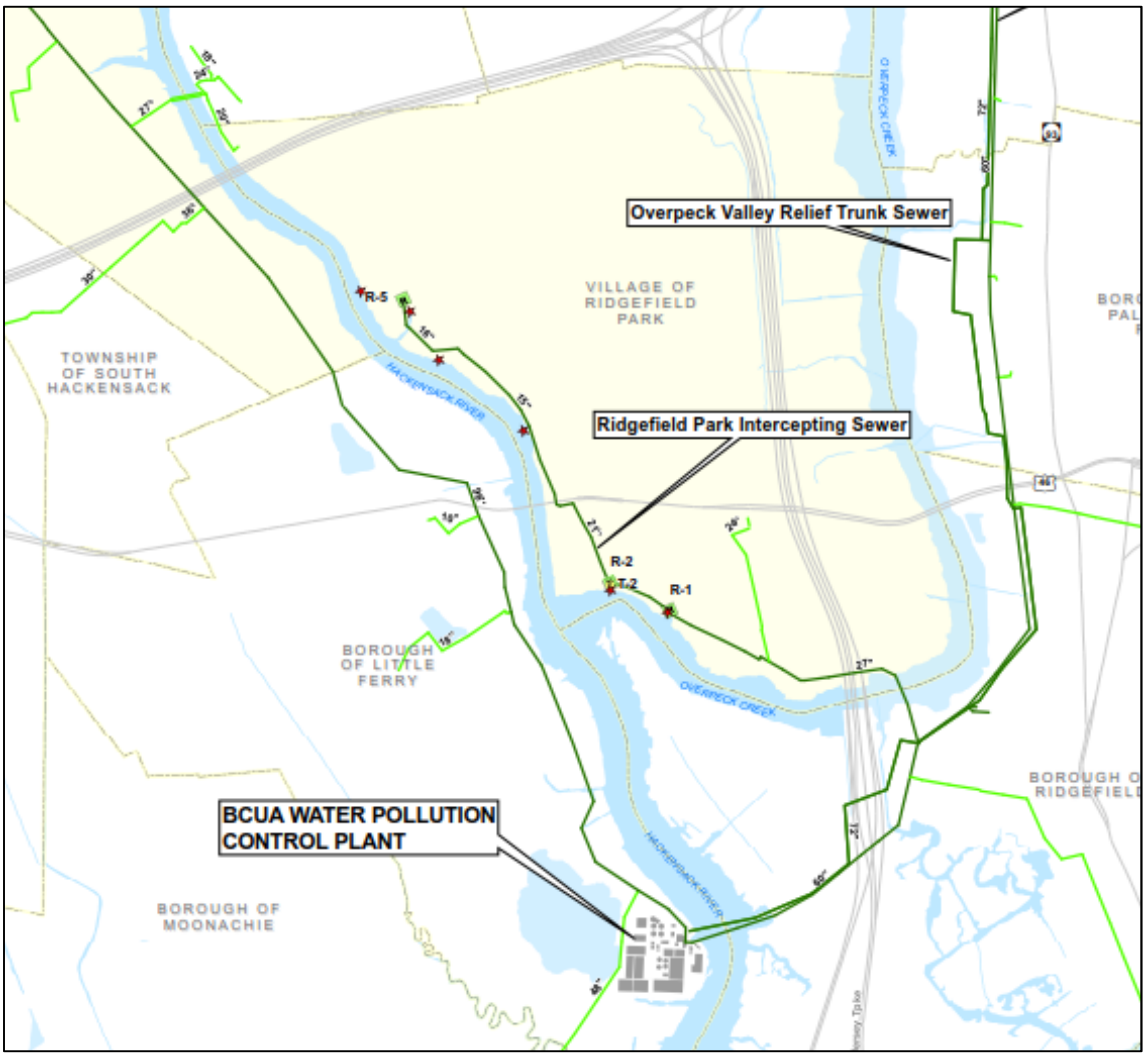

Figure 1-4: Trunk / Intercepting Sewers Servicing the Village of Ridgefield Park

1.2.2 Monitoring of Background Conditions

Extensive flow monitoring was conducted as part of the calibration/verification of the BCUA InfoWorks Model. It is noted that during the flow monitoring period the bottom segment of the Main and Overpeck

Creek Trunk Sewers all operated under surcharged conditions due to the hydraulic grades at the head of the plant in the wet well. It was also noted, that all temporary metering locations as previously discussed generally functioned under free flow conditions and did not surcharge even under wet weather flows as monitored. Additional information on dry weather and wet weather flows, as well as any excess capacity available within segments of the Trunk Sewer System are documented and discussed later in this report.

1.2.3 Model Calibration and Verification

The BCUA InfoWorks Model was calibrated and verified using time series data collected during the monitoring period. The most common way to evaluate if collection system model performance is acceptable, is through the application of individual storm event numerical criteria. This approach isolates individual storm events with distinct start and end times from a continuous model simulation and evaluates metrics such as model-meter agreement for peak flow, overall volume, and peak depth against a numerical standard. The most widely used standard of this kind comes from the Wastewater Planning Users Group (WaPUG) (now known as the Chartered Institution of Water and Environmental Management or CIWEM) Code of Practice for the Hydraulic Modeling of Sewer Systems. Table 1-1 summarizes the numerical calibration/validation criteria that are part of this standard.

Table 1-1: WaPUG Model Calibration/Validation Criteria

Category	Dry weather flow	Wet weather flow
Peak flow	±10%	+25% to -15%
Volume	±10%	+20% to -10%
Unsurcharged depth	±4 inches	±4 inches
Surcharged depth	N/A	+20 inches to -4 inches
Time of peaks	Within 1 hour	Similar

WaPUG requires that there be an adequate number of wet weather events for use in calibration and verification of the model. For the reporting of individual event statistics, a total of 15 storm events were identified (10 for calibration and 5 for validation). The overall goal was to obtain and utilize storms that represented a wide variety of durations, peak intensities, total rainfalls and that were distributed through different seasons, this goal was achieved. An estimate of typical return periods for storms of similar rainfall volume and peak rainfall intensity, noted that monitored rainfall return periods for volume fell between 0.2 – 15.9 months, while return periods for peak rainfall intensity ranged from 0.1 – 28.1 months. Accordingly, the calibration and verification were conducted using numerous storms of varying rainfall intensities and total rainfall volumes. Additional information on the storms used in the calibration and verification of the model is provided in Section 6 of the BCUA Sewer System Characterization Report dated February 2019.

1.2.3.1 Dry Weather Calibrations

The Base Sanitary Flow (BSF) diurnal patterns (weekday and weekends) developed from long term metered flow was assigned during the calibration period to all sub-catchments within a meter basin area with BSF flows allocated based on population fraction. Similarly, Ground Water Infiltration (GWI) flows were allocated based on pipe inch-mile fraction within the metered basin. Certain portions of the BCUA tributary network exhibited high GWI, while in other areas, the GWI was lower. Monthly varying GWI factors were assigned only in metered basins where highly variable GWI responses were identified and monthly varying factor were needed to get a reasonable match between flow metering data and the model

simulation results. Comparisons between predicted and observed DWF were quantified in tabular form using the criteria listed in Table 1-1 and visually through observed versus model predicted plots at each flow monitoring location. The results of the dry weather flow (DWF) calibrations are included in Section 8 of the BCUA Sewer System Characterization Report dated February 2019.

The monitoring period used for model calibration ranged from March to September 2017. While summers typically will provide an extended dry flow (DWF) period of several weeks within a typical year, there were few weeks during the monitoring period without significant rainfall. Two DWF periods were identified during the Spring and Fall of the 2017 flow monitoring period, which were used for DWF calibration. Comparing the model's DWF results to an extended dry weather period in the flow monitoring period is a more robust comparison than simply comparing selected DWF days pulled from the flow monitoring record. Two continuous DWF weeks were identified in April and September from the 2017 data. Each of these weeks consisted of at least a six-day continuous dry weather period that also include different seasons. For these different periods, the DWF calibration statistics from Table 1-2 were calculated. Table 1-2 provides an example of the calculation of these DWF statistics and is representative of the same tables computed for each flow meter that are provided in the characterization report.

Table 1-2: Example Model DWF Calibration/Validation Results

DWF calibration statistics – September 2017			
Category	Simulated	Measured	% Error
Peak Flow (Avg)	1.32	1.48	**-10.7%**
Volume	8.22	8.70	**-5.5%**
Depth (Avg)	3.18	2.94	**8.3%**

DWF daily peak flows (MGD)		
DWF day	Simulated	Measured
9/24/17	1.37	1.50
9/25/17	1.37	1.59
9/26/17	1.30	1.47
9/27/17	1.30	1.42
9/28/17	1.30	1.39
9/29/17	1.30	1.42
9/30/17	1.30	1.58

1.2.3.2 Wet Weather Calibration

Following completion of the DWF calibration, the wet weather flow (WWF) calibration was initiated. As discussed above as part of the calibration and validation process the continuous record of flow meter data was utilized. For the reporting of individual event statistics, a total of 15 storm events were identified (10 for calibration and 5 for validation). These event statistics, as well as the WWF peak flow versus peak flow and volume versus volume plots are included in Section 7 of the Characterization Report. As previously noted the return periods for volume fell between 0.2 – 15.9 months, while return periods for peak rainfall intensity ranged from 0.1 – 28.1 months.

Overall WWF calibration was completed with variable success with some locations calibrating successfully and others not fully meeting the calibration criteria. Meters 03-04 and 09-10 were locations on the Overpeck

Creek Trunk Sewer and the Overpeck Relief Sewer downstream from a junction chamber. Since flows co-mingled in the junction chambers; it was very difficult to model the flow split downstream of the chamber. Accordingly, the metering data from the two sites were summed to determine model calibration/verification since the total peak volume and flow can be well established. It should be noted that the since meters 03-04 and 09-10 were summed, the depth of flow criterion could not be evaluated. In addition, temporary Meter 1 was impacted by negative flows during dry and low flow wet weather period in the Overpeck Creek Trunk, which made calibration and verification impossible. Accordingly, the calibration performance totals in Table 1-3 were prepared to illustrate how well the overall mode calibration performance met the calibration goals without consideration of temporary Meter 1. The results are characterized as the number of storms that fall into various categories of compliance with the calibration/validation criteria listed below:

Table 1-3: 2017 WWF Calibration Performance Summary

Meter	0 of 2* or 3 of 3 criteria	1 of 2* or 2 of 3 criteria	0 of 2* or 1 of 3 criteria	0 of 3 criteria
Temp Meter 02	3	2	3	1
Temp Meter 03+04*	10	0	0	0
Temp Meter 05	1	3	5	1
Temp Meter 06	10	0	0	0
Temp Meter 07	8	1	1	0
Temp Meter 08	0	7	3	0
Temp Meter 09+10*	1	9	0	0
Totals	**33**	**22**	12	2
Totals (%)	**47.8%**	**31.9%**	**17.4%**	**2.9%**
Cumulative Totals (%)		79.7%	97.1%	100.0%
***Note: Sites Temp 03+04 and Temp 09+10 only have 2 criteria – peak flow and volume**				

Overall approximately 80% of the metering locations met the calibration criteria for at least two out of the three criteria, and 97% met at least one criterion. Less than 3% of the data points failed to meet the WaPUG criteria at all. These results fall within the WaPUG overall criteria to meet the individual criteria for two out of three storms.

1.2.3.3 Model Verification

Five storm events were selected for model validation. As for the calibration storm events, the selection of validation storms was made to get a wide sample of storm types and conditions that extended throughout the range of the flow monitoring period. Again, overall WWF validation was completed with variable success with some locations calibrating successfully and others not fully meeting the calibration criteria. As previously noted, temporary Meters 03-04 and 09-10 were located on the Overpeck Creek Trunk Sewer and the Overpeck Relief Sewer downstream from a junction chamber. Accordingly, the metering data from the two sites were summed to determine model calibration/verification since the total peak volume and flow can be well established. It should be noted that the since temporary Meters 03-04 and 09-10 were summed, the depth of flow criterion could not be evaluated. Again, temporary Meter 1 was impacted by negative flows during dry and low flow wet weather period in the Overpeck Creek Trunk, and thus was not included. Accordingly, the verification performance totals in Table 1-4 were prepared to illustrate how well the overall model performance met the validation goals without consideration of temporary Meter 1. The results are characterized as the number of storms that fall into various categories of compliance with the calibration/validation criteria listed in Table 1-4.

Table 1-4: Modified WWF Validation Performance Summary

Meter	0 of 2*, or 3 of 3 criteria	1 of 2*, or 2 of 3 criteria	0 of 2*, or 1 of 3 criteria	0 of 3 criteria
Temp Meter 02	1	1	1	0
Temp Meter 03+04*	4	1	1	0
Temp Meter 05	1	0	4	0
Temp Meter 06	5	0	0	0
Temp Meter 07	4	1	0	0
Temp Meter 08	0	3	2	0
Temp Meter 09+10*	2	3	0	0
Totals	**17**	**9**	**7**	**0**
Totals (%)	**51.5%**	**27.3%**	**21.2%**	**0.0%**
Cumulative Totals (%)		78.8%	100.0%	100.0%
***Note: Sites Temp 03+04 and Temp 09+10 only have 2 criteria – peak flow and volume**				

Overall, approximately 80% of the metering locations met the calibration criteria for at least two out of the three, or two out of two criteria, and 100% met at least one criterion. No model is perfect, however overall the verification process showed that the model does accurately represent the existing dry and wet weather flows within the BCUA transport system.

1.2.4 Receiving Water Quality Monitoring and Modeling

The development of the Baseline Compliance Monitoring Program was conducted as part of a cooperative agreement with the NJ CSO Group headed by the Passaic Valley Sewerage Commission (PVSC). As part of the NJ CSO Group collaboration, a field sampling and analytical testing program for existing ambient pathogen water quality conditions in the participating CSO permittees receiving waters was implemented. The NJ CSO Group Compliance Monitoring Program Report (CMP Report), dated June 2018, was submitted to the NJDEP by PVSC, as the lead organization of the NJ CSO Group. The CMP Report provided a full narrative of the implemented Baseline Compliance Monitoring Program (BCMP), including the program description; the field sampling and the field and laboratory analytical methods used; the data quality objectives; an evaluation of data completeness, precision, and representativeness; and presentations and discussion of the program results. The intent of the program was to generate adequate data to establish existing ambient water quality conditions for pathogens in the CSO receiving waters and to update, calibrate and validate a pathogen water quality model of the receiving waterbodies.

The BCMP involved baseline, source, and wet weather event sampling of waterbodies throughout the region, with three sampling locations situated at or near the Village of Ridgefield Park and Hackensack, including sampling stations maintained by the New Jersey Harbor Discharges Group (NJHDG). Baseline sampling was performed twice a month in May and June; weekly in July, August, and September; and monthly from October through April. Source sampling occurred at the same time as baseline sampling, but targeted major influent streams within the study area to establish non-CSO loadings. Event sampling was timed to coincide with rainfall, to capture 3 discrete wet-weather events, and was limited to select sampling locations.

The CMP Report organizes the baseline, source, and event sampling data by waterbody grouping, station number, and specific waterbody. A total of 23 baseline and source events were completed from April 2016 through March 2017, while the wet weather event sampling was completed across 4 sampling dates in June 2016, January 2017, and April 2017. All samples collected were analyzed for fecal coliform and enterococcus and samples from freshwater locations were also analyzed for E. coli. During field sampling, field measurements were also made for: temperature, salinity, dissolved oxygen, light penetration (secchi depth), and turbidity. Depending on the sampling location, samples were collected at either 1 or 2 depths. For event sampling, locations were typically sampled twice per day for 3 days.

Using information collected as part of the baseline monitoring, the NJ CSO Group developed a Pathogen Water Quality Model (PWQM) for consideration in the CSO LTCP alternatives selection and Compliance Monitoring Program. The PWQM was created to supplement ambient in-stream pathogen monitoring under the baseline CMP and facilitate the evaluation of CSO control approaches and alternatives. Water quality data collected under the baseline CMP and collection system characterization work was being used to calibrate and validate the PWQM. It is a computational model of the receiving waters that may be used to simulate bacteria concentrations in the waters of the New York-New Jersey Harbor complex under existing conditions and to assess the attainment of applicable water quality standards under potential future conditions. The PWQM will function as a validated tool that reasonably replicates the observed timing, duration, and magnitude of ambient pathogen data in the study area and that is capable of simulating pathogen water quality conditions for varying CSO strategies to assist in assessing the CSO impacts on ambient pathogen water quality. It is anticipated that this model will be used to evaluate the overall effectiveness of the Regional CSO LTCP.

1.2.5 Sensitive Areas Report Summary

1.2.5.1 General

The BCUA does not own nor operate any CSO Outfalls and thus did not need to conduct and complete a Sensitive Areas Report, however this is a requirement of the other permittees within the BCUA CSO Group. Consistent with the requirements of the National CSO Control Policy, the NJPDES CSO Permits stipulates that the highest priority must be given to controlling overflows to sensitive areas. The permits define sensitive areas as designated Outstanding National Resource Waters; National Marine Sanctuaries; waters with threatened or endangered species and their habitat; waters used for primary contact recreation (including but not limited to bathing beaches); public drinking water intakes or their designated protection areas; and shellfish beds. If a CSO outfall discharges to a sensitive area, the CSO outfall is to be eliminated or relocated wherever physically possible and economically achievable, and where elimination or relocation is not feasible, treatment of the overflow deemed necessary to meet water quality standards must be provided. The implementation schedule for the LTCP must also place the highest priority to controlling CSOs to sensitive areas.

A thorough assessment of the potential need for a higher prioritization of any specific CSO discharge location located in Fort Lee, Hackensack, and Ridgefield Park due to the presence of sensitive areas has been conducted. This work includes a detailed investigation of the subject waterbodies performed by the NJ CSO Group on behalf of the participating permittees, as described in the Identification of Sensitive Areas Report, dated June 2018, submitted by PVSC to the NJDEP. Additional considerations made by individual members, including incorporating discussions and input from the Supplemental CSO Teams, was also made.

1.2.5.2 Ridgefield Park and Hackensack Sensitive Area Analysis

Based on these examinations, it was determined that the Wanda Canoe Club utilizes Riverside Park in Ridgefield Park (behind the Department of Public Work (DPW) building). Canoeing (boating) is listed as a secondary contact and thus, while the area may be of interest from a public viewpoint it does not constitute a sensitive area under the permit. A review of the proximity of the launch area to surrounding outfalls from Hackensack (001A and 002A) and Ridgefield Park (006A, 005A, and 004A) shows that the closest outfalls are those in Ridgefield Park. Each of these outfalls (006A, 005A, and 004A) is located around 600 – 800 feet away from the launch area and thus not in the immediate area (within 100 feet) of the launch. In addition, this segment of the Hackensack River is tidal and thus the impact of these outfalls impact both upstream and downstream areas. The City of Hackensack and Village of Ridgefield Park will keep in mind the public concern of this area in its planning, however they both have determined that no sensitive areas exist within its receiving waters and community and no elevated or accelerated CSO controls for sensitive areas are required.

1.2.5.3 Fort Lee Sensitive Area Analysis

The sturgeon was identified as a threatened and endangered species in the Hudson River, which it the watercourse Fort Lee discharges into. The NJDEP identified outfalls discharging to the sturgeon habitat as sensitive in their Sensitive Areas Report approval letter dated April 8, 2019. However, discussions with the NJDEP are ongoing, as research provided in the NJ CSO Group sensitive areas report indicated the sturgeon was not sensitive to CSO discharges.

1.2.6 Characterization of System Performance

The EPA National CSO Control Policy indicates that CSO control alternatives be assessed on a “system-wide, annual average basis”. This can be accomplished by selecting a typical hydrologic year and using

the precipitation records for the period in a continuous simulation modeling application of the hydraulically connected system. The selection of a typical hydrologic record serves to provide a representative and unbiased prediction of average design rainfall conditions that incorporates the variability observed in the historical records. In conjunction with the NJ CSO Group, local historical rainfall data and storm patterns were analyzed and calendar year 2004 was selected as the typical year hydrologic dataset for the LTCP efforts by the NJ CSO Group permittees. With the submission of the Typical Hydrologic Year Report by PVSC on behalf of the NJ CSO Group, the NJDEP responded in May 2018 that the submitted report addressed all its questions and comments to its satisfaction.

As previously noted, the InfoWorksICM model can be used as a predictive tool to assess the existing sewer system performance under varying rainfall conditions. Accordingly, a model simulation was conducted using the BCUA InfoWorksICM model to develop overflow information for the Borough of Fort Lee, the City of Hackensack, and the Village of Ridgefield Park combined sewer systems using the historic precipitation data for 2004 from Newark Airport. The typical year model simulation for the current sewer systems provides detailed information on the response of each drainage basin within the system to individual rainfall volumes and intensities, and results in an estimated number of district wide overflow events and the total annual overflow volume from all CSO outfalls. The results from the analysis, including predictions of the number of overflow events, volume, and duration, and the peak flow occurrence by outfall location are summarized in Table 1-5 below. The durations noted are the predicted total cumulative time of CSO discharge through the year for the indicated outfall location. The system-wide event counts, durations, and peak flows are the predicted maximum values observed across all outfalls for the typical year.

Table 1-5: Typical Year Existing Condition Annual CSO Characterization by Outfall

Municipality	Outfall No.	Outfall Name	Annual Total			Maximum
			No. Overflow Events	Overflow Volume (MGl)	Duration (hours)	Peak Flow (MGD)
Fort Lee	001A		58	82.5	Not reported	Not reported
Fort Lee	002A		20	4.7	Not reported	Not reported
	Municipal-wide Total		**not appl.**	**87.2**	**not appl.**	**Not appl.**
	Municipal-wide Maximum		**58**	**82.5**	**not appl.**	**Not appl.**
Hackensack	001A	Anderson Street	76 days	105.3	Not reported	Not reported
Hackensack	002A	Court Street	76 days	151.3	Not reported	Not reported
	Municipal-wide Total		**not appl.**	**256.6**	**not appl.**	**Not appl.**
	Municipal-wide Maximum		**76 days**	**151.3**	**not appl.**	**Not appl.**
Ridgefield Park	001A	Bergen Turnpike	19	6.3	70	19.4
Ridgefield Park	002A	Main Street and Bergen Turnpike	13	0.4	11	6.2
Ridgefield Park	003A	Christie Street	45	15.3	300	31.9
Ridgefield Park	004A	Mount Vernon Street	53	24.1	614	56.3
Ridgefield Park	005A	Industrial Avenue	25	3.6	121	5.0
Ridgefield Park	006A	Hackensack Avenue	12	0.5	39	2.6
	Municipal-wide Total		**not appl.**	**50.3**	**not appl.**	**Not appl.**
	Municipal-wide Maximum		**56**	**24.1**	**614**	**56.3**
BCUA District	**District-wide Total**		**not appl.**	**394**	**not appl.**	**Not appl.**
BCUA District	**District-wide Maximum**		**not appl.**	**151.3**	**not appl.**	**Not appl.**

1.3 Approach to Development and Evaluation of Alternative Controls

The development and evaluation of alternative controls is the initial step in the planning process needed to develop the site-specific projects proposed for implementation under the LTCP. There are a large variety of control methods that could be utilized to reduce or eliminate discharges from the combined sewer system and this report represents the screening process and evaluative methods used to reduce the number of control alternative that will be ultimately evaluated for implementation. It should be noted that the information contained within this report is preliminary and does not represent the Long-Term Control Plan. Rather the information contained herein will be used over the next eleven months to develop a more detailed evaluation of the recommended alternatives, to choose those that will be ultimately implemented.

1.3.1 Report Goals and Objectives

The USEPA recommends a three-step sequence for the development of alternatives:

1. Definition of water quality objectives.

2. Definition of a range of CSO control goals to meet the CSO component of the water quality goals.
3. Development of alternatives to meet the CSO control goals.

The USEPA expects the long-term CSO control plan to evaluate controls that will ultimately result in compliance with water quality standards (WQS) and suggests no more than four overflows per year or 85% capture by volume of CSO discharges. However, the solution will also depend on the permittees ability to fund these projects. Accordingly, it suggests it would be prudent to evaluate various levels of control such as, but not limited to achieving zero overflow events per year, an average of 1 to 3, 4 to 7, and 8 to 12 overflow events per year. Alternatively, the long-term plan could evaluate controls that achieve 100% capture, 90% capture, 85% capture, 80% capture, and 75% capture for treatment. The water quality-based requirements of the Clean Water Act can be met using either the Presumption Approach or the Demonstration Approach in the evaluation of alternatives.

The “Presumption” Approach refers to a program that is presumed to meet WQS using the following criteria for combined sewer flows remaining after the minimum treatment of primary clarification, solids and floatables disposal, and disinfection of effluent, if necessary. The Presumption Approach requires the following:

- No more than an average of four overflow events per year occurs from a hydraulically connected system as the result of a precipitation event. The Department may allow up to two additional overflow events per year.
- Elimination or the capture for treatment of no less than 85% by volume of the combined sewage collected in the Combined Sewer System (CSS) during precipitation events on a hydraulically connected system-wide annual average basis.
- Elimination or removal of no less than the mass of the pollutants, identified as causing water quality impairment.

The “Demonstration” Approach refers to a program that uses a receiving water model to meet the water quality-based requirements of the Clean Water Act taking into consideration the following:

- The planned control program is adequate to meet WQS and protect designated uses, unless WQS or uses cannot be met because of natural background conditions or pollution sources other than CSOs.
- The CSO discharges remaining after implementation of the planned control program will not preclude the attainment of WQS or the receiving waters’ designated uses or contribute to their impairment.
- The planned control program will provide the maximum pollution reduction benefits reasonably attainable.
- The planned control program is designed to allow cost effective expansion or cost-effective retrofitting if additional controls are subsequently determined to be necessary to meet WQS or designated uses.

This Development and Evaluation of Alternatives Report has been created to provide a comprehensive review and analysis of a range of CSO control strategies to meet the CSO component of the water quality goals. Per the requirements of CSO NJPDES Individual Permits, the list of alternatives that have been evaluated includes:

- Green infrastructure (CSO municipalities)
- Increased storage capacity in the collection system (Regional Authority and CSO municipalities)

- Sewage Treatment Plant (STP) expansion and/or storage at the plant. This includes evaluation of the capacity of each of the unit processes or treatment trains to determine the amount of CSO discharge reduction that could be achieved by utilizing additional treatment capacity while maintaining compliance with all permit limits. (Regional Authority)
- Inflow/Infiltration (I/I) reduction to free up storage capacity or conveyance in the sewer system and/or treatment capacity at the STP (CSO municipalities)
- Sewer separation (CSO municipalities)
- Treatment of the CSO discharge (CSO municipalities)
- CSO related bypass of the secondary treatment portion of the STP (Regional Authority)

As noted above, some of these alternative controls will need to be evaluated by the individual CSO municipality (Fort Lee, Hackensack, or Ridgefield Park), and others by the BCUA. This report utilizes the information gathered as part of the System Characterization to develop and evaluate the range of CSO control alternatives listed above. The hydrologic and hydraulic model reviewed by the NJDEP has been used to simulate the existing conditions as well as the response of the system under each of the alternatives after construction and operation. The CSO control alternatives including constructing and implementing various remedial controls, and combination of such controls and activities, were evaluated according to their practical and technical feasibility and water quality benefits.

The program objectives addressed herein are:

- To evaluate an appropriate range of CSO control alternatives, including source controls, collection system controls, storage technologies and treatment technologies.
- To develop project cost and operation and maintenance costs for control alternatives.
- To evaluate the full range of alternatives with respect to meeting water quality standards and protecting designated uses.
- To organize the evaluation of controls in a technical framework and approach that is understandable and consistent with the CSO Control Policy and EPA Guidance.

The program goal is to develop a range of control alternatives that are capable of cost-effectively improving water quality within the impacted receiving waters, and that could be further evaluated for implementation under the Regional CSO LTCP. The contents of this report collectively relate to each of these goals and objectives and provide the information necessary for the members of the BCUA CSO Group to advance the LTCP process to the Selection and Implementation of Alternatives step.

2 General Information

2.1 Public Participation Process

Public outreach and input are an important component of the LTCP progress, and the project team has endeavored to provide opportunities for public education and awareness, as well as to gain feedback on the CSO control alternatives. Below is a summary of activities undertaken by the BCUA CSO Group. Efforts by the individual municipalities (Fort Lee, Hackensack, and Ridgefield Park) are documented in their individual Reports. This section only covers activities since the Public Participation Process report was last submitted on January 04, 2019, and prior activities are documented in that report. Much of the subsequent outreach took place through quarterly meetings of the BCUA CSO Group Supplemental Team; however individual permittees may have conducted additional outreach on their own.

Mott MacDonald presented on the progress of the alternatives evaluation to the BCUA CSO Group's Supplemental Team on March 12, 2019. The presentation provided an update to the Team on the status of the overall project, alternatives to be considered, and where those alternative controls will be located. A follow up presentation providing more specifics on alternative controls and their locations and anticipated costs was presented to the Team on May 15, 2019. To advertise the event, the members posted an announcement on indiviudal websites and copies of the event flier were distributed to the entire SCSO Team and posted in member municipal buildings and other public locations. Any members of the public that attended heard an overview of the potential CSO control alternatives and presentations by each of the members of the CSO group. The presentation slides were included in the meeting minutes, which were posted to the BCUA website [https://www.bcua.org/index.asp?SEC=AEBEC7FF-B96F-485D-B704-7E9E888905A0&Type=B_BASIC].

BCUA intends to continue the coordination of the Supplemental CSO Team and has undertaken efforts to expand public participation by extending invitations for additional members. BCUA also plans to provide assistance to members of the BCUA CSO Group in the coordination and development of additional municipal meetings, and in developing additional information for publication on member websites.

To further advance the public involvement, minutes and presentations for all prior meetings were posted to the BCUA website. In addition to the quarterly SCSO Team meetings, the BCUA provided a rain barrel and literature to the Village of Ridgefield Park for their May 11, 2019 Earth Day public event at Riverside Park.

2.2 Approximated Future Conditions

The model was developed in 2018 and needs to be applicable during the entire planning period. It has been assumed that the alternative controls that are selected through the LTCP process will be constructed and implemented over a 30-year period. As such, the year 2050 has been selected as the future baseline condition for both the CSS municipalities and the District.

2.2.1 Projections of Population Growth

The Permit requires the permittee to simulate "conditions as they are expected to exist after construction and operation of the chosen alternative(s)" (Part IV.G.4.e). The intent is to reduce the risk that foreseeable changes in the community, or in this case the BCUA District (District), will reduce the effectiveness of the proposed regional LTCP facilities. The impact of population growth within the individual CSS municipalities will be reported on in their individual reports. Accordingly, the review of

population and commercial / industrial growth in this discussion will be limited to wastewater flows to the BCUA Little Ferry Water Pollution Control Facility (WPCF)

An evaluation of anticipated changes to the District's population was undertaken. Discussions were also held with the BCUA to document any new large commercial /industrial developments planned within the District, and whether planned changes were anticipated in the size or scope of the service district.

It is noted that there is a great deal of uncertainty in future projections and that as the planning horizon increases the uncertainty increases dramatically. This is evidenced below in cases where a variety of reputable sources produce differing projections. The goal was to select future conditions that would be a reasonable, yet conservative, estimate of likely future conditions. It is noted that actual future conditions could vary substantially due to demographic trends, economic conditions, changes in technology, climate impacts and myriad other influences beyond the control of the Permittee.

Population data for Bergen County was analyzed to evaluate population growth within the District. The Bergen County Wastewater Management Plan notes that approximately 62% of the County population is tributary to the BCUA WPCF in Little Ferry. Population projections are just that, projections and accordingly, several population projections methods were sourced to select a reasonable projection for the future baseline. These are summarized as follows:

2.2.1.1 U.S. Census Bureau

The United State Census Bureau is considered an authoritative source for population data. Data is available from the 1970, 1980, 1990, 2000 and 2010 censuses, as well as population projections through 2018, see Table 2-1.

Table 2-1: US Census Population Data

Year	Population
1970	897,148
1980	845,385
1990	825,380
2000	884,118
2010	905,116
mid-2018 (US Census Estimate)	936,692
2050 Projection	1,064,330
% BCUA District Population	62%
Projected BCUA District Population	659,880

Sources: https://www.co.bergen.nj.us/data-resources-and-technology/census-data
https://www.census.gov/quickfacts/fact/table/bergencountynewjersey,US/PST045218

As can be seen from the data, the population in the County dropped between 1970 and 1990 but has been increasing since. The annual population growth between 1990 and 2018 averaged approximately 0.39% per year, while the projected annual population growth between 2010 and 2018 is estimated at approximately 0.41% per year. To project the population to 2050, it was estimated that growth within the County would average 0.40% per year throughout this period. Accordingly, the projection from Census data indicates that the 2050 population for Bergen County would be around 1,064,330 persons. Using the 62% of Bergen County population being serviced by the BCUA it is projected that the population within the BCUA Service District would grow to 659,880 people as noted above.

2.2.1.2 North Jersey Transportation Authority (NJTPA)

The NJTPA is a metropolitan planning organization with federal authorization. It is responsible for the 13 northern counties in New Jersey and is responsible for overseeing certain transportation related projects and studies. The NJTPA updates its regional forecasts for population, households and employment every four years.

In 2017, NJTPA completed the latest set of forecasts. Final forecasts were approved by the NJTPA Board on November 13, 2017 and extend to 2045. The NJTPA employs the Demographic and Employment Forecast Model (DEFM). According to their website:

> The DEFM uses regional and county level forecasts of employment, population and households produced from a regional econometric modeling effort and allocates these forecasts to a localized Traffic Analysis Zone (TAZ) level. It also aggregates the TAZ level information to the municipal level. The DEFM uses data elements that influence location behavior to perform this allocation analysis including:
>
> - Current land use data (residential, commercial, industrial and vacant land);
> - Composite zoning estimates for density;
> - Highway and transit accessibility;
> - Historical growth; and
> - Known project developments.

The NJTPA forecasts strong growth in the population of Bergen County, but as noted above the NJTPA forecasts only extend to 2045 and fall short of the 2050 planning period. The forecast, which was extended to 2050, is summarized in Table 2-2 below, as can be seen the results are similar to the Census extrapolation:

Table 2-2: NJTPA population projections

Description	2010 Census	NJTPA Projection 2015	NJTPA Projection 2045	Projection 2050
Bergen	905,143	928,736	1,030,502	1,049,459
BCUA WMP % of Overall Population	62%	62%	62%	62%
Estimated BCUA District Population	**561,190**	**575,820**	**638,910**	**650,660**

Source: https://www.njtpa.org/data-maps/demographics/forecasts.aspx

2.2.1.3 New Jersey Department of Labor

Population and labor force projections on a county-wide basis have been developed by the New Jersey Department of Labor extending to 2034 as shown in Table 2-3. It is worthwhile to note that the estimated population for July 2014 as noted is just slightly less than the mid-2018 population estimate from the US Census Bureau of 936,690 persons. A review of the data indicates that the Department projects an

annual population growth of about 0.75% per year, which is about double rate estimated from the Census Bureau data during the 2010-1018 period.

Table 2-3: NJ Department of Labor Population Projection

County	Census Estimates		Projections to July 1,			
	4/1/2010	7/1/2014	2019	2024	2029	2034
Bergen	905,116	933,600	967,500	998,700	1,031,500	1,065,500

Source: https://nj.gov/labor/lpa/dmograph/lfproj/lfproj_index.html

To obtain an estimated population for 2050, we assumed that BCUA and Bergen County will grow at the same rate from the period of 2034 to 2050 and that the percentage of population serviced within the County would remain at 62%. The projected county population was extrapolated to 2050, which yields the following estimates shown in Table 2-4.

Table 2-4: New Jersey Department of Labor population estimates for Bergen County

	Estimated Population by Year			
Description	2010	2014	2034	2050
Bergen County	905,116	933,600	1,065,500	1,200,207
BCUA % of Overall Population	62%			62%
BCUA District Population	**562,190**			**745,480**

Based on the NJ Department of Labor population estimates it is projected that the 2050 population within the BCUA Service District would be 745,480 people. This estimate is not in agreement with other projections and thus is suspect.

2.2.1.4 BCUA Wastewater Management Plan Projections

The Bergen County Utilities Authority provides wastewater service to approximately half of Bergen County including the other members of the BCUA CSO Group. The Authority updated their Wastewater Management Plan (WMP) in 2014 to plan for anticipated changes to their service area and population over the next 20 years (2013 – 2033). As part of this planning document the BCUA made projections on population increases that may occur within their service area, in addition to new areas that may be serviced. The population growth projection data from the WMP was expanded to estimate the population projections to 2050 as illustrated in Table 2-5. Note, the BCUA has initiated an update to the BCUA WMP which will estimate future flows, however, the new WMP is not complete at this time, but will be applicable to future LTCP work.

Table 2-5: Interpolation of WMP Projected Population Growth to 2033

Present (2013) Population Served:	Municipality	20-Year Future (2033) Population Served:	Extension to 2050 Projected Population
341	Alpine	341	376
26,982	Bergenfield	28,462	31,364
8,266	Bogota	8,773	9,667
6,219	Carlstadt	6,812	7,507
23,780	Cliffside Park	25,047	27,601
8,508	Closter	9,428	10,389
8,675	Cresskill	9,375	10,331
4,952	Demarest	5,432	5,986
17,608	Dumont	18,461	20,343
5,683	East Rutherford	6,144	6,770
0	Edgewater	12,744	14,043
7,444	Emerson	7,737	8,526
27,564	Englewood	30,324	33,416
5,352	Englewood Cliffs	5,832	6,427
13,961	Fairview	14,768	16,274
34,196	Fort Lee	40,303	44,412
43,528	Hackensack	46,981	51,771
4,741	Harrington Park	5,281	5,819
11,909	Hasbrouck Heights	13,156	14,497
3,464	Haworth	4,024	4,434
10,418	Hillsdale	11,738	12,935
9,075	Leonia	9,975	10,992
10,715	Little Ferry	11,282	12,432
9,736	Maywood	10,909	12,021
7,973	Montvale	8,860	9,763
2,778	Moonachie	3,231	3,560
16,580	New Milford	18,180	20,034
0	North Bergen	101	111
4,704	Northvale	5,131	5,654
5,800	Norwood	6,400	7,053
5,837	Old Tappan	6,417	7,071
8,049	Oradell	8,509	9,377
19,803	Palisades Park	21,023	23,166
26,777	Paramus	29,690	32,717
8,823	Park Ridge	9,976	10,993
11,208	Ridgefield	12,395	13,659
12,864	Ridgefield Park	13,757	15,160
3	Ridgewood	3	3
11,506	River Edge	12,613	13,899
9,712	River Vale	10,059	11,085
5,607	Rochelle Park	6,120	6,744
10,096	Rutherford	11,181	12,321
0	Saddle River	0	0
2,360	South Hackensack	2,702	2,977
40,303	Teaneck	43,790	48,255
14,611	Tenafly	15,418	16,990
72	Teterboro	85	94
8,839	Washington	9,523	10,494
11,064	Westwood	12,091	13,324
5,848	Woodcliff Lake	6,635	7,311
7,857	Wood-Ridge	9,370	10,325
562,190	**Total**	**626,590**	**690,470**
	2050 Population Projections without Edgewater		**676,430**

While the Town of Edgewater is not currently within the existing Little Ferry WPCF service district the BCUA anticipates that the Edgewater WPCF will be abandoned in the future and that the wastewater flows will be pumped to the Overpeck Interceptor near Victoria Terrace in Ridgefield approximately 3,000 feet upstream from the head end of the Little Ferry WPCF. Accordingly, the WMP includes the population of Edgewater in its future population projections. Nevertheless, it is anticipated that the per capita flow

from the Edgewater Service Area will be significantly different from the BCUA Service Area based on existing Edgewater WPCF records. Accordingly, the future flow projections for Edgewater will be based on existing meter data with the assumption that future control of extraneous flows will make up for any population increase. Accordingly, it is projected that the BCUA Service District population will be 676,430 people.

2.2.2 Summary of Anticipated Population Growth

Mott MacDonald considered various sources of data on population growth as outlined above to determine a reasonable increase in population within the BCUA Service District during the thirty-year planning period for the Regional CSO LTCP. Overall four sources of population growth were used and extrapolated to 2050 to provide a reasonable understanding of the overall potential population growth within the County. The projections ranged from a low of 650,660 people based on further extrapolation of projections from the NJTPA to a high of 745,480 people based on further extrapolation of projections from the NJ Department of Labor. A conservative approach would be to take all four population projections as summarized in Table 2-6 and to average those for planning purposes. As previously noted the 0.75% annual increase from the NJ Department of Labor is much higher than projections from other methods and does raise of overall average. Accordingly, the three results which were similar were averaged to produce a reasonable projection of the 2050 population.

Table 2-6: Summary of Population Projections to 2050

Data Source	Conservative Projected Population 2050 (people)	Average Projected Population 2050 (people)
NJTPA	650,660	650,660
US Census Projection	659,880	659,880
NJ Department of Labor	745,480	
BCUA WMP Extended Projections	676,430	676,430
Average	**683,110**	**662,320**

The population increases within the BCUA District needed to be distributed within the BCUA InfoWorks model in some reasonable manner. The existing model uses populations within each drainage area (essentially municipal service areas) to establish per capita flows for dry weather. As noted, the BCUA WMP population projections as illustrated in Table 2-5 are also based on individual municipalities and thus using this data, which falls below the projected average based on a conservative approach, and above the average if we neglect the NJ Department of Labor approach in Table 2-6, is reasonable. Accordingly, modeling conducted to evaluate future conditions will be based on extrapolating the BCUA WMP population projections by municipality, as noted in Table 2-5. In summary the population is projected to increase from 562,190 to 677,430, resulting in an additional 114,240 residents.

2.2.3 Other Future Wastewater Flows

As previously mentioned it is anticipated that the Edgewater WPCF will be abandoned at some point in the future and the wastewater flows conveyed to the BCUA Little Ferry WPCF. Information obtained from the BCUA indicates that the Edgewater Pumping Station will be designed for an average daily flow of 4 MGD and a peak wet weather flow of around 18 MGD under wet weather conditions. The drainage area associated with the Edgewater WPCF has been incorporated into the BCUA InfoWorks modeled for future

baseline condition to reflect these conditions. This was done by assigning a typical diurnal pattern from a similar area to the 4 MGD average dry weather flow. To replicate wet weather conditions RTK values were assigned to produce a peak flow of 18 MGD during the typical year.

In addition to the Edgewater WPCF the American Dream Complex, a year-round family entertainment center near Giants Stadium in the Meadowlands, is scheduled to partially open soon. It is projected by the BCUA that when this complex is fully functional that it will contribute an average daily flow of approximately 900,000 gpd. The American Dream Complex discharges to the Southwest Interceptor and does not mix with combined flows until it reaches the WPCF. These flows have been anticipated as future flow conditions and entered into the BCUA model. The approximate locations where these two future wastewater inputs will develop is illustrated in Figure 2-1.

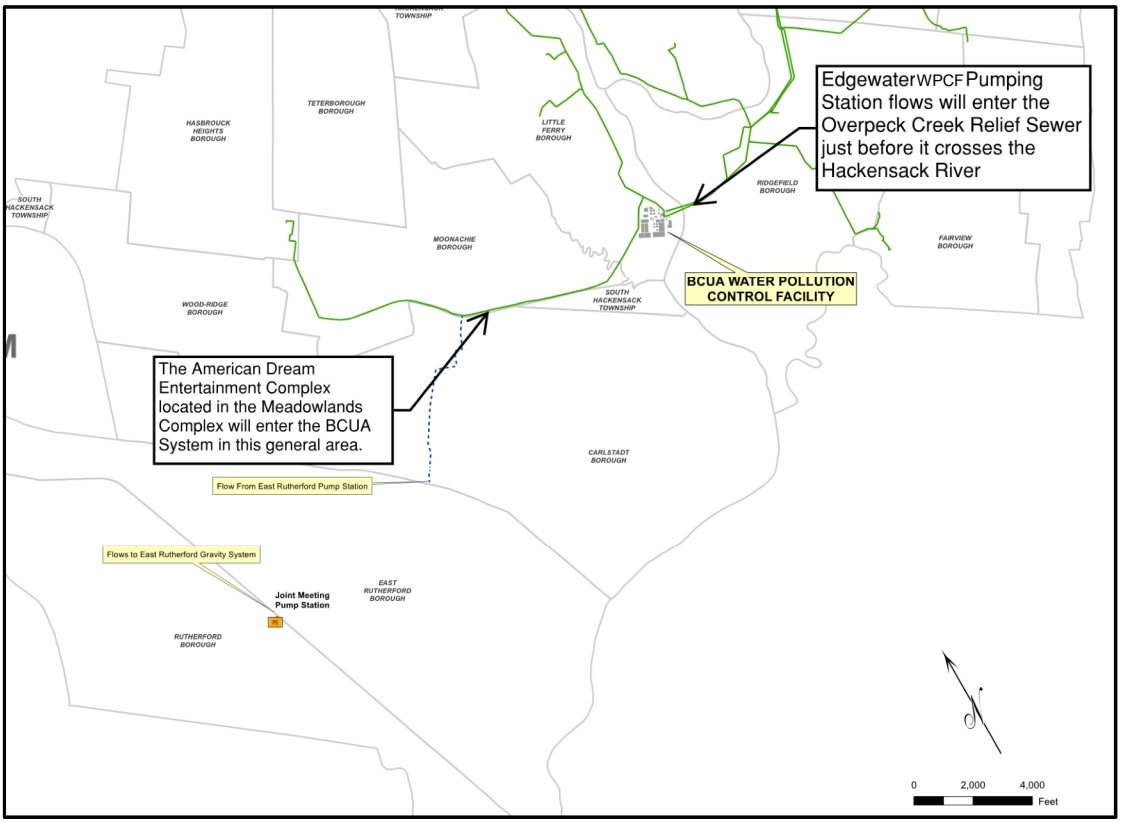

Figure 2-1: Future Major Wastewater Inputs to the BCUA WPCF

2.2.4 Summary of Future Conditions

Overall it is estimated that the average daily flow at the Little Ferry WPCF will increase by an estimated 12.0 MGD, see Table 2-7, by the year 2050 as noted in the below table. The average daily flow treated at the Little Ferry WPCF in 2014 was 77.3 MGD, which will bring the projected wastewater flows at the Little Ferry WPCF in 2050 to 89.3 MGD.

Table 2-7: Summary of future baseline flow increases

New Wastewater Source	Projected Flow Increases to Little Ferry WPCF (MGD)
2050 Population Growth (114,240 people@65 gpcpd)	7.1
Edgewater WPCF	4.0
American Dream Complex	0.9
Total	**12.0**

3 Water Quality Objectives

3.1 Applicable Water Quality Standards

The NJDEP has established water quality standards for each of the receiving waters within the State of New Jersey based on their intended use and whether they are freshwater or saline waters. The standards are based on both bacterial and physical/chemical standards such as levels of dissolved oxygen, turbidity, nutrients, pH, etc. Discharges from combined sewer overflows contribute pathogens, and thus the parameter of interest for CSOs is the bacterial standards. Bacterial standards are typically set with monthly geometric mean and single sample maximums set at levels to protect the watercourse's primary or intended use. The following outlines the bacterial standards and protected uses for each water quality classification:

Class	Description	Bacterial Standards	Monthly Geometric Mean	Single Sample Max	Protected Uses
SC	Saline Ocean	Entero	35	104	Primary Contact, Shellfishing
SE1	Saline Estuary	Entero	35	104	Primary Contact
SE2	Saline Estuary	Fecal	770	NA	Secondary Contact
SE3	Saline Estuary	Fecal	1500	NA	Secondary Contact
FW2	Fresh Water	E. coli	126	235	Primary Contact and Public Water Supply

The BCUA Service District includes three combined sewer communities. The City of Hackensack and the Village of Ridgefield Park are both located in the Hackensack River Watershed (HUC-11 02030103180 – Hackensack River below and including Hirshfeld Brook), which is in State Watershed Management Area 5. The third combined sewer community within the BCUA Service District is the Borough of Fort Lee. Fort Lee is within the Hudson River drainage basin and their CSO overflow pipes discharge to the Hudson River. The Hudson River in the region of Fort Lee has a water quality classification of SE-2.

The combined sewers from the City of Hackensack discharge into the Hackensack River which has a surface water quality classification of Saline Estuary one (SE-1) at the outfalls. The Village of Ridgefield Park is situated on a ridge that divides two sub-watersheds, to the east is the Overpeck Creek Watershed and to the west is the Hackensack River. The Village of Ridgefield Park combined sewers discharge to two watercourses, the Hackensack River (SE1) on the west side of the Village and the Overpeck Creek which has a surface water classification of Freshwater 2, Non-Trout and Saline Estuary 2 (FW2-NT/SE2) on the south side of the Village (Figure 3-1). There is a tide gate structure on the Overpeck Creek at the New Jersey Turnpike (Route 95). The combine sewer discharges are downstream of this point in the tidal (saline estuary) portion of the creek into the Overpeck Creek (SE2).

The saline regions of the Hackensack River appear on the NJDEP's 303(d) List of Water Quality Limited Waters primarily for chemical and pesticide contamination including, but limited to: Dioxin, Heptachlor epoxide, PCB in Fish Tissue, DDT and its metabolites in Fish Tissue, Mercury in Fish Tissue, Chlordane in Fish Tissue, and Dieldrin issues extending the full length of the Estuary from its confluence with Newark Bay up to the USGS river gage at the Oradell dam. These contaminants impact the designated use of fish consumption for SE1, 2, and 3 classified waters. In addition, low dissolve oxygen levels that impact

aquatic life have been detected primarily in the SE-3 designated region, and high enterococcus levels in the SE1 region of the river, which includes the reach along Hackensack and Ridgefield Park.

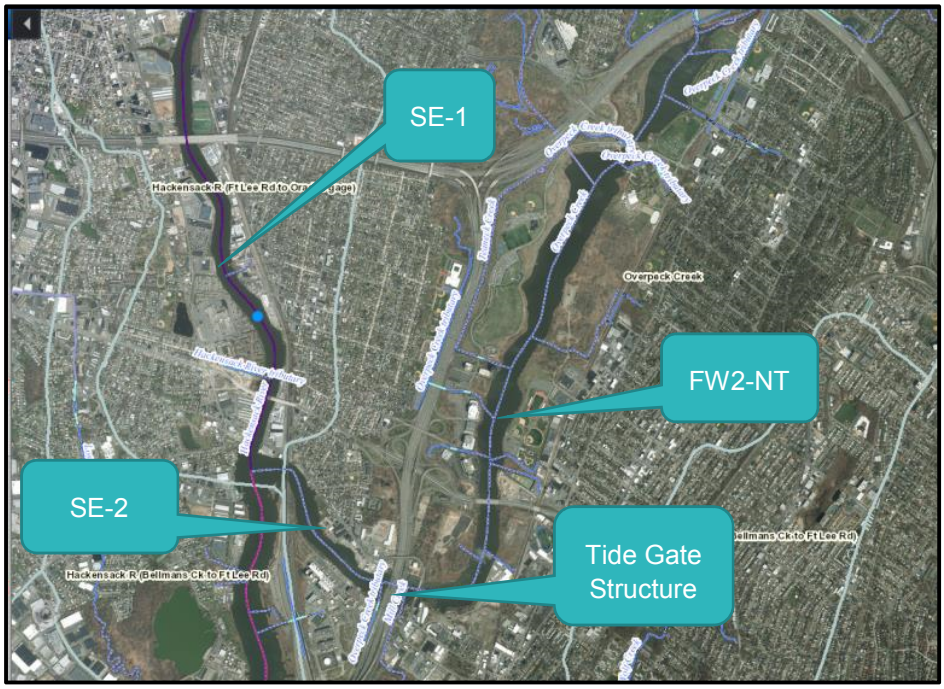

Figure 3-1: WQS Around Hackensack and Ridgefield Park

Overpeck Creek is also listed on the NJDEP 303(d) List of Water Quality Limited Waters for chemical and pesticide contamination including Chlordane in Fish Tissue, DDT and its metabolites in Fish Tissue, Dioxin, and PCB in Fish Tissue, all of which impact the designated use of fish consumption for SE2 and FW2 waters. In addition, high levels of Escherichia coli (E-coli) were also detected in the FW-2 section of the Creek, which impacts recreation within the region. It should be noted that there are no CSO discharge points within the fresh-water segment of Overpeck Creek.

3.2 Existing CSO Control Alternatives

The BCUA does not own nor operate any CSO Outfalls. Fort Lee, Hackensack, and Ridgefield Park are currently in compliance with all NJPDES Permit requirements. Solids/Floatables Control facilities were constructed at all CSO outfalls and have been well maintained and operated since then. Other existing CSO controls that have been implemented by individual municipalities, if any, will be documented in their individual reports.

3.3 Range of CSO Goals Being Evaluated

The EPA CSO Control Policy and the BCUA CSO Group Individual NJPDES Permits require that the CSO Long Term Control Plan be developed using either the Presumptive or Demonstrative approach as outlined below.

3.3.1 Presumptive Approach

The "Presumption" Approach refers to a program that, once fully implemented, is presumed to meet WQS. The Presumption Approach requires the following:

- No more than an average of four overflow events per year occurs from a hydraulically connected system as the result of a precipitation event. The Department may allow up to two additional overflow events per year.
- Elimination or the capture for treatment of no less than 85% by volume of the combined sewage collected in the CSS during precipitation events on a hydraulically connected system-wide annual average basis.
- Elimination or removal of no less than the mass of the pollutants, identified as causing water quality impairment.

3.3.2 Demonstration Approach

The "Demonstration" Approach refers to a program that uses a receiving water model to meet the water quality-based requirements of the Clean Water Act taking into consideration the following:

- The planned control program is adequate to meet WQS and protect designated uses, unless WQS or uses cannot be met as a result of natural background conditions or pollution sources other than CSOs.
- The CSO discharges remaining after implementation of the planned control program will not preclude the attainment of WQS or the receiving waters' designated uses or contribute to their impairment.
- The planned control program will provide the maximum pollution reduction benefits reasonably attainable.
- The planned control program is designed to allow cost effective expansion or cost-effective retrofitting if additional controls are subsequently determined to be necessary to meet WQS or designated uses.

This Development and Evaluation of Alternatives Report has been created to provide a comprehensive review and analysis of a range of CSO control strategies to meet the CSO component of the water quality goals.

Since the BCUA does not own nor operate any CSO discharges, its approach to the LTCP is different in that it will coordinate and review with its member municipalities as to the options available and work with them to develop a Regional LTCP. While preliminary data from the Baseline Compliance Monitoring Program and Model Development undertaken by the NJ CSO Group (which includes members of the BCUA CSO Group) is available, it is not finished. Preliminary data does indicate that some regional water bodies are currently meeting existing water quality standards, while others are not. Exceedances are not however restricted to combined sewer communities or their area of impact within tidal regions.

While preliminary investigations using the predictive mode of the model have been undertaken, additional information is needed to bring confidence to the results. Accordingly, investigations on the relative impact of partial or complete elimination of all CSO discharges within the Gateway Region (Passaic River, Hackensack River, Newark Bay, and Overpeck Creek) will not be completed within the framework of this report. The NJPDES permit requires the selection of either the Presumptive or Demonstrative Approach to the LTCP with the selection reasoning detailed within this report, the members of the BCUA CSO Group

are requesting that that decision, as to which approach will be taken, be delayed until submission of the final LTCP. This will afford the NJ CSO Group to undertake additional refinement of its river model data and scenarios prior to making that decision. It is also consistent with EPA guidance as described in the EPA's Combined Sewer Overflows, Guidance for Long-Term Control Plans, September 1995.

> *"The CSO Control Policy recommends flexibility in allowing a municipality to select controls that are cost-effective and tailored to local conditions. For this reason, the choice between the demonstration approach and presumption approach does not necessarily have to be made before a municipality commences work on its LTCP. In some cases, it might be prudent for a municipality to assess alternatives under both approaches. In addition, if a municipality has CSOs that occur to two different water bodies, a control plan that includes the demonstration approach for one receiving water and the presumption approach for the other may be appropriate."*

It is anticipated that each member of the Group will provide additional information in their individual reports.

4 Development of Alternatives

A multi-tiered approach was applied to the development of alternatives. The intent is to give adequate attention to the breadth of alternatives available but to limit the list of alternatives evaluated to a reasonable amount. This is consistent with Chapter 3 of EPA's Guidance for Long Term Control Plans which states:

> *"Ultimately, the NPDES permitting authority should be satisfied that the municipality has studied all reasonable options in developing a list of final alternatives for evaluation and that the evaluation process incorporates all identified concerns"*
>
> *"As noted previously, in the initial alternative development steps, the number of alternatives necessary to cover the range of control levels for each CSO can be very large. Judgment is necessary to develop a manageable array of alternatives. It is important to remember that the iterative screening of alternatives is flexible and not a rigid process."*
>
> *"Given the number of specific control measures within each of these categories and the range of sizing options for specific measures, initially it might be practical to consider general categories, such as storage or treatment, rather than specific storage or treatment technologies."*
>
> *"Alternatively, it might be appropriate to identify "representative" technologies, with the understanding that specific technologies would be considered as part of more detailed evaluations."*
>
> *"Once a range of CSO control goals has been developed and approaches to structuring CSO control alternatives have been identified, the next step is to develop specific alternatives to achieve the various CSO control goals."*

Technologies are not mutually exclusive. Different technologies can be applied in different locations or combined at the same location.

While the BCUA owns and operates the trunk / intercepting sewer systems (trunk sewers) that transport flows to the WPCF it does not own or operate any of local collector sewers, which are owned and operated by each individual municipality. The regional evaluation by the BCUA CSO Group considered general control approaches primarily consisting of the seven alternatives listed as required in Part IV G 4.e.

i. Green infrastructure [To be evaluated by Municipal Permittee]
ii. Increased storage capacity in the collection system. [To be evaluated by Municipal Permittee and in this report]
iii. STP expansion and/or storage at the plant (an evaluation of the capacity of the unit processes must be conducted at the STP resulting in a determination of whether there is any additional treatment and conveyance capacity within the STP). Based upon this information, the permittee shall determine (modeling may be used) the amount of CSO discharge reduction that would be achieved by utilizing this additional treatment capacity while maintaining compliance with all permit limits. [To be evaluated in this report]
iv. If applicable, the evaluation of dry and wet weather flows that entering the combined sewer system from separately sewered municipalities, and in all cases Inflow/Infiltration (I/I) reduction in the entire collection system that conveys flows to the treatment works to free up

storage capacity or conveyance in the sewer system to inform the evaluation of I/I reduction as an alternative in the LTCP. [Potentially evaluated by Municipal Permittee]

v. Sewer separation. [To be evaluated by Municipal Permittee]

vi. Treatment of the CSO discharge. [To be evaluated by Municipal Permittee]

vii. CSO related bypass of the secondary treatment at the STP portion of the STP in accordance with N.J.A.C. 7:14A-11.12 Appendix C, II C.7 [To be evaluated in this report]

These alternatives were applied to the individual permittee's CSO facilities and water quality goals to evaluate if an alternative can be implemented within the facilities under the permittee's control and significantly contribute to meeting the goals. If an alternative met these requirements, it was carried forward to the in the municipal permittees' evaluation.

4.1 Development and Screening of General CSO Controls

CSO universe of control technologies have been developed over many years and has grown extensively as efforts to address CSOs have progressed. Some controls technologies would be applicable to permittees with CSO outfalls. Other control technologies would only be applicable to the regional authority providing transport and treatment by a regional authority that owns the WPCF, but does not own or operate any CSO Outfalls. Within the BCUA CSO Group, general control approaches dealing with CSO Outfalls (items i, ii, iv, v, and vi above) will be evaluated individually by Fort Lee, Hackensack, and Ridgefield Park and reported upon in their individual reports. Each municipality is responsible for selecting their own criteria for performing the screening and for the subsequent analysis. Since the BCUA does not own nor operate any CSO outfalls, the BCUA will screen and address those technologies appropriate for an entity that owns transport and treatment facilities, namely:

- Increased Storage Capacity
- Expansion and/or storage at the WPCF and;
- CSO related bypass of the secondary treatment.

It was noted to the permitee that the alternative controls selected by one municipality could impact the hydraulics in another CSO municipality. Therefore, the evaluation of alternatives should be based on not increasing the hydraulic grade line in the trunk sewer beyond existing baseline flows (Maximum peak flow discharged to the trunk sewer in 2004) unless otherwise approved by the Authority.

4.2 Infiltration and Inflow

While some may argue that excessive infiltration and inflow within the separately sewer municipalities consume hydraulic capacity within both the transport and treatment facilities owned and operated by the BCUA, the BCUA does not have the legal authority to require any form of I/I control within individual municipalities. Accordingly, while the BCUA has made efforts in the past to work with individual municipalities to locate and eliminate sources of infiltration and inflow, they have no legal means of enforcement. Nevertheless, must if not all of the municipalities within the district have initiated house inspections prior to the issuance of certificates of occupancy to locate and remove sump pumps that discharge to the sanitary sewer system at the time individual units are sold. The location and elimination of these sources of inflow should provide some relief from this source of inflow within separate sewered communities over the long term.

4.3 Trunk Sewer System Optimization

Sewer system optimization involves transport system controls and modifications that affect wastewater flows and loads once the wet weather flows have entered the trunk sewer or transport system. Options for system optimization include measures that maximize the volume of flow stored in the transport system and thus maximizes the capacity of the system to convey flow to the treatment plant. Sewer system optimization techniques have no direct impact on water quality, but do have the potential to reduce the volume of CSO events by allowing additional flows to be processed at the WPCF.

4.3.1 Increased Storage Capacity in the Transport System

Options for increased storage capacity rely on maximizing the volume of flow stored in the transport system or increasing the transport capacity of the system. Maximizing the use of the existing system involves ongoing maintenance and inspection of the collection system and can include minor modifications/repairs to existing structures to increase the volume of flow retained in the system. The BCUA maintains its trunk sewers. Recent monitoring of the Main and Overpeck Creek Trunk Sewers did not indicate any significant sedimentation within the trunk sewer systems servicing the combined sewered municipalities that would reduce transport capacity. Accordingly, changes in operation and maintenance of the trunk sewer system are not being considered as an alternative control by the BCUA.

4.3.2 Additional Conveyance

Conveyance is a technology that transports the combined sewage from an area to a location where the flow can then be stored, or treated and discharged. Conveyance is accomplished by providing additional conveyance pipes or upsizing the existing conveyance pipe to a greater capacity. This practice can effectively reduce overflow volume and frequency in the affected areas. Large conveyance projects can be expensive and may require a lengthy permitting process.

The BCUA finished construction of the Overpeck Creek Relief Sewer in 2010 to increase conveyance capacity in the Overpeck Creek drainage area. The InfoWorksICM model indicates that the existing BCUA Main and Overpeck Trunk/Relief Sewers servicing the combined sewer municipalities have adequate capacity for wet weather flows and that the total available transport capacity exceeds the current design flow for the WPCF by approximately 100 MGD, and the current NJPDES permitted average daily flow by over 110 MGD. Accordingly, additional conveyance capacity is not being considered as an alternative control by the BCUA.

4.3.2.1 Regulator Modifications

A CSO regulator can be uniquely configured to control combined sewer overflow frequency and volume. The existing overflow control structures may be modified based on site-specific conditions. For example, regulator modifications may include increasing the overflow weir height and length or raising the overflow pipe elevation. This technology is especially effective for CSO outfalls with high overflow frequency and low overflow volume, because the additional volume held back in the system is small and less likely to have negative impacts on upstream conditions.

The BCUA owns and operates three CSO regulators within the Village of Ridgefield Park. A review of the flow characteristics at each of these regulators indicates that the regulator size and weir elevations

already maximize flows to the Ridgefield Park Branch Intercepting Sewer and the WPCF. Additional regulator modifications are not being considered as an alternative control for either the BCUA or Ridgefield Park.

4.3.2.2 Real Time Control

Real Time Control (RTC) is a highly automated system in which sewer level and flow data are measured at key points in the sewer system and used to operate systems controls to maximize the storage capacity of the CSS and limit overflows. The collected data is typically transferred to a control device where program logic is used to operate gates, pump stations, inflatable dams and other control components. Local dynamic controls are used to control regulators to prevent flooding and system wide dynamic controls are used to implement control objectives, such as maximizing flow to the treatment plant or transferring flows from one portion of the CSS to another to fully utilize the system. Predictive control, which incorporates use of weather forecast data, is a feature that is used to prevent surcharging of the system, but it is complex and requires sophisticated operational capabilities. Real Time Controls are typically used within the combined sewer municipality to provide inline storage and are best utilized with rainfall monitoring and predictive software. While the Main and Overpeck Trunk Sewers servicing the combined sewer municipalities may have available storage capacity during certain storms, the BCUA services an extensive area where precise rainfall impacts cannot be adequately predicted. Accordingly, real time control of the BCUA Trunk Sewer System is not being considered as an alternative by the BCUA.

4.4 Sewage Treatment Plant Expansion or Storage

The BCUA NJPDES Permit requires that it evaluate the potential for accepting additional wet weather flows from the combined sewer communities to reduce the frequency and volume of CSO discharges. The following sections outline these alternatives.

It should be noted that the BCUA has indicated to the combined sewer communities that the cost of improvements to the WPCF to provide additional wet weather treatment capacity would need to be borne by the communities that benefit.

4.4.1 Additional Treatment Capacity

The InfoWorks computer model indicates that, under present conditions, the Main and Overpeck Creek Trunk/Relief Sewers adequate capacity for existing wet weather flows that occur during the typical year. While the BCUA does not own nor operate any CSO outfalls, the volume of CSOs discharge within the district can be potentially reduced by increasing the treatment capacity of plant and allowing CSO Communities to transfer more wet weather flows to the WPCF for treatment. The BCUA will further evaluate this alternative control as part of their alternative evaluation and screening process as discussed in Section 7.

4.4.2 Wet Weather Blending

Blending is the practice of allowing portions of the wet weather peak flow to bypass certain treatment facilities at the plant. In blending, wet weather flows are typically routed through primary treatment, allowed to bypass secondary and tertiary treatment, and then recombined with effluent from all processes prior to disinfection and discharge to the environment. This practice may require increasing the capacity of primary treatment and disinfection facilities, but does not require the upsizing of secondary treatment

facilities, which can be the costlier components. CSO communities can then make use of the increased wet weather peak flow capacity by providing more flow to the plant instead of CSO outfalls. The BCUA will further evaluate this alternative control as part of their alternative evaluation and screening process as discussed in Section 7.

4.4.3 Storage

The objective of storage is to reduce overflows by capturing and storing wet weather flows, greater than CSS transport/treatment plant capacity, for controlled release back into the system, once treatment and conveyance capacity have been restored. Storage facilities are typically located within the combined sewer system to attenuate peak flows in the CSS and can provide a relatively constant flow into the treatment plant from combined sewer communities after peak events. Assuming the flows can reach the WPCF, storage facilities can also be used at the WPCF to equalize/reduce peak flows and to keep the sewage treatment plant operating at or near its peak unit treatment capacity for extended periods of time. Storage technologies do not prevent water from entering the CSS or treat bacterial loads in CSO discharge, but are very effective at reducing or eliminating CSO events if used within the combined sewer system or if additional wet weather flows can be transported to the WPCF. Storage technologies typically have high construction and O&M costs compared to other CSO control technologies, but can also be a very reliable means of achieving CSO control goals.

Storage has been considered by each of the member municipalities for use within their own combined sewer system. Storage for the BCUA will only be considered as needed to maximize the amount of wastewater that can be treated at the WPCF.

While storage can be developed in different forms, point storage is the one that would be most effective at the BCUA WPCF. Point storage can be provided by above-ground or underground storage facilities such as tanks and equalization basins. These off-line facilities are placed at specific points in the system to detain peak flows for a controlled return of flows back to the system and to keep the WPCF operating at or near its peak treatment capacity for extended periods.

This technology reduces overflow quantity and frequency by storing all or a portion of diverted wet weather combined flows in off-line storage tanks. Stored flows are then returned to the WPCF for treatment once system capacity becomes available. Storage tanks are generally fed by gravity and the stored flow is typically pumped back to the interceptor after the storm. The ability to pump back however relies on transport and treatment capacity being available. The benefit of off-line storage tanks is that they are well suited for early action projects at critical locations.

A disadvantage of off-line storage tanks is that they typically require large land area for installation. Off-line storage tanks typically have higher costs per volume captured compared to other technologies. Additionally, if the existing sewers are deep such as those near the BCUA WPCF, then the storage tank must also be deep, or the flows must be pumped into the tanks, either of which results in additional construction costs. Operation and maintenance costs can also be high. Depending on the application, odor problems may also be an issue. However, storage tanks may be a very effective means of equalizing flow at the WPCF and will be considered further.

4.5 SIU Controls

The NJPDES CSO Permit requires that impacts from significant indirect users (SIUs) contributing to the CSOs are minimized. Under the current rules and regulations, each SIU is required to incorporate a level of pretreatment prior to discharge to the sewer system based on the loading and toxicity of the SIU

contributions. BCUA monitors SIUs for compliance with the pretreatment requirements. There are two SIUs within the combined sewer area serviced by BCUA, summarized in Table 4-1.

Table 4-1: SIU Summary

SIU Name and Address	CSO Basin	Contributing Flow Rate	Description
Custom Silicon Wafer 80 Railroad Avenue Ridgefield Park	RP-006A	250 gpd	Manufacture of SEMI Prime Silicon Wafers
General Aviation and Electronics 30 Jersey Place Hackensack	H-002A (Court Street)	645 gpd	Sheet Metal Fabrication

The discharge from these SIUs were analyzed to assess whether, during overflow events, the discharge would negatively affect water quality, focusing on toxic metals and organics. Their discharges are sampled under existing industrial pretreatment permits issued by BCUA. It is noted that the permits require only certain parameters to be measured. Sampling reports were provided by BCUA. The flow characteristics of this outfall are provided in Table 4-2 and Table 4-3.

Table 4-2: Overflow Data for Ridgefield Park CSO-006A; Custom Silicon Wafers

CSO Basin 006, Ridgefield Park	Overflow statistics for typical year, 2015 Baseline
Number of overflows	12
Annual volume (MG)	0.5
Annual duration (hrs.)	39
Average flow rate (MGD)	0.31

Note: Data taken from Ridgefield Park Combined Sewer System Characterization Report.

Table 4-3: Overflow Data for Hackensack CSO-002A; General Aviation and Electronics

CSO Basin 002A Court Street Hackensack	Overflow statistics for typical year, 2015 Baseline
Number of overflows	76
Annual volume (MG)	151.5
Annual duration (hrs.)	456 (76 overflow days, assumed 6 hrs. per day)
Average flow rate (MGD)	7.97

Note: Data taken from Hackensack Combined Sewer System Characterization Report.

Based on the concentration and the discharge flow rate, the annual mass load was calculated for each measured contaminant over the annual duration of overflow events for the typical year, existing conditions as reported in the System Characterization Reports. The number of days with overflows (rather than the duration) was reported for Hackensack. To provide a conservative analysis, the overflow was assumed to last 6 hours for each day on which an overflow occurred. The analysis conservatively assumed 100% of the SIU load during the overflow (i. e., no portion of the contaminants was conveyed to the treatment plant), which is also a worst-case assumption.

To estimate the average concentration of each contaminant in the overflow attributable to SIUs, the mass load was divided by the annual volume of overflow. Because the objective was to assess the effect of the SIUs, concentrations of the contaminants of interest in the combined sewer flow without SIUs were not considered. The results are presented in Table 4-4. All concentrations are very low, less than 10^{-4} (0.0001) mg/L or 0.1 ppb. This is attributable to dilution, as the average flow rate of the CSO is about 1,900 times larger than the flow rate from Custom Silicon Wafers and about 1,000,000 times larger for General Aviation and Electronics.

To assess whether the estimated concentrations were problematic, they have been compared with the EPA's aquatic life criteria (*National Recommended Water Quality Criteria – Aquatic Life Criteria Table, USEPA, undated*), using the lower, i.e. more restrictive, of the values for salt and fresh water. It was found that none of the estimated concentrations exceeded the criteria when criteria were available. It is noted that some criteria were not indicated by the EPA for the organic compounds and for some metals.

Given that the concentrations are low and do not exceed EPA criteria, it was determined that special measures to prevent or limit discharges from SIUs during wet weather are not warranted.

Table 4-4:Comparison of SIU Overflow Contribution to EPA Criteria

	Custom Silicon Wafer			General Aviation			
	SIU Average Discharge (mg/L)	SIU Load (lb./yr.)	CSO Average Discharge (mg/L)	SIU Average Discharge (mg/L)	SIU Load (lb./yr.)	CSO Average Discharge (mg/L)	EPA Aquatic Life Chronic Criteria, mg/L
METALS							
Arsenic	0.000	7E-07	4E-07	0.000	1E-05	1E-08	0.036
Cadmium	0.005	8E-06	4E-06	0.001	5E-05	4E-08	0.001
Chromium	0.022	4E-05	2E-05	0.095	8E-03	6E-06	0.011
Copper	0.031	5E-05	2E-05	0.099	8E-03	6E-06	0.003
Nickel				0.015	1E-03	1E-06	0.008
Lead	0.030	5E-05	2E-05	0.000	3E-05	2E-08	0.003
Zinc	0.163	3E-04	1E-04	0.132	1E-02	8E-06	0.081
Silver	0.001	1E-06	6E-07	0.001	7E-05	5E-08	0.002
Mercury	0.000	4E-08	2E-08				0.001
NONMETALS							
Bis(2-ethylhexyl) phthalate	0.003	5E-06	2E-06	0.025	2E-03	2E-06	
Bromodichlorometha ne				0.004	3E-04	3E-07	
Dibromochlorometha ne				0.002	2E-04	1E-07	
Methylene chloride	0.001	1E-06	5E-07	0.001	4E-05	3E-08	
Chloromethane	0.004	6E-06	3E-06	0.003	2E-04	2E-07	
Bromomethane	0.000	7E-07	3E-07				
Benzene	0.001	9E-07	4E-07				
Chloroform				0.010	8E-04	6E-07	
Dimethyl phthalate				0.005	4E-04	3E-07	
Acrolein				0.003	2E-04	2E-07	
Bromoform				0.000	2E-05	2E-08	
Toluene				0.000	1E-05	1E-08	
4-Chloro-3-Methyl Phenol				0.002	2E-04	2E-07	

4.6 Summary of Alternatives Screening

The three approaches that warrant further evaluation by the BCUA are expansion of treatment, or storage at the BCUA WPCF, and the CSO related bypass of the secondary treatment facilities and blending of the effluent prior to discharge to the Hackensack River. Each of the alternative is discussed in greater detail in Section 7.

5 Costing

The implementation of the regional CSO LTCP is anticipated to be implemented over an estimated thirty-year period and will represent a massive undertaking for the regional CSO municipalities. While each of the permittee have been issued an individual NJPDES Permit the timing for development and the start of implementation of the CSO LTCP is generally the same for all permittee. To address the issue and to prevent redundancy of effort, the Passaic Valley Sewerage Commission started the NJ CSO Group and invited individual permittee to join in developing several of the tasks and report requirements under the permit.

In 2003, the NJDEP reissued a General NJPDES Permit for authorities and municipalities that owned and/or operated facilities that either controlled or transported/treated wet weather flows from combined sewer systems. Under the General Permit, the permittees that owned or operated CSO outfalls were required to develop a technical guidance manual (TGM) that could be used to evaluate the effectiveness and project costs associated with various alternative control methods for reduction or treatment of CSO discharges. The TGM was developed by Mott MacDonald for use by members of the NJ CSO Group in developing the CSO LTCP Reports submitted in April 2007.

The TGM as developed by the NJ CSO Group in 2005 still provides a great deal of valuable information that could be used by the Group in the present undertaking. PVSC retained Greeley and Hansen/CDM Smith in 2017 to update the TGM in terms of alternative controls that should be considered and to update construction and operation and maintenance cost factors that need to be considered. For consistency between individual reports, PVSC has made the updated TGM available for use by members of the BCUA CSO Group. The entire PVSC Combined Sewer Overflows Update Technical Guidance Manual dated January 2018 is being included in Section 11 of this Report. While this information is being used by all members of the BCUA CSO Group, the TGM is only being included in the BCUA report to avoid redundancy.

To be consistent with other permittees, PVSC guidance from the January 8th memorandum and meeting on March 21, 2019 was used to develop present worth costs for all the alternatives, to combine O&M and full capital costs for each control technology. A discount rate of 2.75% was used (Rate of Federal Water Projects, NRCS Economics, Department of the Interior) with a life span of 20 years. The following equation was then utilized to calculate the present worth factor to convert from annual O&M costs to present worth.

$$(P/A, i\%, n) = ((1+i)^n-1)/((i(1+i)^n)$$

The above was multiplied by the annual O&M costs and added to the construction costs to obtain the total life cycle cost. For the given life cycle and interest rate the P/A factor is 15.227. Salvage value was assumed to be $0, as it is assumed no resale value will result from the Control Technologies utilized.

6 Available Land Analysis

6.1 Background

As previously noted, each member of the BCUA CSO Group will be undertaking the development and evaluation of alternative controls applicable to their overall system. The Borough of Fort Lee, City of Hackensack, and Village of Ridgefield Park have NJPDES Permits to cover discharges from their combined sewer systems to area receiving waters. As part of the development and evaluation of alternative controls, each permittee needs to undertake an evaluation of potential parcels/sites where alternative control facilities might be installed to reduce the frequency and volume of CSOs. Each municipality will report on their available land use within their individual reports. While the BCUA does not own or operate any CSO outfalls, they have a responsibility under the permit to evaluate potential changes/improvements to its Little Ferry WPCF. This section summarizes the steps that were taken to evaluate land availability within the existing BCUA property to evaluate where space may be available to implement potential improvements to the WPCF. Figure 6-1 illustrates the transport and treatment facilities owned and operated by the BCUA in the region of the Little Ferry WPCF.

Figure 6-1: BCUA Little Ferry WPCF

While the CSO communities have some flexibility as to where control alternatives might be constructed within their municipalities, including open or underutilized spaces not necessarily immediately adjacent to the CSO outfalls, the BCUA does not have that flexibility. Alternative controls being evaluated by the BCUA need to be integrated into the existing wastewater facility and accordingly need to be constructed near, and in conjunction with existing units. Accordingly, only existing open space associated with the existing BCUA property can be considered.

6.2 Data Compilation and Processing

The BCUA property was analyzed using aerial photography to evaluate existing land use / land cover, and the general relationship between the transport and treatment facilities. As illustrated in Figure 6-2 BCUA Trunk Sewers enter the BCUA from the Northwest (96” Main Trunk), the Southwest (48” Southwest Trunk) and the east (Overpeck 60” Trunk and 72” Relief Sewers). The head end of the WPCF is located at the point where the 96” Main and 60” Overpeck Creek Trunks converge. As illustrated below, most of the

BCUA Property is surrounded by water; the Hackensack River to the east and the former clay pit retention basin to the west.

Figure 6-2: BCUA WPCF Transport and Treatment Facilities

6.3 Evaluation

As illustrated in Figure 6-3 the BCUA does have relatively open space on the north and south sides of the site that do not contain any buildings or other surface structures. The difficulty with utilizing any of the property on the north side for alternative control consideration is that it is removed from the headwords of the WPCF (circle) and the general direction of wastewater flow through the treatment process, which is to the southwest. In addition to the two large parcels, there are smaller parcels of open space which are also available between unit processes. As previously noted, the proximity of the open space (control alternative) to the other unit processes is important. Accordingly, it is anticipated that the open space available in between units and the open space located to the south will be those most suitable for construction and installation of piping or processes needed to expand capacity of the WPCF. Preliminary site locations for alternative controls will be illustrated in greater detail in Section 8 of the report.

Figure 6-3: BCUA WPCF Open Space

7 Alternatives Evaluation

7.1 Introduction

As previously noted, the BCUA does not own nor operate any CSO discharges. Accordingly, its approach to the LTCP is different, in that the Authority will not be choosing between the Presumptive or Demonstrative approach. It will coordinate and review with its member municipalities as to the options available, and work with them to develop a Regional LTCP. It is anticipated that each member will choose the approach that best suits their system.

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7.2 Presumptive and Demonstrative Approaches

While preliminary data from the Baseline Compliance Monitoring Program and Model Development undertaken by the NJ CSO Group (including members of the BCUA CSO Group) is available, it is not finished. Preliminary data indicates that some regional water bodies are currently meeting existing water quality standards, while others are not. Non-compliance with water quality standards are not, however, restricted to combined sewer communities or their area of impact within tidal regions. While preliminary investigations using the predictive mode of the model have been undertaken, additional information is needed to bring confidence to the results. Accordingly, investigations on the relative impact of partial or complete elimination of all CSO discharges within the Gateway Region (Passaic River, Hackensack River, Newark Bay, and Overpeck Creek) will not be completed within the framework of this report. The NJPDES permit requires the selection of either the Presumptive or Demonstrative Approach to the LTCP with the selection reasoning detailed within this report, the members of the BCUA CSO Group are requesting that that decision, as to which approach will be taken, be delayed until submission of the final LTCP. This will afford the NJ CSO Group opportunity to undertake additional refinement of its river model data and scenarios prior to making that decision. It is anticipated that each member of the Group will provide additional information in their individual reports.

7.2.1 Evaluation Criteria

7.2.1.1 Siting Issues

Siting of facilities intended to increase the capacity of the BCUA WPCF to accept and treat addition flows from combine sewer communities will be limited to the available sites identified in Section 6. Due to limited land available and the nature of the facilities – large industrial looking facilities with potential odor issues – they will be limited to the area within and adjacent to the existing plant.

7.2.1.2 Institutional Issues

Institutional issues refer to permitting requirements, likelihood of receiving permits, and timeline to receive permits, regulatory compliance in terms of water quality improvements, and ownership of the site (public vs. private). Regulatory considerations such as Green Acres, flood hazard area, wetlands, and threatened or endangered species are also evaluated, as well as zoning/planned development of the site by the municipality.

Permitting is a major institutional issue and is typically a major factor in a project's design schedule. The following is a list of anticipated major permits applicable to the alternatives being analyzed:

- Waterfront Development Permit – Construction will take place within the waterfront development area, which extends from inland from the mean high water (MHW) line a minimum of 100 feet and a maximum of 500 feet, with the development area being truncated at the first paved public road or surveyable property line beyond 100 feet from MHW. The portion of the project within the Waterfront Development Area would also need to comply with the applicable Flood Hazard Area requirements. Restrictions are much more stringent for in water work, including the Flood Hazard Area prohibitions regarding placement of fill in the floodway. Waterfront Development Permits are typically issued within 90-days from receipt of an approvable application (construction permits).
- Flood Hazard Area Permit – A flood hazard area permit will be required for work within the floodplain outside of the Waterfront Development Area. Since the floodplain at the BCUA WPCF is tidal, much of the work will be eligible for permit-by-rule; however certain facilities may require individual permits. There may be impacts to riparian zone vegetation along the existing watercourses. Flood Hazard Area Permits are 90-day construction permits, however, there are mechanisms which could delay the issuing of a permit beyond 90-days.
- Soil Conservation District (SCD) Certification – The New Jersey Department of Agriculture implements the New Jersey Soil Erosion and Sediment Control Act, New Jersey Statutes Annotated (N.J.S.A. 4:24-39). The program is administered by the local soil conservation district, in this case the Bergen County Soil Conservation District. The program requires soil erosion and sediment control measures be in place during land disturbance activities and requires certification of a soil erosion and sediment control plan for all disturbances greater than 5,000 sf. It also addresses the post construction and long-term site stabilization. Certification from the soil conservation district is a 90-day construction permit. If the project exceeds one acre of disturbance then authorization to discharge stormwater during construction is required from the State, generally under a NJPDES Master General Permit, Program Interest Group 5G3, which is an online approval based on the SCD certification.
- Treatment Works Approval – Treatment Works Approval is required for modifications to the sanitary and combined sewer systems, and to the treatment process at the plant. There are regulatory thresholds for when a treatment works approval is required; however the activities associated with a LTCP would easily exceed those thresholds. Treatment Works Approval Permits are 90-day construction permits.
- Stormwater Management – While not specifically a permit, the State claims jurisdiction over major developments for projects that require Land Use permits. The Stormwater Rules (N.J.A.C. 7:8) primarily concern themselves with stormwater quantity, quality and recharge. The BCUA WPCF is also located in the Hackensack Meadowlands State Planning Area and would be subject to the stormwater management rules associated with the planning area, which are similar to the State requirements. Since BCUA is in a tidal flood hazard area, quantity of discharge is not expected to be an issue. Recharge of groundwater may be a challenge due to shallow groundwater, but depending on the existing soils the site may be exempt. The quality of discharge will need to be addressed for projects that create more than on quarter acre of new impervious area. The current rules require 50% TSS removal from redeveloped impervious surfaces and 80% removal from new surfaces. Proposed changes to the Stormwater Rules formalize the Department's policy surrounding implement green stormwater infrastructure may impact the requirements. The proposed changes are not in effect yet but may be finalized prior to the LTCP selection.
- Army Corps of Engineer Nationwide 404 Permit – The Army Corps of Engineers (USACE) regulates tidal waterways within New Jersey. The USACE does not regulate upland areas, as such, only disturbances below the MHW line would be regulated by USACE. Other agencies such as United States Fish and Wildlife Service (USFWS), National Oceanic and Atmospheric Administration (NOAA) and United States Coast Guard (USCG) may concurrently review the permit application. A more

detailed impact analysis such as an Essential Fish Habitat Assessment may be required as part of the USACE submission. USACE permits do not have a set review timeframe.

- Wetlands Permits – Any wetland habitats identified landward of the MHW would be regulated as freshwater wetlands. A wetland delineation and investigation would be accomplished based on the 1989 Federal Manual for Identifying and Delineating Jurisdictional Wetlands, which is the recognized wetland delineation manual for the State of New Jersey. Any proposed impacts to identified freshwater wetlands or transition areas would be subject to the rules applied under N.J.A.C. 7:7A. Freshwater Wetland Permits do not have a set review timeframe; however, if submitted concurrently with a Waterfront Development Permit and/or Flood Hazard Area Permit may be issued within 90-days.
- Tidelands – The State lays claim to all lands now or formerly flowed by the tide, where the land is held in public trust. Projects making use of the land must either obtain a tidelands license (lease) or be granted (purchase) the riparian rights. All such grants and licenses must be approved by the Tideland committee in a process that takes several months, and in case of granting riparian rights, the appraised market value must be paid to the State. The State has tidelands claims along the Hackensack River, it is not known if any of these areas have been granted in the past. This will need to be investigated following selection of the final implementation plan.

Figure 7-1: NJDEP Geo-Web showing Tidelands claims (light blue) near the BCUA WPCF

- Local Permits – Depending on the nature of the project there are several local permits that may be required. These may include zoning permits, construction permits, land use board approval, road opening permits, etc. Since the potential sites are adjacent to the plant and consistent with the area's industrial use, local permits are not considered a major obstacle, except at the north edge of the plant which is a residential area.
- Green Acres – There are park areas in the vicinity of, but not adjacent to the plant that are in the Green Acres program. Any impacts to Green Acres property would be considered a diversion and mitigation would be required. The mitigation (diversion of use) process is long and expensive and should be avoided where possible. There are also parcels owned by the Meadowlands Conservation Trust, which likely has restrictions against development.

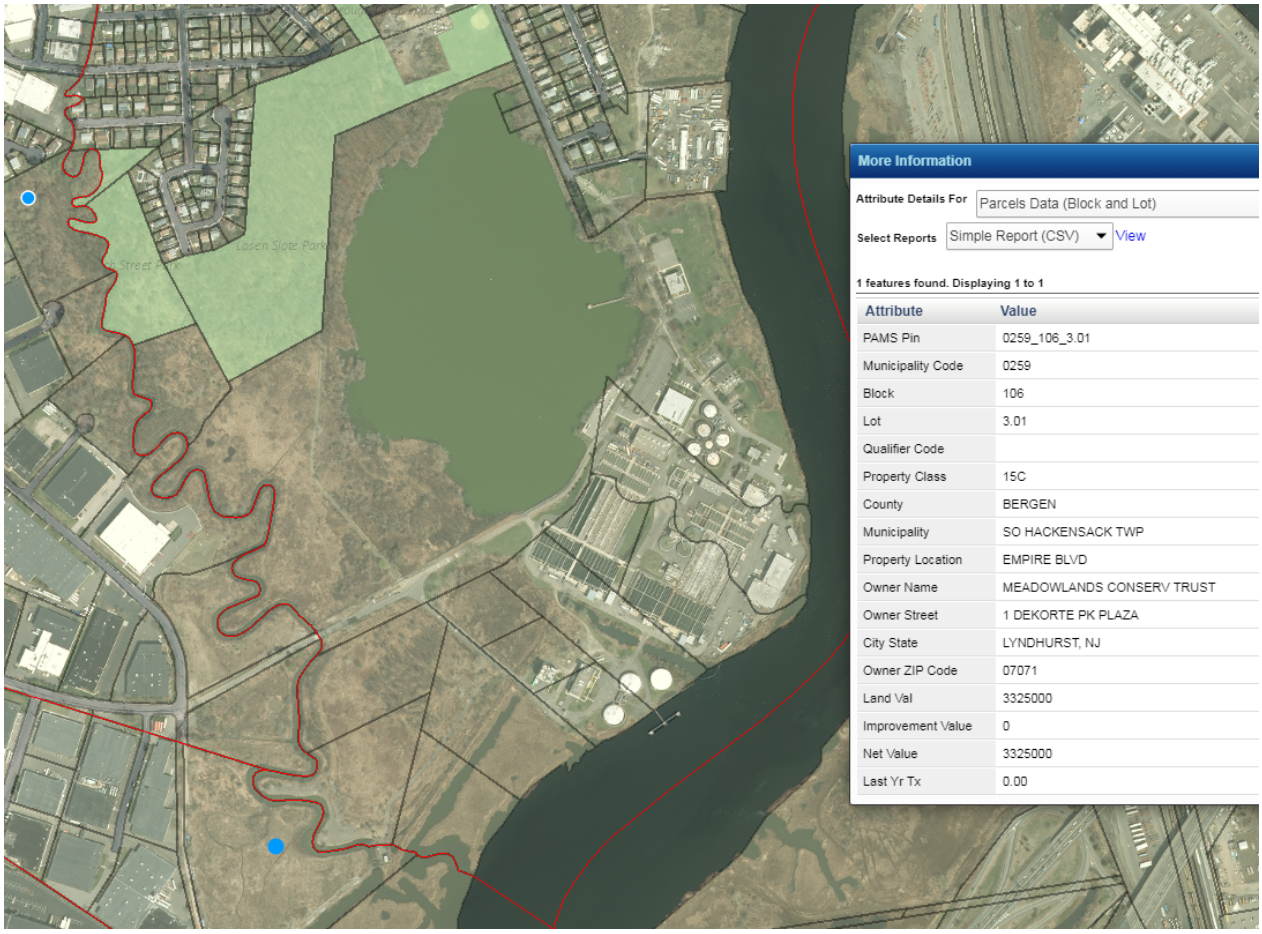


Figure 7-2: NJ GeoWeb open space layer.

- County and State Highway Permits – Since the facilities are being restricted to the area around the plant, no such permits are envisioned.
- Railroad Occupancy – There are no rail line through the BCUA WPCF, so no such permits are envisioned.

7.2.1.3 Implementability

Implementability refers to considerations that could present challenges or prevent the construction of an alternative. This includes such factors as:

- Site access – If space is available, but it cannot be efficiently accessed, the cost to construct and maintain LTCP facilities could be prohibitive. This could be a consequence of geography or existing infrastructure.
- Ownership and ease of acquisition or easement – Ultimately, the BCUA is responsible for the operation and maintenance for LTCP facilities, however, BCUA will not be financially responsible for the costs associated with constructing or maintaining these facilities, that will be the obligation of the municipal CSO permittees that benefit from them. Therefore, they must be able to acquire (purchase) the

property the facilities are sited on, or obtain permanent easements that will allow for maintenance, as well as potential future upgrades.

- Land area available – CSO control facilities are large and often do not lend themselves to be distributed to sites remote from the WPCF or interceptors that carry combined flows. While some challenges associated with land area can be overcome through diversion piping, doing so may greatly increase the overall project cost.
- Environmental considerations – In addition to the permits required as discussed under institutional issues other factors such as soil type are relevant to some of the alternatives for infiltration/dewatering, foundation support, as well as for tunneling construction/excavation.
- Compatibility with existing infrastructure – This considers any existing structures or utilities that would need to be relocated or decommissioned. Relocation of utilities can greatly increase the cost of a specific project and may have a potential impact on the local community, as it often requires shutting down of the utility while it is being relocated.

7.2.1.4 Public Acceptance

Public acceptance refers to the degree to which community residents, businesses and institutions would be impacted or perceive the alternative to be favorable or unfavorable. This includes considerations such as:

- Construction disturbance – Construction brings a variety of unwelcome impacts to a community, such as traffic, dust, noise and vibration. These are unavoidable to some degree, but the facilities construction methods selected can serve to reduce or augment these concerns. For example, an alternative that required pile driving produces much more noise and vibration than traditional excavation, or other potential methods for pile installation. The duration of the construction, and to a certain extent the method of construction can impact costs and should also be a considered.
- Visibility – Residents prefer solutions that are aesthetically pleasing and have an expectation that their community will be left looking as nice or better than it did prior to the project. There may also be concerns that the visual impact may reduce property values.
- Impact to community spaces – Public areas such as parks are amenities; and if their functionality is diminished the public will object.
- Community character – Communities are generally built around common land uses. For example, industrial areas are generally separate from residential areas. Accordingly, opposition could be expected if an industrial looking CSO facility was sited in a residential area. Likewise, facilities that could be perceived as not good for business may not be accepted in a commercial area. Facilities that potentially produce noise or odor are also expected to create opposition.
- Traffic impacts – Traffic impacts may occur during construction and after construction. During construction, consideration must be given to the location and length of time of the impacts. Acceptance to impacts that may persist after construction would depend on the severity of the impact, both in terms of the number of residents impacted and magnitude of inconvenience.
- Cultural resources – Sites of historic significance should be avoided. It is also possible that the historic significance of sites may be highly localized and not detected until the plan is well advanced.
- Environmental justice – In general the project must avoid in fact and perception that the impacts are skewed towards those with lower socio-economic standing.
- Community resources – Projects that impact community resources are likely to receive higher levels of opposition. Community resources may include, schools, houses of worship, emergency services and community centers. This may include projects that directly impact community resources such as taking

part of the property or indirectly by impeding access or function such as key routes to hospitals or for emergency services.

Public acceptance can take many forms. In some areas, residents and business may not be concerned and accept the construction; however, it is also possible for stronger levels of community opposition to occur. Opposition groups can be extremely vocal, active and well-funded. There is also the possibility that opposition groups can influence local elections in favor of those that oppose the CSO LTCP or mount legal challenges. While public outreach such as the CSO Supplemental Team and public meetings can mitigate these challenges, it cannot altogether eliminate them as risks to the project.

8 Development and Evaluation of Alternatives

As previously noted, each member of the BCUA CSO Group will be undertaking their own evaluations on which control alternatives best suit their ability to comply with their NJPDES Permit. Each alternative may correlate directly to a technology that passed through the screening or be built of several technologies with the overall alternative selection based on the CSO control goals. This report will restrict itself to the alternative controls that are outlined in the BCUA NJPDES Permit, which are applicable to a WPCF, and cannot be evaluated by the individual CSO municipalities. BCUA CSO Group members will report upon the development and evaluation of their alternative controls within their individual reports.

8.1.1 BCUA Transport Capacity

While the BCUA owns and operates the trunk / intercepting sewer systems (trunk sewers) that transport flows to the WPCF, it does not own or operate any of local collector sewers, which are owned and operated by each individual municipality. This report and analysis are limited to those segments of the BCUA transport system that service the combined sewer municipalities, namely the lower Main Trunk Sewer, the Overpeck Creek Trunk Sewer, and the Overpeck Creek Relief Sewer.

As part of the Regional Sewer System Characterization Report (June 2018) Mott MacDonald developed a comprehensive sewer system model of the entire BCUA District using the InfoWorksICM modeling software, covering the forty-four separate sewered areas, with special emphasis on the combined sewer systems within the Borough of Fort Lee, the City of Hackensack, and the Village of Ridgefield Park. The models for the combined sewer communities were prepared by the municipalities and imported into the overall BCUA model. This model will be used in the development of the CSO LTCP. Information from the model has also been used to generate data and information on the existing transport capacity of the BCUA transport system servicing the combined sewer communities of Fort Lee, Hackensack, and Ridgefield Park.

8.1.1.1 Theoretical Transport Capacity

The InfoWorks Model does calculate theoretical pipe capacities for each reach within the model using Manning's Equation and pipe geometry (size, shape, slope, and surface roughness). A typical sewer system design ties to a constant pipe shape and size to the extent practical unless there is an increase or decrease in flow. When the pipe size and shape are constant the maximum flow capacity of the pipe is a function of the pipe slope. Larger piping systems, such as the trunk sewers, are typically constructed using relatively shallow slope, which makes it difficult to maintain a constant slope through the each. While Manning's Equation is a useful tool in developing an understanding of capacity in the system, this method produces calculated capacities that can vary greatly within the same region due to changes in the measured slope. Accordingly, it may be difficult to determine how these changes in slope, whether positive or negative, impact the overall capacity of the sewer. Manning's Equation only evaluates the impact of geometry upon maximum pipe flow of a single reach. The InfoWorks model overcomes these difficulties by using hydraulic calculations to understand the interaction between sewer reaches. Nevertheless, Manning's Equation can provide us with a general understanding of the maximum flow capacity available within the Trunk Sewers.

Information from the InfoWorks model has been extracted and used to review the overall maximum pipe flow capacity from each major segment of the BCUA Trunk Sewer servicing the combined sewers within the district as follows:

A. BCUA MAIN TRUNK SEWER

The BCUA Main Trunk Sewer, as previously illustrated in Figure 1-3, provides wastewater transport capacity to the combined sewers within the City of Hackensack and the separate sewer communities that are not located in the Overpeck Creek drainage area. The trunk sewer begins as a 96-in diameter sewer just upstream of the headworks of the BCUA WPCF in Little Ferry, within the WPCF limits, the 48" Southwest Interceptor enters the 96" Main Interceptor. It changes to an 84-inch sewer just upstream of the City of Hackensack just before it crosses the river. This is the segment of the Main Trunk Sewer that services the City of Hackensack. Information from the InfoWorks model, including upstream and downstream nodes (manholes) and inverts, reach length, and pipe diameter are provided in Table 8-1. In addition, the full pipe capacity of each reach as calculated by Manning's Equation through the model is provided as follows:

Table 8-1: BCUA Main Trunk Sewer Calculated Peak Flow Capacity

BCUA Main Trunk Sewer

US node ID	DS node ID	US invert level (ft AD)	DS invert level (ft AD)	Length (ft)	Diameter (in)	Slope (ft/ft)	Area of Pipe (ft)	Manning's n	Full Pipe Capacity (MGD)	Section Full Pipe Capacity (MGD)
BCUA_Break_150	WWRF_SCREEN	-24.26	-29	78.5	96			0.013	1448.45	
BCUA_Break_255	BCUA_Break_150	-23.582	-24.26	352.3	96				258.58	
BCUA_72	BCUA_Break_255	-22.67	-23.582	474.2	96				258.58	
BCUA_910	BCUA_72	-22.46	-22.76	700.1	96				122.04	
BCUA_Break_151	BCUA_910	-22.219	-22.46	500	96				129.53	
BCUA_Break_152	BCUA_Break_151	-22.026	-22.219	400	96				129.53	
BCUA_911	BCUA_Break_152	-21.07	-22.026	1979.6	96				129.53	
BCUA_Break_155	BCUA_911	-20.911	-21.07	671.9	96				90.73	
BCUA_Break_195	BCUA_Break_155	-20.87	-20.911	172.6	96				90.73	
BCUA_Break_154	BCUA_Break_195	-20.71	-20.87	356.3	96				125.1	
BCUA_Break_153	BCUA_Break_154	-20.618	-20.71	202.6	96				125.1	
BCUA_912	BCUA_Break_153	-20.24	-20.618	840.2	96				125.1	
BCUA_Break_156	BCUA_912	-20.065	-20.24	477.3	96				113.03	
BCUA_Break_157	BCUA_Break_156	-19.717	-20.065	945.1	96				113.03	
					96	0.00042	50.26	0.013		121
BCUA_917	BCUA_Break_157	-19.35	-19.717	1135	96				106.04	
BCUA_Break_158	BCUA_917	-19.337	-19.35	32.4	96				120.3	
BCUA_Break_159	BCUA_Break_158	-19.138	-19.337	476.8	96				120.3	
BCUA_Break_160	BCUA_Break_159	-18.49	-19.138	1555.6	96				120.3	
BCUA_918	BCUA_Break_160	-17.95	-18.49	1297.4	96				120.3	
BCUA_920	BCUA_919	-17.5	-17.97	1349.8	96				110.01	
BCUA_919	BCUA_918	-17.97	-17.95	131.6	96				-72.67	
BCUA_Break_161	BCUA_920	-17.23	-17.5	357.5	96				162.02	
					96	0.00039	50.26	0.013		117
HACKENSACK_MH_J1	BCUA_Break_161	-16.5	-17.23	2574.5	96				92.18	
HACKENSACK_MH_BCUA9A	HACKENSACK_MH_J1	-16.26	-16.5	2251.8	96				56.52	
HACKENSACK_MH_BCUA9B	HACKENSACK_MH_BCUA9A	-15.81	-16.26	1933.2	96				83.52	
HACKENSACK_MH_BCUANode1	HACKENSACK_MH_BCUA9B	-15.5	-15.81	939.8	96				99.43	
BCUA9C	HACKENSACK_MH_BCUANode1	-15.35	-15.5	825.5	96				73.79	
BCUA_Break_190	BCUA9C	-15.28	-15.35	1268.7	96				43.79	
BCUA_924	BCUA_Break_190	-15.22	-15.28	731.2	96				53.4	
BCUA_Break_180	BCUA_924	-14.98	-15.22	1392.3	96				77.4	
					96	0.00019	50.26	0.013		81
BCUA_925	BCUA_Break_180	-14.3	-14.98	326.5	96				269.06	
BCUA_Break_217	BCUA_925	-14.29	-14.3	123.9	96				52.85	
BCUA_Break_218	BCUA_Break_217	-14.282	-14.29	102.1	96				52.85	
BCUA_Break_219	BCUA_Break_218	-14.27	-14.282	144	96				52.85	
BCUA_Break_181	BCUA_Break_219	-14.184	-14.27	1075.9	96				52.85	
BCUA_Break_220	BCUA_Break_181	-14.105	-14.184	981.6	96				52.85	
BCUA_929	BCUA_Break_220	-14.08	-14.105	310.2	96				52.85	
BCUA_930	BCUA_929	-13.65	-13.97	1021.7	96				73.08	
				3064.2	96	0.00017	50.26	0.013		78
BCUA_930	BCUA_929	-12.65	-12.97	1021.7	84				73.08	
BCUA_Break_262	BCUA_930	-12.4	-12.45	78.4	84				104.3	
BCUA_Break_263	BCUA_Break_262	-12.3	-12.4	91.6	60			0.013	55.62	
BCUA_Facility_91	BCUA_Break_263	-12.3	-12.3	65.1	84				0	
BCUA_931	BCUA_Facility_91	-12.15	-12.3	351.5	84				85.3	
BCUA_Break_221	BCUA_931	-12.008	-12.05	469.3	84			0.013	38.97	
BCUA_Break_226	BCUA_Break_221	-11.69	-12.008	1053	84			0.013	71.78	
BCUA_Break_222	BCUA_Break_226	-11.601	-11.69	305.6	84			0.013	70.52	
BCUA_Break_223	BCUA_Break_222	-11.459	-11.601	486.1	84			0.013	70.52	
BCUA_932	BCUA_Break_223	-11.45	-11.459	31.1	84			0.013	70.52	
BCUA_Break_244	BCUA_932	-11.22	-11.45	566.3	84			0.013	83.22	
BCUA_Break_224	BCUA_Break_244	-11.212	-11.22	28.8	84			0.013	70.4	
BCUA_Break_225	BCUA_Break_224	-11.14	-11.212	246.4	84			0.013	70.4	
BCUA_933	BCUA_Break_225	-10.92	-11.14	811.1	84			0.013	68.01	
				5606	84	0.00037	38.48	0.013		79

As illustrated, the peak flow capacity can change significantly between individual sewer reaches making it difficult to determine an average flow capacity within a region. Accordingly, the information developed from the model was broken down into sewer segments wherein the general slope in the pipe was relatively constant. These were used to calculate the average maximum flow capacity within that segment of sewer. The individual sewer reaches used in the segment calculation have been highlighted in the table.

As previously noted the individual flow calculations can vary significantly. For example, the first segment illustrated shows that individual sewer reaches between Break_910 and Break_156 varied from a low of 90.7 MGD to a high of 129.5 MGD. Using the average slope and total distance between these reaches produces an average maximum flow capacity of 121 MGD within this region. As noted, the maximum flow capacity of this sewer varies from around 80 MGD in the upper reaches just north of the City of Hackensack to around 120 MGD near the WPCF. It should be noted that the model indicates that the last segment of the sewer near the WPCF is typically surcharged due to hydraulic conditions within the influent wet well.

B. Overpeck Valley Trunk and Relief Sewers

The Overpeck Valley Trunk and Relief Sewers, as previously illustrated in Figure 1-2 and Figure 1-4, provide wastewater transport services to those municipalities within the Overpeck Creek drainage area, including the Borough of Fort Lee (all of Fort Lee's combined sewers are actually in the Hudson River drainage area) and the Village of Ridgefield Park. The Overpeck Creek Trunk Sewer had a history of capacity issues during wet weather and the Overpeck Creek Relief Sewer was designed to eliminate these issues. The Overpeck Valley Relief Sewer was completed in 2010.

The existing Overpeck Valley Trunk Sewer (OVTS or Trunk Sewer) consists of 42″, 48″ and 60″ reinforced concrete sewer pipe and extends from Englewood to the Little Ferry WPCF. Sewage flow is added to the sewer along the route at approximately fifteen (15) locations. The Overpeck Valley Relief Sewer (OVRS or Relief Sewer) was constructed parallel to the existing OVTS sewer and is interconnected with the existing system. All sewer connections to the system continue to be connected to the existing Trunk Sewer, exception for Fort Lee sewer connections at Van Nostrand Avenue and Rockwood Place, the Fort Lee sewers were connected directly to the new Relief Sewer. The Trunk and Relief Sewers are interconnected along the route by six (6) Junction Chambers, as partially illustrated in Figure 8-1 and two (2) Interconnections. Junction Chambers were designed to combine the flow in the Trunk and Relief sewers and to redistribute the flow downstream between the new and old sewers, with the goal of maximizing the sewer's joint conveyance.

The Junction Chambers are located where the Relief Sewer must cross the existing Trunk Sewer and are designed to allow flow from the existing Trunk Sewer to flow into the new Relief Sewer in order relieve a high flow condition in the Trunk. The total wastewater flows are shared by both sewers and both systems will operate together. The system is designed to

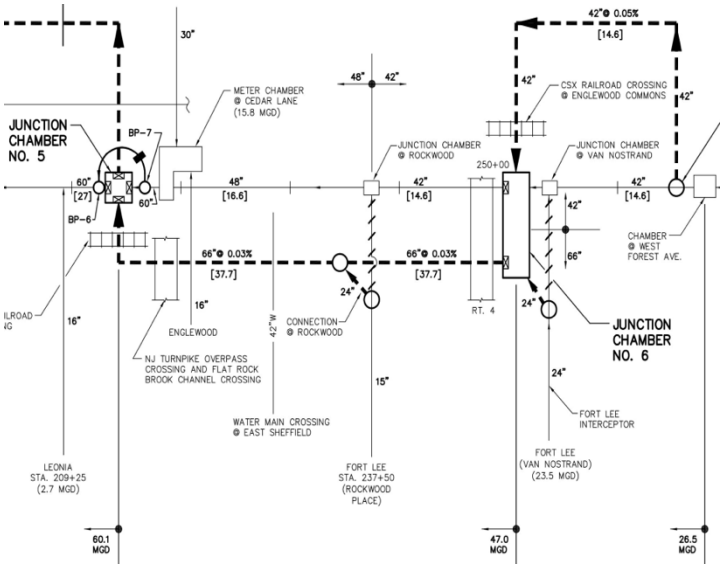


Figure 8-1: OVTS and OVRS Flow Schematic

automatically provide hydraulic relief to the Trunk Sewer by allowing the flow to split between the existing and new systems at each Junction Chamber and to allow the excess flow in the Trunk Sewer system to be relieved at each interconnection. Accordingly, the total capacity of the system is inter-dependent on the capacity of the other two sewers. This fact made modeling difficult, but the modeling data obtained can provide a meaning summary of flow capacity by looking at the sum of flow at each junction chamber. Table 8-2 provides data on the Overpeck Valley Trunk Sewer and Table 8-3 on the Relief Sewer Peak Flow Capacity as calculated for each segment as highlighted.

Table 8-2: Overpeck Valley Trunk Sewer Peak Flow Capacity

BCUA Overpeck Creek Trunk Sewer

US node ID	DS node ID	US invert level (ft AD)	DS invert level (ft AD)	Length (ft)	Diameter (in)	Slope (ft/ft)	Area of Pipe (ft)	Manning's n	Full Pipe Capacity (MGD)	Section Full Pipe Capacity (MGD)
WWRF_SCREEN	SCREEN_1	-29	-29	114.6	96				0	
BCUA_Break_254	SCREEN_1	-28.95	-29.53	89.6	60				135.42	
BCUA_Break_253	BCUA_Break_254	-28.2	-28.95	636.7	60				57.78	
BCUA_1525	BCUA_Break_253	-18.04	-28.2	139.2	60				454.83	
				980.1		0.00183	19.63	0.013		72
BCUA_Break_252	BCUA_1525	-17.52	-18.04	410.9	60				59.89	
BCUA_Break_251	BCUA_Break_252	-16.86	-17.52	578.6	60				56.86	
BCUA_1524	BCUA_Break_251	-16.33	-16.86	444.1	60				58.16	
BCUA_1523	BCUA_1524	-16.22	-16.33	414.6	60				27.42	
BCUA_1522	BCUA_1523	-15.665	-16.22	302	60				72.17	
BCUA_Break_188	BCUA_1522	-15.11	-15.665	343.2	60				67.7	
BCUA_1052	BCUA_Break_188	-14.33	-15.11	678.5	60				57.08	
OP_Chamber_2	BCUA_1052	-14.18	-14.33	473.3	60				29.97	
				3645.2	60	0.00106	19.63	0.013		55
								Weighed Average		58
BCUA_1051	OP_Chamber_2	-14	-14.18	12.3	60				203.38	
METER_1_2	BCUA_1051	-13.25	-13.35	837.1	60				18.4	
BCUA_Break_186	METER_1_2	-12.66	-13.25	579.4	60				53.72	
BCUA_1045	BCUA_Break_184	-12.53	-12.53	97.6	60				0	
BCUA_Break_184	BCUA_Break_186	-12.53	-12.66	764.8	60				21.95	
BCUA_907	OP_Chamber_3	-12.26	-12.26	51.8	60				0	
						0.00082	19.63	0.013		48
OP_Chamber_3	BCUA_1045	-12.26	-12.53	1558	60				22.16	
OP_MH16B	BCUA_907	-11.84	-12.26	940.9	60				35.57	
BCUA_2	OP_MH16B	-10.82	-11.84	803.6	60				59.98	
BCUA_Break_191	BCUA_2	-10.5	-10.82	869.3	60				32.3	
BCUA_1	BCUA_Break_191	-10.3	-10.5	515.7	60				33.15	
BCUA_906	OP_MH4A	-10.07	-10	250.5	60				-28.14	
						0.00050	19.63	0.013		38
OP_MH4A	BCUA_1	-10	-10.3	488.4	60				41.72	
METER_7	BCUA_906	-9.52	-10.07	1692.2	60				30.35	
BCUA_905	METER_7	-9.38	-9.52	1257	60				17.77	
OP_MH34A	BCUA_905	-9.2	-9.38	1041.6	60				22.13	
METER_8	OP_MH34A	-8.75	-9.2	185.4	60				82.93	
METER_9	METER_8	-8.27	-8.75	1713.8	60				28.17	
BCUA_904	METER_9	-7.92	-8.27	1205.6	60				28.68	
BCUA_10	BCUA_904	-7.58	-7.92	1205.4	60				28.27	
JC-5	BCUA_10	-7.47	-7.58	691.6	60				21.23	
						0.00030	19.63	0.013		29
BCUA_1494	JC-5	-7.33	-7.47	351.4	60				33.6	
BCUA_903	BCUA_1494	-6.15	-7.33	591.1	48				41.48	
BCUA_Break_183	BCUA_903	-6.08	-6.15	189.7	48				17.83	
BCUA_902	BCUA_Break_183	-6.06	-6.08	77.6	48				14.91	
BCUA_Break_182	BCUA_902	-5.92	-6.06	465.5	48				16.1	
METER_45	BCUA_Break_182	-5.909	-5.92	37.9	48				15.83	
BCUA_901	METER_45	-5.74	-5.909	574.8	48				15.92	
JC-6	BCUA_901	-5.3	-5.74	601.8	42				17.58	
				2889.8		0.00075	9.62	0.013		18
BCUA_1035	JC-6	-5.3	-5.3	18.5	42				0	
METER_43	BCUA_1035	-4.553	-5.1	675.3	42				18.51	
BCUA_1036	METER_43	-4.51	-4.553	116	42				12.52	
BCUA_1037	BCUA_1036	-4.12	-4.51	791.3	42				14.44	
BCUA_1038	BCUA_1037	-3.79	-4.12	613.6	42				15.08	
OP_MH65	BCUA_1038	-3.78	-3.79	324.4	42				3.61	
BCUA_Facility_141	OP_MH65	-3.78	-3.78	183.4	42				0	
						0.00056	9.62	0.013		15

As illustrated in the above table the maximum transport flow capacity for the Trunk Sewer ranges from around 55 MGD near the WPCF to a low of 15 MGD upstream of junction chamber 6 (JC-6). The model does indicate that the segments of the Trunk Sewer near the BCUA WPCF are surcharged due to the influent pumping station and accordingly these segments do not operate under the free flow conditions as required for using Manning's Equation to determine pipe-full capacity.

Table 8-3: Overpeck Valley Relief Sewer Peak Flow Capacity

BCUA Overpeck Relief Trunk Sewer

US node ID	DS node ID	US invert level (ft AD)	DS invert level (ft AD)	Length (ft)	Diameter (in)	Slope (ft/ft)	Area of Pipe (ft)	Manning's n	Capacity (cfs)	Section Full Pipe Capacity (MGD)	Section Full Pipe Capacity (MGD)
OP_Chamber_1_Ga	WWRF_SCREEN	-29.5	-29.5	31.2						0	
OP_MH1	OP_Chamber_1	-30.85	-31.79	912	72					87.89	
OP_MH2	OP_MH1	-28.08	-30.85	838.8	72					157.31	
OP_MH3	OP_MH2	-27.51	-28.08	650.2	72					81.05	
OP_MH4	OP_MH3	-26.42	-27.51	794.1	72					101.42	
OP_MH5	OP_MH4	-25.65	-26.42	613	72					97.02	
				3839.3		0.00100	28.27	0.013	134.1		87
OP_MH6	OP_MH5	-16.13	-16.64	699.7	72					73.91	
OP_MH6A	OP_MH6	-16.06	-16.13	214	72					49.51	
OP_Chamber_2	OP_MH6A	-15.95	-16.06	24.2	72					184.52	
				937.9		0.00074	28.27	0.013	114.9		74
									Weighed Average		84
OP_MH7	OP_Chamber_2	-15.32	-15.95	1030.7	72					67.68	
OP_MH8	OP_MH7	-15.29	-15.32	515.1	72					20.89	
OP_MH9	OP_MH8	-14.88	-15.29	362.4	72					92.08	
OP_MH9A	OP_MH9	-14.66	-14.88	200.7	72					90.64	
OP_MH10	OP_MH9A	-14.53	-14.66	162.4	72					77.44	
OP_MH11	OP_MH10	-14.29	-14.53	417.1	72					65.66	
OP_MH12	OP_MH11	-14.06	-14.29	414.9	72					64.46	
OP_MH13	OP_MH12	-13.82	-14.06	420	72					65.43	
OP_MH14	OP_MH13	-13.52	-13.82	420.1	72					73.15	
OP_Chamber_3	OP_MH14	-13.45	-13.52	94.9	72					74.37	
						0.00062	28.27	0.013	105.4		68
OP_MH15	OP_Chamber_3	-13.37	-13.45	170.7	72					59.25	
OP_MH16	OP_MH15	-13	-13.37	487.2	72					75.44	
OP_MH16A	OP_MH16	-12.93	-13	358	72					38.28	
OP_MH17	OP_MH16A	-12.83	-12.93	212.9	72					59.33	
OP_MH18	OP_MH17	-12.65	-12.83	259.9	72					72.04	
OP_MH19	OP_MH18	-12.41	-12.65	457.8	72					62.68	
OP_MH20	OP_MH19	-12.2	-12.41	427.1	72					60.7	
OP_MH22	OP_MH21	-12.13	-12.41	245.7	72					92.41	
OP_MH21	OP_MH20	-12.41	-12.2	489.9	72					-56.68	
OP_MH23	OP_MH22	-12.03	-12.13	554.4	72					36.77	
OP_MH24	OP_MH23	-11.72	-12.03	425.7	72					73.87	
OP_Chamber 4B	OP_MH24	-11.68	-11.72	363.9	72					74.37	
						0.00040	28.27	0.013	84.4		55
OP_MH25	OP_Chamber 4B	-11.61	-11.68	350	72					38.71	
OP_MH26	OP_MH25	-11.65	-11.61	352	72					-29.18	
OP_MH27	OP_MH26	-11.55	-11.65	78	72					98.04	
OP_MH28	OP_MH27	-11.44	-11.55	495.4	72					40.79	
OP_MH29	OP_MH28	-11.1	-11.44	498	72					71.53	
OP_MH30	OP_MH29	-11.1	-11.1	497.9	72					0	
OP_MH31	OP_MH30	-10.71	-11.1	498	72					76.6	
OP_MH32	OP_MH31	-10.72	-10.71	536	72					-11.82	
OP_MH33	OP_MH32	-10.6	-10.72	460	72					44.21	
OP_MH34	OP_MH33	-10.56	-10.6	498	72					24.53	
				4263.3		0.00026	28.27	0.013	68.6		44

BCUA Overpeck Relief Trunk Sewer

US node ID	DS node ID	US invert level (ft AD)	DS invert level (ft AD)	Length (ft)	Diameter (in)	Slope (ft/ft)	Area of Pipe (ft)	Manning's n	Capacity (cfs)	Section Full Pipe Capacity (MGD)	Section Full Pipe Capacity (MGD)
OP_MH35	OP_MH34	-10.48	-10.56	506.4	66					27.28	
OP_MH36	OP_MH35	-10.46	-10.48	233.2	66					20.1	
OP_MH37	OP_MH36	-10.34	-10.46	489.3	66					33.99	
OP_MH38	OP_MH37	-10.09	-10.34	304	66					62.24	
OP_MH39	OP_MH38	-9.77	-10.09	308	66					69.96	
OP_MH40	OP_MH39	-9.45	-9.77	496	66					55.13	
OP_MH41	OP_MH40	-9.32	-9.45	495	66					35.18	
OP_MH42	OP_MH41	-9.39	-9.32	391	66					-29.04	
OP_MH43	OP_MH42	-9.29	-9.39	461.1	66					31.97	
OP_MH44	OP_MH43	-8.96	-9.1	461.7	66					37.8	
OP_MH45	OP_MH44	-8.8	-8.96	454	66					40.75	
OP_MH46	OP_MH45	-8.7	-8.8	463.2	66					31.89	
JC-5	OP_MH46	-11.36	-10.96	48.7	66					-196.79	
				5111.6		0.00037	23.76	0.013	64.4		42
									Weighed Average		43
OP_MH47	JC-5	-8.6	-9.7	90.9	66					238.78	
OP_MH48	OP_MH47	-8.56	-8.6	86.4	66					46.71	
OP_MH49	OP_MH48	-8.46	-8.56	356.7	66					36.34	
OP_MH50	OP_MH49	-8.33	-8.46	452.3	66					36.8	
OP_MH51	OP_MH50	-8.28	-8.33	167.6	66					37.49	
OP_MH52	OP_MH51	-8.15	-8.28	374.3	66					40.45	
OP_MH52A	OP_MH52	-8.15	-8.15	110.8	66					0	
OP_MH53	OP_MH52A	-8.03	-8.15	358.3	66					39.72	
OP_MH54	OP_MH53	-7.97	-8.03	204	66					37.22	
OP_MH55	OP_MH54	-7.82	-7.97	500.9	66					37.56	
JC-6	OP_MH55	-7.76	-7.82	176.2	66					40.06	
				2878.4		0.00030	23.76	0.013	58.3		38
OP_MH56	JC-6	-7.28	-7.76	162.9	42					35.3	
OP_MH58	OP_MH57	-6.94	-7.35	393.8	42					20.98	
OP_MH57	OP_MH56	-7.47	-7.31	101.7	42					-25.8	
OP_MH59	OP_MH58	-6.55	-6.94	394.5	42					20.45	
OP_MH61	OP_MH60	-6.49	-6.66	327.9	42					14.81	
OP_MH60	OP_MH59	-6.66	-6.55	180	42					-16.08	
OP_MH62	OP_MH61	-6.34	-6.49	317.2	42					14.14	
OP_MH63	OP_MH62	-6.2	-6.34	285.5	42					14.4	
OP_MH64	OP_MH63	-6.03	-6.2	347.9	42					14.38	
OP_MH65	OP_MH64	-4.06	-4.15	112.6	42					18.38	
						0.00141	9.62	0.013	37.8		24

As illustrated in Table 8-3, the maximum transport flow capacity for the Relief Sewer ranges from around 84 MGD near the WPCF to a low of 24 MGD upstream of Junction Chamber 6 (JC-6). Again, the model does indicate that the segments of the Relief Sewer near the BCUA WPCF are surcharged due to the influent pumping station and accordingly these segments do not operate under the free flow conditions as required for using Manning's Equation.

As previously noted, the wastewater flows in the Overpeck Valley Trunk and Relief Sewers intermingle at each of the six junction chambers within the system. The hydraulics within each of the junction chambers vary greatly and change as flow patterns within the trunk or relief sewer change. Accordingly, the InfoWorks model could not accurately predict the split as to the amount of flow entering the trunk vs the relief sewer downstream of each chamber. Nevertheless, the model was able to accurately predict the total flow that would enter and exit each chamber. In the same manner, the theoretical flow capacity of the Overpeck Valley Trunk and Relief Sewers is interconnected and can best be described as a total flow capacity between junction chambers. As illustrated in Table 8-4, the total flow capacity of the joint trunk

sewer ranged from approximately 140 MGD near the WPCF to 40 MGD upstream of Junction Chamber 6 as follows:

Table 8-4: Total Theoretical Flow Capacity for Overpeck Valley Trunk/Relief Sewers

Segment Description	Overpeck Trunk Sewer Flow Capacity (MGD)	Overpeck Relief Sewer Flow Capacity (MGD)	Total Flow Capacity Overpeck Trunk and Relief Sewers (MGD)
JC1 – JC2	58	84	143
JC2 – JC3	48	68	116
JC3 – JC4	38	55	92
JC4 – JC5	29	43	72
JC5 – JC6	18	38	55
JC6- MH65	15	24	40

While the maximum flow capacity of the Overpeck Valley Trunk and Relief Sewers is important in understanding the ability of the BCUA Transport System to transport wet weather flows from Fort Lee and Ridgefield Park these municipalities are connected to the Overpeck trunk system by means of smaller branch intercepting sewers owned and operated by the BCUA. Accordingly, it is also important to evaluate the maximum flow capacity available within each of these branch intercepting sewers.

C. Ridgefield Park Branch Intercepting Sewer

Ridgefield Park is tributary to the Overpeck Valley Trunk Sewer by means of the Ridgefield Park Branch Intercepting Sewer that generally runs along the Hackensack River from its connection just south of Overpeck Creek to its northern terminus at Industrial Avenue in Ridgefield Park as illustrated in Figure 8-2.

The Ridgefield Park Branch Intercepting Sewer has three wastewater connection points from the Village of Ridgefield Park These occur at Regulators R-1, R-2, and R-5. While Ridgefield Park has other regulators that they own and operate the dry/wet weather flow from Village owned regulators are tributary to the drainage area associated with one of the above three regulators. The dry/wet weather flows associated with Regulators 004A/004B and 006 are all tributary to the drainage basin for Regulator 005. In the same manner the dry/wet weather flows associated with Regulator 003 are tributary to drainage the basin for Regulator 002.

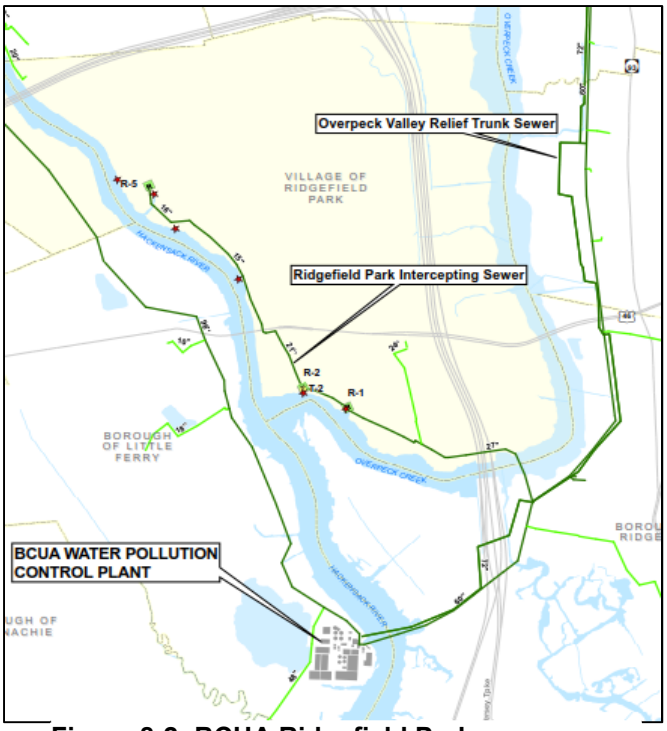


Figure 8-2: BCUA Ridgefield Park Branch Intercepting Sewer

The BCUA Ridgefield Park Branch Intercepting Sewer starts as a 15” pipe at Regulator 005, increases to a 21” pipe near Route 46 and just north of Regulator 002, and then increases to a 27” pipe beyond the connection point for Regulator 001 at the junction with a second BCUA Branch Intercepting Sewer (24”) that services the eastern portion of the Village, which is primarily serviced by separate sewers. As noted in Table 8-5, the average peak flow capacity of the 15” pipe is 1.7 MGD, the average peak flow capacity of the 21” pipe is 3.0 MGD, and the average peak flow capacity of the 27” sewer is 5.7 MGD.

Table 8-5: BCUA Ridgefield Park Intercepting Sewer Peak Capacity

BCUA Ridgefield Park Branch Intercepting Sewer

US node ID	DS node ID	US invert level (ft AD)	DS invert level (ft AD)	Length (ft)	Diameter (in)	Slope (ft/ft)	Area of Pipe (ft)	Manning's n	Full Pipe Capacity (cfs)	Full Pipe Capacity (MGD)	Section Full Capacity (MGD)
RIDGEFIELDPARK_MH_BCUA5+20	BCUA_1051	-13.61	-14	528.6	27					5.05	
RIDGEFIELDPARK_MH_BCUA9+55	RIDGEFIELDPARK_MH_BCUA5+20	-13.28	-13.61	432.3	27					5.14	
RIDGEFIELDPARK_MH_BCUA11+55	RIDGEFIELDPARK_MH_BCUA9+55	-10.12	-10.27	207.4	27					5	
RIDGEFIELDPARK_MH_BCUA15+20	RIDGEFIELDPARK_MH_BCUA11+55	-9.85	-10.12	363.5	27					5.07	
RIDGEFIELDPARK_MH_BCUA19+00	RIDGEFIELDPARK_MH_BCUA15+20	-9.56	-9.85	368.6	27					5.21	
RIDGEFIELDPARK_MH_BCUA22+23	RIDGEFIELDPARK_MH_BCUA19+00	-9.31	-9.56	329.5	27					5.12	
RIDGEFIELDPARK_MH_BCUA22+91.6	RIDGEFIELDPARK_MH_BCUA22+23	-9.25	-9.31	60.2	27					5.87	
RIDGEFIELDPARK_MH_BCUA27+83.5	RIDGEFIELDPARK_MH_BCUA22+91.6	-8.21	-9.25	494.5	27					8.53	
						0.00095	3.98	0.014	8.9		5.7
RIDGEFIELDPARK_MH_BCUA29+20	RIDGEFIELDPARK_MH_BCUA27+83.5	-7.55	-7.64	134.2	21					2.46	
RIDGEFIELDPARK_MH_BCUA29+66	RIDGEFIELDPARK_MH_BCUA29+20	-7.5	-7.55	44.9	21					3.18	
RIDGEFIELDPARK_MH_BCUA31+83	RIDGEFIELDPARK_MH_BCUA29+66	-7.29	-7.5	219.7	21					2.94	
RIDGEFIELDPARK_MH_BCUA35+29	RIDGEFIELDPARK_MH_BCUA31+83	-6.96	-7.29	344.9	21					2.94	
RIDGEFIELDPARK_MH_BCUA39+93	RIDGEFIELDPARK_MH_BCUA35+29	-6.47	-6.96	466	21					3.08	
RIDGEFIELDPARK_MH_BCUA42+72	RIDGEFIELDPARK_MH_BCUA39+93	-6.26	-6.47	278.4	21					2.61	
RIDGEFIELDPARK_MH_F-1	RIDGEFIELDPARK_MH_BCUA42+72	-6.2	-6.26	66.1	21					2.87	
RIDGEFIELDPARK_MH_BCUA44+63	RIDGEFIELDPARK_MH_F-1	-6.05	-6.2	126.8	21					3.27	
RIDGEFIELDPARK_MH_BCUA46+80	RIDGEFIELDPARK_MH_BCUA44+63	-5.84	-6.05	215.7	21					2.97	
RIDGEFIELDPARK_MH_BCUA51+02	RIDGEFIELDPARK_MH_BCUA46+80	-5.41	-5.84	412.6	21					3.07	
RIDGEFIELDPARK_MH_BCUA52+75	RIDGEFIELDPARK_MH_BCUAINT	-5.27	-5.28	12.5	21					2.69	
RIDGEFIELDPARK_MH_BCUAINT	RIDGEFIELDPARK_MH_BCUA51+02	-5.28	-5.41	163.6	21					2.68	
RIDGEFIELDPARK_MH_BCUA55+75	RIDGEFIELDPARK_MH_BCUA52+75	-4.97	-5.27	301.1	21					3	
RIDGEFIELDPARK_MH_BCUA58+66	RIDGEFIELDPARK_MH_BCUA55+75	-4.65	-4.97	375.5	21					2.78	
RIDGEFIELDPARK_MH_BCUA61+97	RIDGEFIELDPARK_MH_BCUA58+66	-4.33	-4.65	230.9	21					3.54	
RIDGEFIELDPARK_MH_BCUA64+65	RIDGEFIELDPARK_MH_BCUA61+97	-3.88	-4.33	293.9	21					3.72	
RIDGEFIELDPARK_MH_BCUA66+41	RIDGEFIELDPARK_MH_BCUA64+65	-3.84	-3.88	181.1	21					1.41	
RIDGEFIELDPARK_MH_BCUA67+33	RIDGEFIELDPARK_MH_BCUA66+41	-3.83	-3.84	50.1	21					1.34	
				1608.7		0.00097	2.41	0.014	4.6		3.0
RIDGEFIELDPARK_MH_BCUA68+84	RIDGEFIELDPARK_MH_BCUA67+33	-3.34	-3.58	150.4	15					1.55	
RIDGEFIELDPARK_MH_BCUA73+80	RIDGEFIELDPARK_MH_BCUA68+84	-2.55	-3.34	499.5	15					1.54	
RIDGEFIELDPARK_MH_BCUA78+59	RIDGEFIELDPARK_MH_BCUA73+80	-1.78	-2.55	487.9	15					1.54	
RIDGEFIELDPARK_MH_BCUA81+06	RIDGEFIELDPARK_MH_BCUA78+59	-1.39	-1.78	247.3	15					1.54	
RIDGEFIELDPARK_MH_BCUA83+00	RIDGEFIELDPARK_MH_BCUA81+06	-1.08	-1.39	246.3	15					1.38	
RIDGEFIELDPARK_MH_BCUA88+30	RIDGEFIELDPARK_MH_BCUA83+00	-0.23	-1.08	463.9	15					1.66	
RIDGEFIELDPARK_MH_BCUA92+43	RIDGEFIELDPARK_MH_BCUA88+30	0.37	-0.23	381.4	15					1.54	
RIDGEFIELDPARK_MH_BCUA93+09	RIDGEFIELDPARK_MH_BCUA92+43	0.54	0.37	106.5	15					1.55	
RIDGEFIELDPARK_MH_BCUA96+58	RIDGEFIELDPARK_MH_BCUA93+09	1.1	0.54	342.5	15					1.57	
RIDGEFIELDPARK_MH_BCUA100+98	RIDGEFIELDPARK_MH_BCUA96+58	1.8	1.1	450	15					1.53	
RIDGEFIELDPARK_MH_BCUA101+43	RIDGEFIELDPARK_MH_BCUA100+98	2.46	2.39	52.2	15					1.42	
RIDGEFIELDPARK_MH_R-5	RIDGEFIELDPARK_MH_BCUA101+43	3	2.46	344.2	15					1.54	
RIDGEFIELDPARK_MH_BCUA105+21	RIDGEFIELDPARK_MH_REG5	4.07	4	26.2	15					2.16	
				2166.9		0.00201	1.23	0.014	2.7		1.7

D. Fort Lee Branch Intercepting Sewers

The Borough of Fort Lee is tributary to the Overpeck Valley Trunk Sewer by means of the North and South Fort Lee Branch Intercepting Sewers that generally run north and west as illustrated in Figure 8-3. The Borough of Fort Lee combined sewer area is not in the Hackensack or Overpeck Valley drainage areas, but is located in the Hudson River drainage area. The Borough of Fort Lee pumps its flow to the BCUA Branch Intercepting Sewers and accordingly it is reasonable that the capacity of the sewers be constant, since there are no flow changes. The North Fort Lee Branch Intercepting Sewer peak flow capacity is illustrated in Table 8-6. As noted in the table the peak flow capacity ranges from 14 – 21 MGD and averages 19.7 MGD. The South Fort Lee Branch Intercepting Sewer peak flow capacity is illustrated in Table 8-7. As noted in the table, the peak flow capacity ranges from 3.8 – 4.7 MGD and averages 4.5 MGD. The anticipated peak flow capacity from both the north and south sewers should average around 24 MGD.

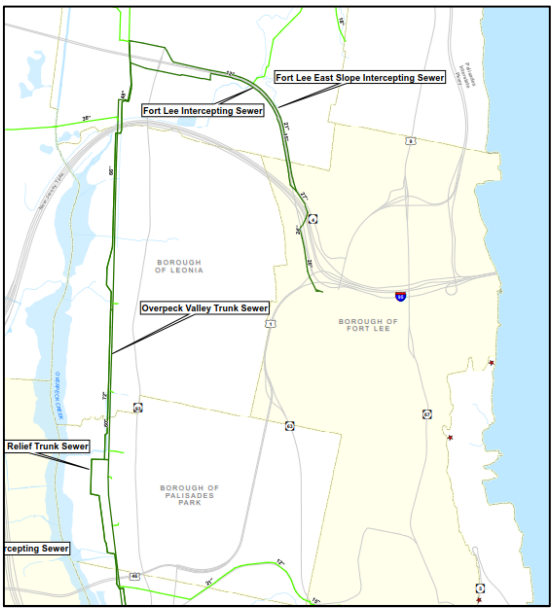


Figure 8-3: BCUA Fort Lee Branch Intercepting Sewers

Table 8-6: BCUA Fort Lee North Branch Intercepting Sewer Peak Capacity

BCUA Fort Lee North Branch Intercepting Sewer

US node ID	DS node ID	US invert level (ft AD)	DS invert level (ft AD)	Length (ft)	Diameter (in)	Slope (ft/ft)	Area of Pipe (ft)	Manning's n	Full Pipe Capacity (MGD)	Section Full capacity (MGD)
OP_MH55A	JC-6	-4.8	-5.27	48.1	24				14.45	
BCUA_1429	OP_MH55A	-2.14	-2.9	26.6	24				24.73	
BCUA_1430	BCUA_1429	2.76	-2.14	333.2	24				17.73	
BCUA_1431	BCUA_1430	9.4	2.76	401.6	24				18.8	
				809.5		0.01812	3.14	0.013		20
BCUA_1432	BCUA_1431	14.45	9.4	225.1	21				15.34	
BCUA_1433	BCUA_1432	16.5	14.45	96	21				14.96	
BCUA_1434	BCUA_1433	30.3	16.5	283.4	21				22.6	
BCUA_1435	BCUA_1434	38.32	30.3	97.7	21				29.35	
				702.2		0.04118	2.41	0.013		21
BCUA_1436	BCUA_1435	47.5	38.32	374.8	24				22.89	
BCUA_1437	BCUA_1436	58.5	47.5	375.1	24				25.04	
BCUA_Break_250	BCUA_1437	66.4	58.5	315.9	21				14.22	
BCUA_1438	BCUA_Break_250	89.2	66.4	865.3	21				16.63	
BCUA_1439	BCUA_1438	110	97.23	248.2	21				23.23	
BCUA_1440	BCUA_1439	117	110.08	165.5	21				18.39	
				1594.9		0.03668	2.41	0.013		20
BCUA_1441	BCUA_1440	127.75	117.17	262.5	18				13.63	
BCUA_1442	BCUA_1441	132.62	127.75	230.5	24				21.26	
BCUA_1443	BCUA_1442	137.18	132.62	212.2	24				21.44	
BCUA_1444	BCUA_1443	139.973	137.18	187.8	24				17.83	
				4832.7		0.01939	3.14	0.013		20
BCUA_1445	BCUA_1444	142.573	139.973	174.9	21				12.49	
BCUA_1446	BCUA_1445	145.168	142.573	174.5	21				12.49	
BCUA_1447	BCUA_1446	149.178	145.168	269.7	21				12.49	
BCUA_1448	BCUA_1447	153.199	149.178	270.4	21				12.49	
FT.LEE_BCUA_27	BCUA_1448	159.5	157.25	239.3	21				8.61	
FT.LEE_BCUA_26	FT.LEE_BCUA_27	163	159.5	172.2	21				12.66	
				1301		0.01770	2.41	0.013		14
FT.LEE_BCUA_25	FT.LEE_BCUA_26	164.56	163	285.1	24				9.37	
FT.LEE_BCUA_MC1	FT.LEE_BCUA_25	167.7	164.56	104.6	24				21.95	
				389.7		0.01206	3.14	0.013		16
FT.LEE_BCUA_24	FT.LEE_BCUA_MC1	169.52	168.4	70.9	21				11.16	
FT.LEE_BCUA_23	FT.LEE_BCUA_24	177.83	174.04	185.1	18				8.42	
FT.LEE_BCUA_20	FT.LEE_BCUA_23	184.19	177.83	278.4	24				19.16	
FT.LEE_BCUA_FlowCh	FT.LEE_BCUA_20	190.8	184.19	267.2	24				19.93	
				545.6		0.02377	3.14	0.013		23
FT.LEE_BCUA_18	FT.LEE_BCUA_FlowCh	195.54	191	449.5	27				17.44	
FT.LEE_BCUA_16	FT.LEE_BCUA_18	199.17	195.54	255.6	27				20.67	
FT.LEE_BCUA_15	FT.LEE_BCUA_16	200.17	199.17	179.5	27				12.95	
				884.6		0.01037	3.98	0.013		20
FT.LEE_BCUA_14	FT.LEE_BCUA_15	204.67	202.95	201.1	24				11.72	
FT.LEE_BCUA_11	FT.LEE_BCUA_14	212.15	207.39	131.6	24				24.11	
FT.LEE_BCUA_09	FT.LEE_BCUA_11	230.27	219.66	547.8	24				17.64	
FT.LEE_BCUA_08	FT.LEE_BCUA_09	234.26	230.21	301.6	24				14.69	
				1182.1		0.02649	3.14	0.013		24
FT.LEE_BCUA_07	FT.LEE_BCUA_08	240.28	234.68	387.4	20				9.37	
FT.LEE_BCUA_05	FT.LEE_BCUA_07	244.33	240.28	262.5	24				15.74	
FT.LEE_BCUA_02	FT.LEE_BCUA_05	248.09	244.33	302.8	24				14.12	
				565.3		0.01382	3.14	0.013		17
									Weighed Average	19.7

Table 8-7: BCUA For Lee South Branch Intercepting Sewer Peak Capacity

BCUA Fort Lee South Branch Intercepting Sewer

US node ID	DS node ID	US invert level (ft AD)	DS invert level (ft AD)	Length (ft)	Height (in)	Slope (ft/ft)	Area of Pipe (ft)	Manning's n	Full Pipe Capacity (MGD)	Section Full Pipe Capacity (MGD)
BCUA_1246	OP_MH54	1.07	-3.92	452.8	15				4.38	
BCUA_1247	BCUA_1246	18.82	1.07	499.9	12				4.34	
BCUA_1248	BCUA_1247	35.73	18.82	489.2	12				4.28	
				989.1		0.03504	0.79	0.013		4.3
BCUA_1249	BCUA_1248	36.77	35.73	43.8	15				6.44	
BCUA_1250	BCUA_1249	38.59	36.77	280.2	15				3.37	
BCUA_1253	BCUA_1250	41.14	38.59	306	15				3.81	
BCUA_1254	BCUA_1253	41.3	41.14	20.8	15				3.66	
BCUA_1251	BCUA_1254	43.1	41.3	217.8	15				3.8	
				868.6		0.00848	1.23	0.013		3.8
BCUA_1252	BCUA_1251	48.23	43.35	147.7	12				4.19	
BCUA_1255	BCUA_1252	64.32	48.23	34.4	12				15.76	
BCUA_1256	BCUA_1255	73.93	64.32	310.7	12				4.05	
BCUA_1257	BCUA_1256	85.09	73.93	280.9	12				4.59	
BCUA_1258	BCUA_1257	86.89	85.09	44.8	12				4.62	
BCUA_1259	BCUA_1258	100.28	86.89	233.8	12				5.51	
BCUA_1260	BCUA_1259	111.91	100.28	241.9	12				5.05	
BCUA_1261	BCUA_1260	116.2	112.95	78.4	12				4.69	
BCUA_1262	BCUA_1261	126.27	116.43	300.1	12				4.17	
BCUA_1263	BCUA_1262	134.84	126.27	306.1	12				3.85	
BCUA_1264	BCUA_1263	140.15	134.84	296.4	12				3.08	
BCUA_1265	BCUA_1264	142.38	140.15	125.9	12				3.07	
				2401.1		0.04124	0.79	0.013		4.7
BCUA_1266	BCUA_1265	145.96	144.9	155.9	15				3.44	
BCUA_1267	BCUA_1266	146.17	145.96	37	15				3.15	
BCUA_1268	BCUA_1267	146.93	146.19	122	15				3.25	
BCUA_1269	BCUA_1268	148.29	146.93	102	15				4.82	
BCUA_1270	BCUA_1269	151.8	148.29	275.1	15				4.72	
FT.LEE_BCUA_36	BCUA_1270	157.38	152.8	274.1	15				4.68	
				966.1		0.01292	1.23	0.013		4.7
								Weighed Average		4.5

8.1.1.2 BCUA Trunk Sewer Capacity from Model

While theoretical peak flow determinations using Manning's Equation can be useful in helping to understand capacity within a system, it is only accurate in areas of constant slope and with free flow conditions. A sewer system that surcharges in upper reaches can increase the peak flow rates downstream beyond their theoretical peak flows predicted by Manning's equation, while surcharging in downstream reaches typically results in a decrease in flow. That is why computer hydraulic models were created so that hydraulic and flow conditions that typically occur within a sewer system can be more accurately determined.

Models are not perfect in that they may not accurately predict all conditions, but on average, they give us a clearer understanding of how the sewer system is operating. A properly calibrated and verified model should be capable of predicting dry and wet weather characteristics within certain guidelines. The most

widely used standard of this kind comes from the Wastewater Planning Users Group (WaPUG) Code of Practice for the Hydraulic Modeling of Sewer Systems. Table 8-8 summarizes the numerical calibration/validation criteria that are part of this standard. The BCUA InfoWorksICM model calibration and verification is detailed in the approved BCUA Sewer System Characterization Report, dated September 26, 2018.

Table 8-8: WaPUG Calibration and Verification Criteria

Category	Dry weather flow	Wet weather flow
Peak flow	±10%	+25% to -15%
Volume	±10%	+20% to -10%
Unsurcharged depth	±4 inches	±4 inches
Surcharged depth	N/A	+20 inches to -4 inches
Time of peaks	Within 1 hour	Similar

Two model runs were conducted to evaluate the BCUA Trunk Sewer System under varying conditions. The first model run was conducted as a continuous simulation using rainfall data from the 2004 typical year and the second was a theoretical artificial rainfall simulation to evaluate the full pipe capacity of segments as well as to evaluate where surcharging or system capacity is an issue. The following sections discuss these evaluations in greater detail.

A. 2004 TYPICAL YEAR ANALYSIS

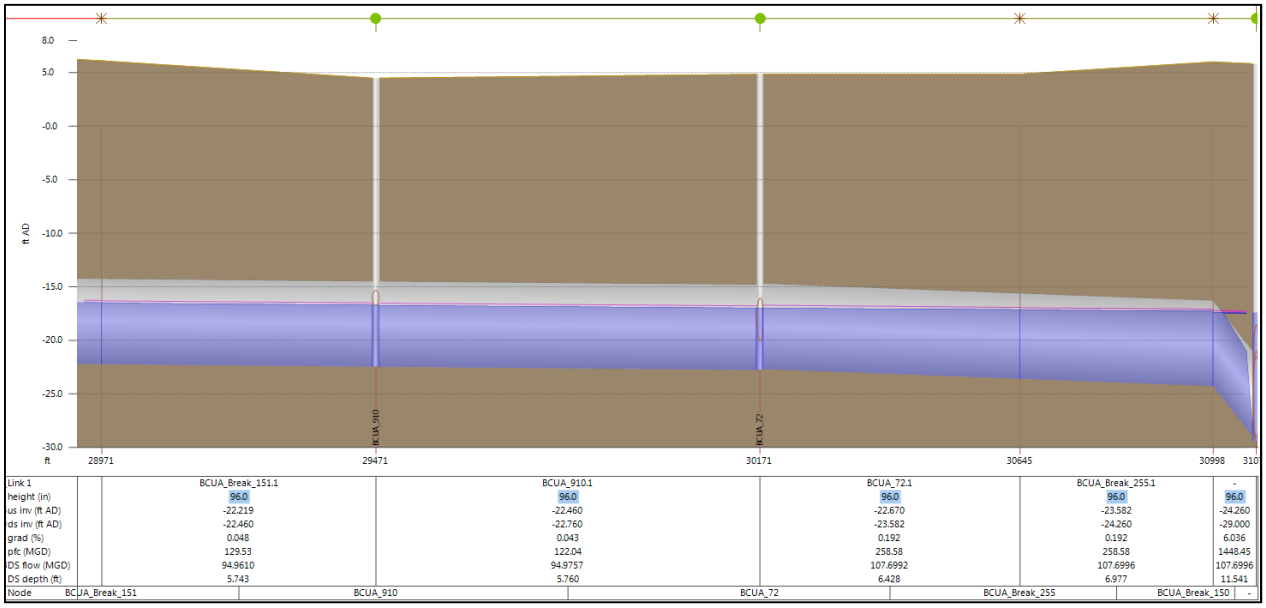
As detailed in the NJPDES, permittees must select, and get approval from the NJDEP, an average rainfall year, known as the typical year, that could be used with a computer model to determine the current conditions such as the frequency of overflows, and then to determine whether the alternative controls as selected can meet the control requirements as specified within the permit. Members of the NJ CSO Group, including the members of the BCUA CSO Group selected rainfall as recorded at the Newark Airport in 2004 as the typical year as detailed in the BCUA Sewer System Characterization Report referenced above. The typical year analysis is a continuous simulation run using InfoWorksICM and the rainfall records from 2004 to determine and evaluate how the overall system responds to rainfall events throughout the year.

It should be noted that the water surface elevation within the wet well varies within a five to six-foot range due to varying flows within the Trunk Sewer, and the operation (on and off) of the various pumps used to transfer flow to the WPCF treatment units. Since we could not model the operations of the pumps, which are typically left to the operator on duty, Mott MacDonald could not predict how the water surface elevation in the wet well would vary over time. Accordingly, an average depth of flow within the wet well was selected for these analyses, and the hydraulic grades as predicted from the model could be more, or less, than illustrated.

Vast quantities of data are generated that can then be used to predict the frequency, peak overflow rates, and overflow volumes that occur within the combined sewer systems within the district. For the BCUA it is important to use the typical year evaluate how the transport and treatment systems respond to individual rainfall events and to evaluate what improvements, if any, are needed to improve overall system capacity. The results of the 2004 analysis will be presented by Trunk Sewer segments.

BCUA Main Trunk Sewer

The BCUA Main Trunk Sewer starts at the BCUA WPCF and services most of the Hackensack River Basin, outside of the Overpeck Valley. This trunk sewer services the combined sewer systems within the City of Hackensack. The area of interest for the Main Trunk Sewer starts at the WPCF and extends north as previously illustrated in Figure 1-3. While a vast amount of data is available for review we will limit the information to that needed to provide a general understanding of how the system responds to wet weather. One area of concern is whether the sewer system surcharges or whether there may be excess sewer capacity available. Pipe hydraulic profiles are being provided as visual documentation of the peak wet weather flows and water surface elevations at several key locations along the Trunk Sewer.



BCUA_910.1	Manhole Reach Designation
96.0	Diameter (in)
-22.460	Downstream Invert (ft)
-22.760	Upstream Invert (ft)
0.043	Pipe Slope (%)
122.04	Full Pipe Capacity (mgd)
94.9757	Downstream Flow (mgd)
5.760	Water Surface Depth (ft)

Figure 8-4: Main Trunk Sewer Near BCUA WPCF

The profile segment illustrated in Figure 8-4 shows the southernmost portion of the Main Truck sewer as it approaches the WPCF. For general reference, the connection at station 72+46 is the junction with the Southwest Trunk Sewer. The numbers located below the profile are defined as noted above. As illustrated above, it appears that the Trunk Sewer under the typical year analysis does not surcharge in the area near the WPCF, however this prediction is based on an average water surface elevation within the influent wet well and thus shows an average condition based on water surface elevations within the wet well. As noted the peak flow predicted within this segment of the Trunk Sewer is around 80 MGD upstream of junction with the Southwest Trunk Sewer and 86 MGD downstream of the junction manhole.

It should be noted that the predicted water surface elevation is approximately 5-6 feet of flow depth in an 8-foot diameter pipe.

Profiles in Figure 8-5 and Figure 8-6 illustrate segments of the Main Trunk Sewer in the region of Hackensack Connection 2 and Connection 1 respectively, while Figure 8-7 illustrates the Main Trunk Sewer upstream of the City of Hackensack. As illustrated, it appears that the Main Trunk Sewer is free flowing and that the flow in the Trunk Sewer increases some 7 MGD from Connection 1 and some 15 MGD from Connection 2. Please note that the information provided in the profile is snapshots in time based on the peak flows predicted within each reach separately throughout the one-year period and that the occurrence of that peak flow may not correspond in time with any other reach. Accordingly, the flows as seen entering the Trunk Sewer from any one connection may not be the peak flow entering the sewer during the modeling period. These profiles are meant to give a general picture of conditions within the Main Trunk during the typical year analysis.

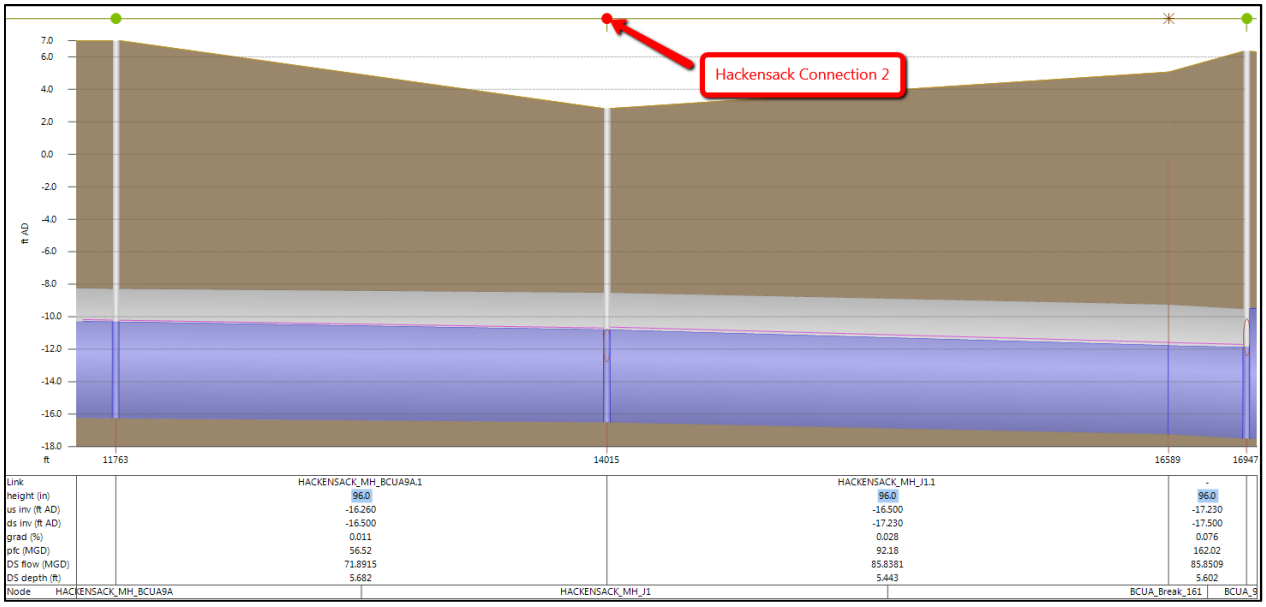


Figure 8-5: Main Trunk Sewer Near Hackensack Connection No.2

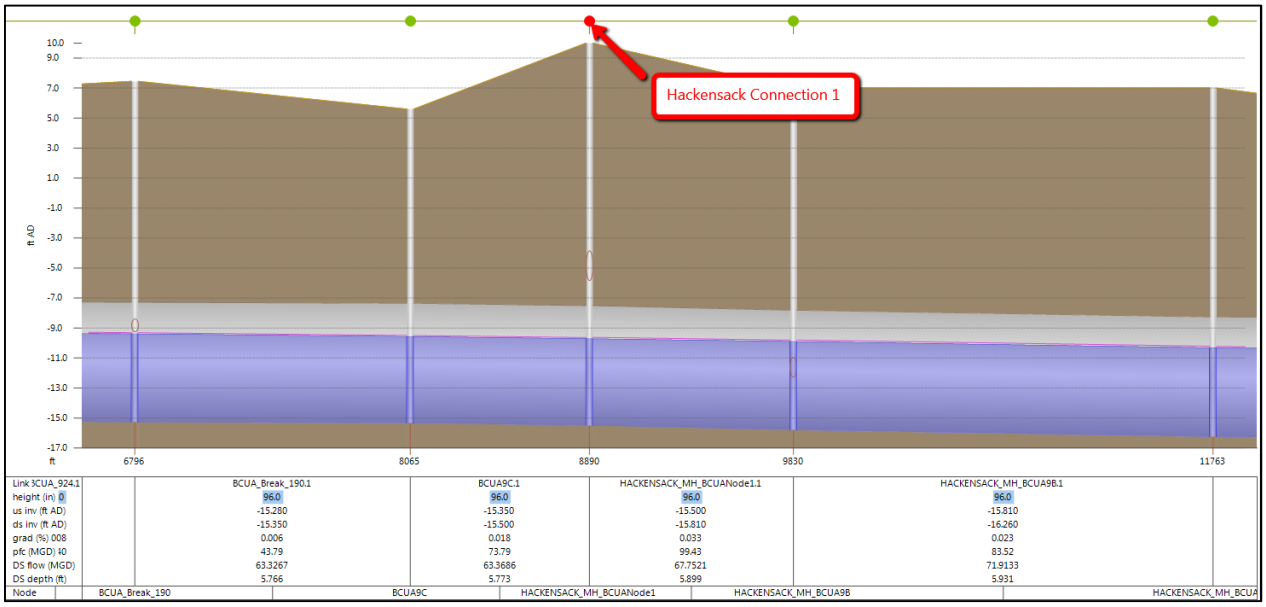


Figure 8-6: Main Trunk Sewer Near Hackensack Connection No. 1

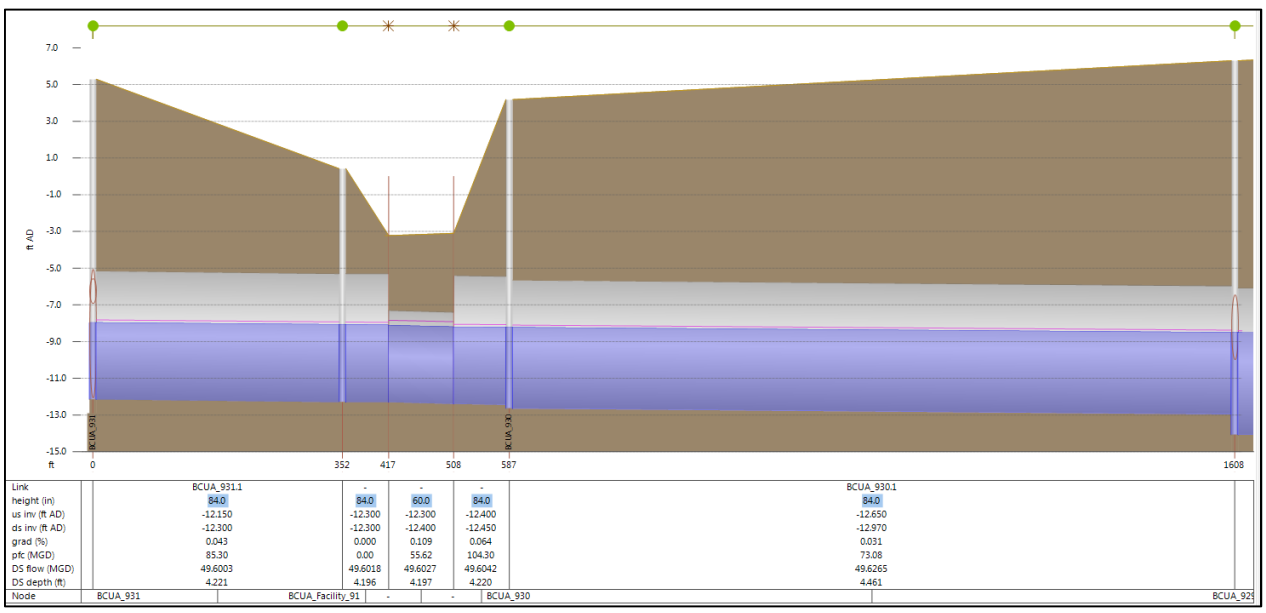


Figure 8-7: Main Trunk Sewer Upstream of City of Hackensack

BCUA Overpeck Valley Trunk and Relief Sewers

The BCUA Overpeck Valley Trunk Sewer (OVTS) and Overpeck Valley Relief Sewer (OVRS) both start at the BCUA WPCF and service most of the Overpeck Creek drainage basin in addition to those

communities within the Hudson River drainage basin. The Trunk and Relief Sewer service the combined sewer systems within the Borough of Fort Lee and the Village of Ridgefield Park. The area of interest for the Overpeck Trunk and Relief Sewers starts at the WPCF and extends east and north as previously illustrated in Figure 1-2 and Figure 1-4. As with the Main Trunk, we will limit the information, to that needed to provide a general understanding of how the system responds to wet weather. One point of interest is whether the sewer system surcharges and whether there is adequate sewer capacity available for wet weather flows. Hydraulic profiles are being provided as visual documentation of the peak wet weather flows and water surface elevations at several key locations along the Overpeck Trunk and Relief Sewers. As previously noted the Trunk and Relief Sewers are interconnected by means of six junction chambers where flow from each enters and then splits at the effluent side of each chamber. Accordingly, the total flow transported by the Trunk and Relief Sewers is the total entering and/or existing each junction chamber.

For general reference, Figure 8-8 illustrates the general piping arrangement between the OVRS and OVTS near the BCUA WPCF. The OVRS enters Junction Chamber 1, which was constructed as a junction chamber between the existing 96” diameter Main Sewer and 72” diameter OVRS, which are both tributary to the bars screens No. 2, while the 60” diameter OVTS is directly tributary to bar screens No.1. All wastewater flows intermingle near the plant and therefore may be tributary to either screen bank.

The profile segments illustrated in Figure 8-9: (OVRS) and Figure 8-10 (OVTS) show the southernmost portion of the Trunk and Relief Sewers sewer as they approach the WPCF. For reference, the connection at station 45+08 on the OVRS is at Junction Manhole 1, as noted above. The numbers located below the profile are defined as noted above. As illustrated below, it appears that both the OVRS and OVTS surcharge in the area near the WPCF due to the water surface elevation at the influent pumping station. As previously noted, the water surface elevation and hydraulic grades as illustrated are based on an average water surface elevation within the influent wet well and thus show an average condition. As previously noted, the model was not able to consistently predict the flow split downstream of the junction chambers because of the complex hydraulic interactions between the Trunk and Relief as flows rates varied. Nevertheless, the total flow passing downstream of each junction was accurate. The peak flow predicted within this segment of both the OVRS and OVTS system is around 53 MGD.

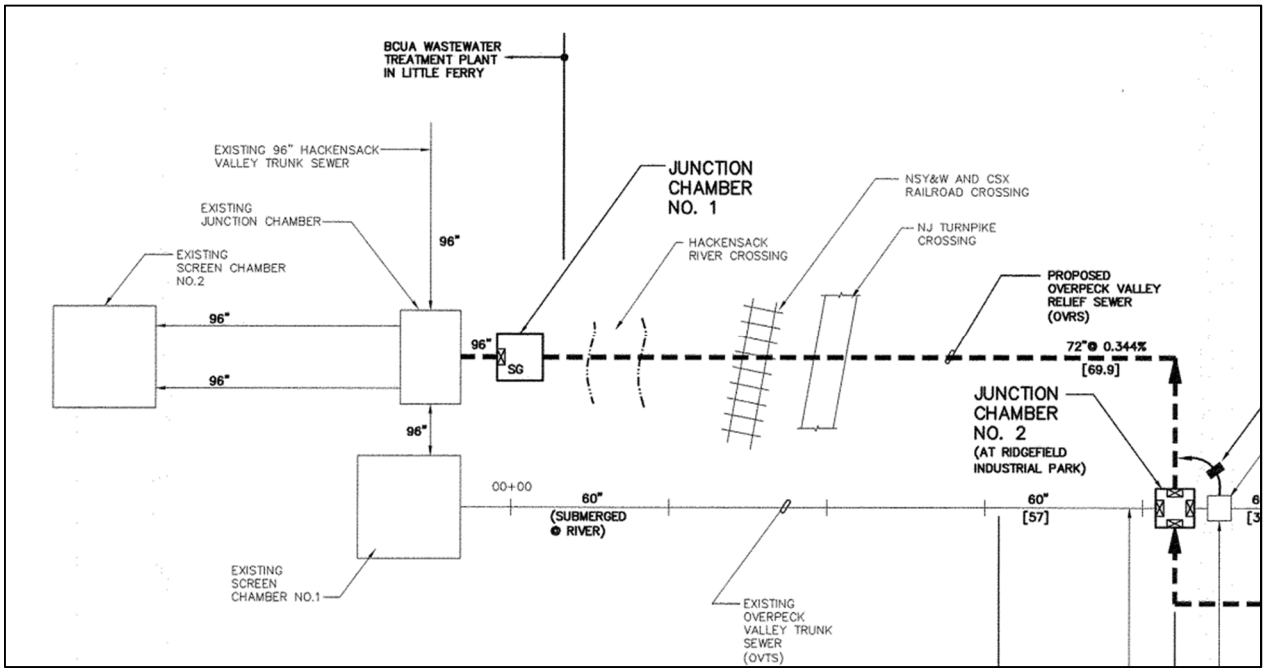


Figure 8-8: Schematic of OVTS and OVRS Near WPCF

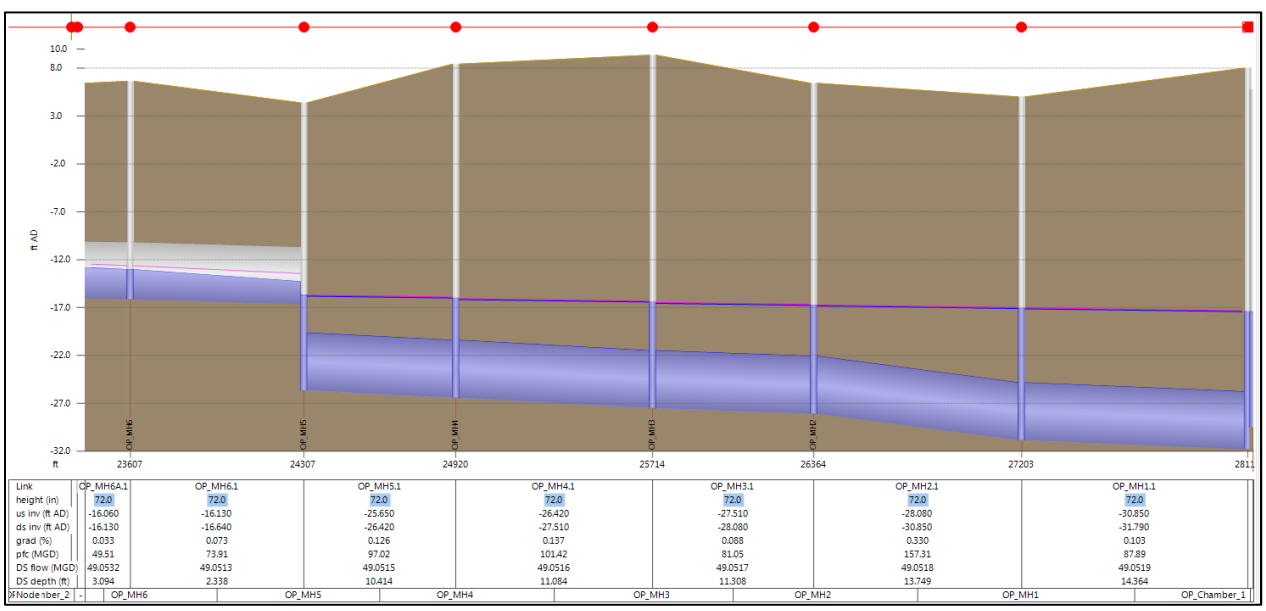


Figure 8-9: Overpeck Valley Relief Sewer Profile at WPCF

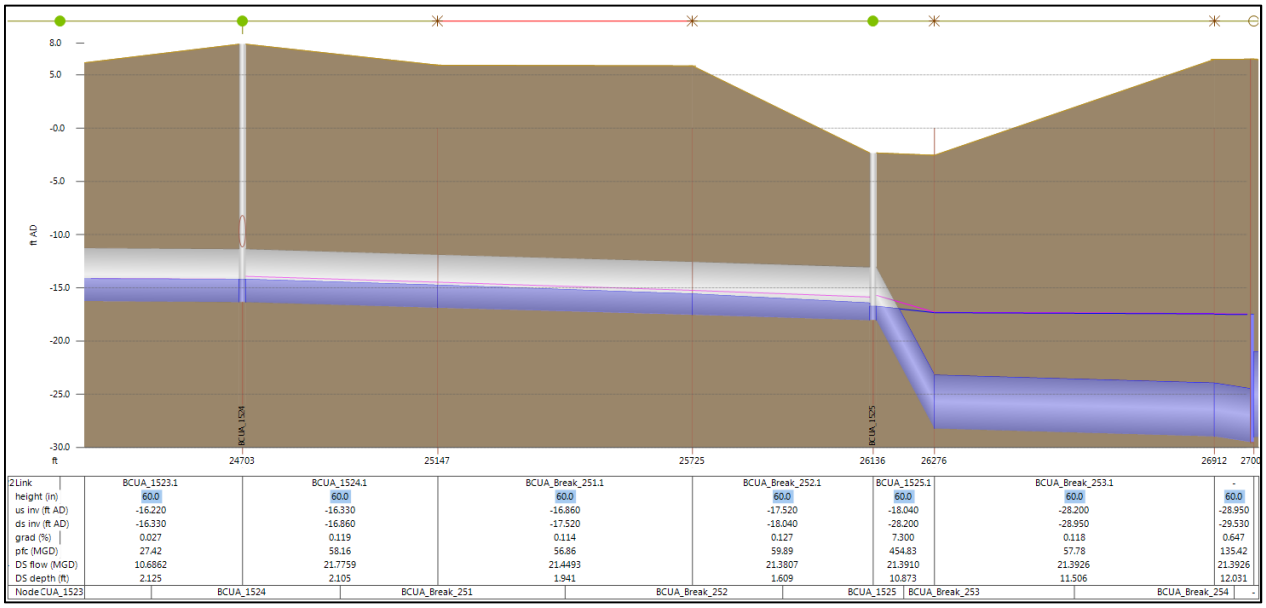


Figure 8-10: BCUA Overpeck Valley Trunk Sewer Profile at the WPCF

Profiles in Figure 8-11 (OVRS) and Figure 8-12 (OVTS) illustrate segments of the Overpeck Valley Trunk Sewers in the region of Junction 2, and the connection of the Ridgefield Park Branch Intercepting Sewer (Village of Ridgefield Park) one reach upstream of the chamber. As illustrated, it appears that the Overpeck Valley Trunk Sewer is free flowing and that the flow in the Trunk Sewer increases from Ridgefield Park. Please note that the information provided in the profile is snapshots in time based on the peak flows predicted within each reach separately throughout the one-year period and that the occurrence of that peak flow may not correspond in time with any other reach. Accordingly, the flows as seen entering the Trunk Sewer from any one connection may not be the peak flow entering the sewer during the modeling period. These profiles are meant to give a general picture of conditions within the Trunk and Relief sewers during the typical year analysis.

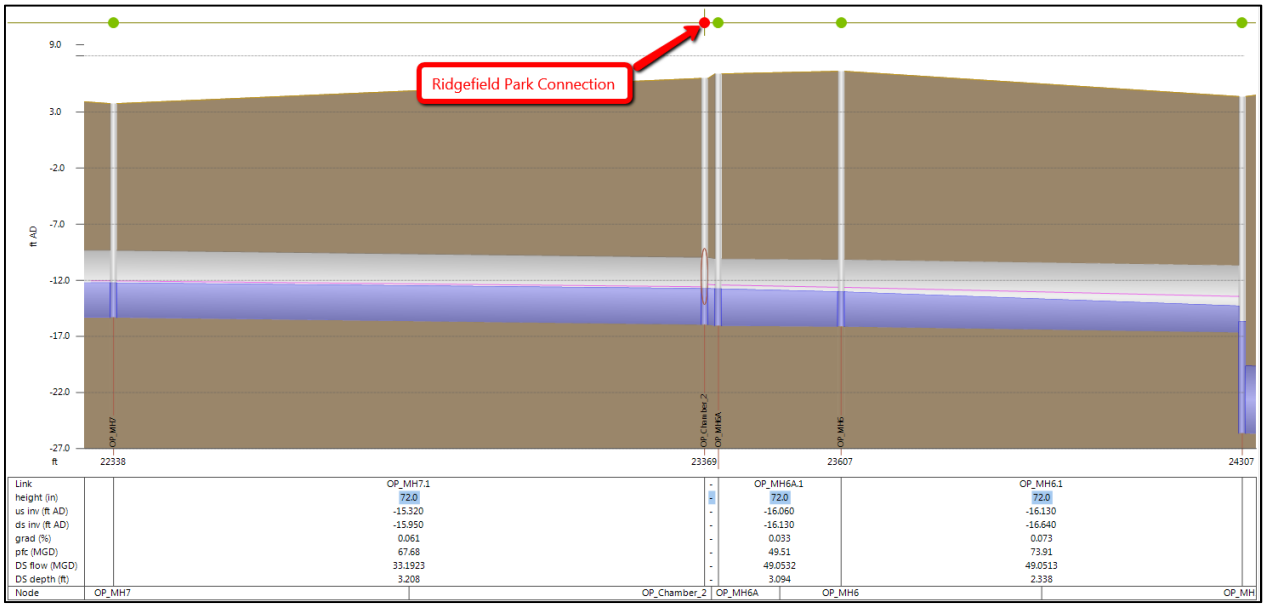


Figure 8-11: OVRS at Junction Chamber 2

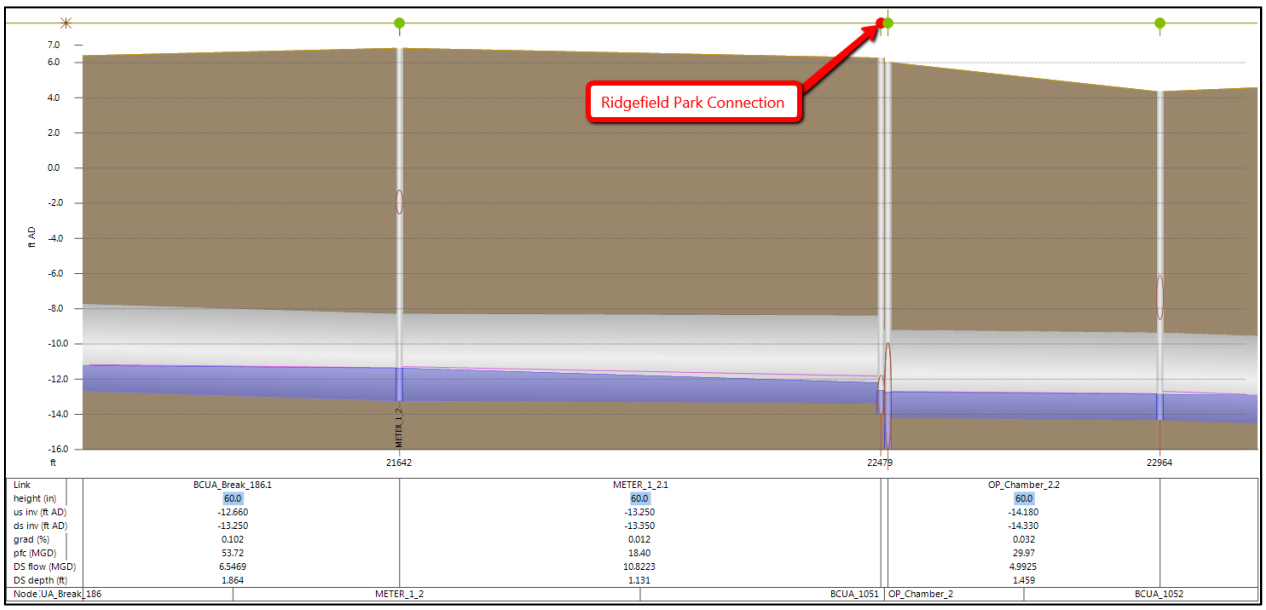


Figure 8-12: OVTS at Junction Chamber 2

As noted above, the Ridgefield Park Branch Intercepting Sewer (RPBIS) connects to the OVRS just upstream of JC-2. Figure 8-13 to Figure 8-15 illustrate the peak hydraulic grade in the RPBIS from the OVRS to the end of the sewer at Regulator 005 near Industrial Avenue. While the 2004 typical year analysis indicates that the BCUA Main and Overpeck Valley Trunk Sewers do not surcharge the downstream end of

the Ridgefield Park Branch Intercepting Sewer, it has limited capacity and the model does indicate that the sewer may surcharge during the typical year.

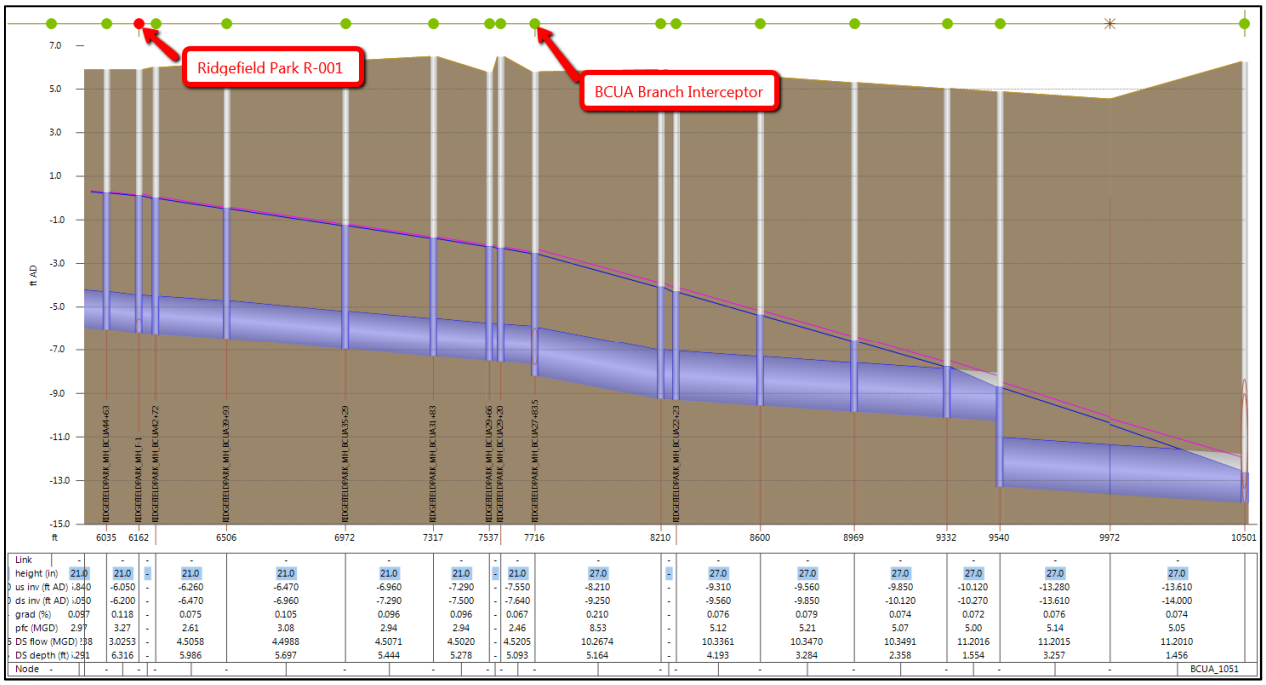
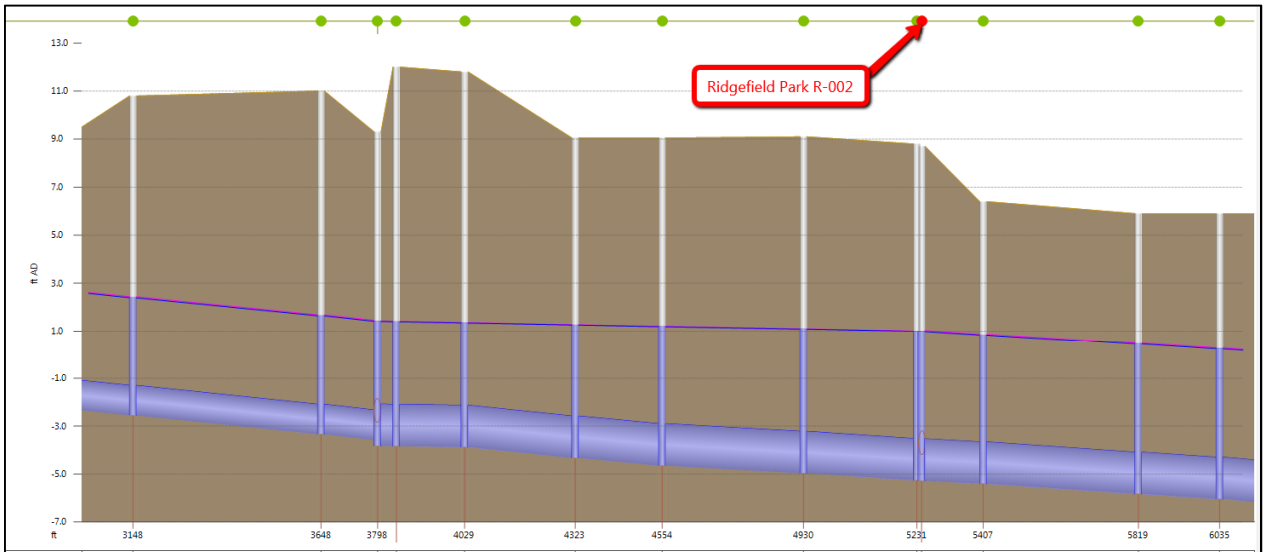


Figure 8-13: RPBIS from OVRS to Regulator 001



CLink K_MH_BCUA78+59.1	RIDGEFIELDPARK_MH_BCUA73+80.1	-	-	-	-	-	-	-	-	-	-	-
height (in)	15.0	15.0	-	21.0	21.0	21.0	21.0	21.0	21.0	21.0	21.0	21.0
us inv (ft AD)	-2.550	-3.340	-	-3.840	-3.880	-4.330	-4.650	-4.970	-5.280	-5.410	-5.840	-6.050
ds inv (ft AD)	-3.340	-3.580	-	-3.880	-4.330	-4.650	-4.970	-5.270	-5.410	-5.840	-6.050	-6.200
grad (%)	0.158	0.160	-	0.022	0.153	0.139	0.085	0.100	0.079	0.104	0.097	0.118
pfc (MGD)	1.54	1.55	-	1.41	3.72	3.54	2.78	3.00	2.68	3.07	2.97	3.27
DS flow (MGD)	2.4355	2.4553	-	2.5712	2.3548	2.0777	2.1092	2.1897	3.0290	3.0275	3.0238	3.0253
DS depth (ft)	4.984	4.994	-	5.224	5.588	5.841	6.052	6.265	6.242	6.286	6.291	6.316
Node	RIDGEFIELDPARK_MH_BCUA73+80	-	-	-	-	-	-	-	-	-	-	-

Figure 8-14: RPBIS Regulator 002

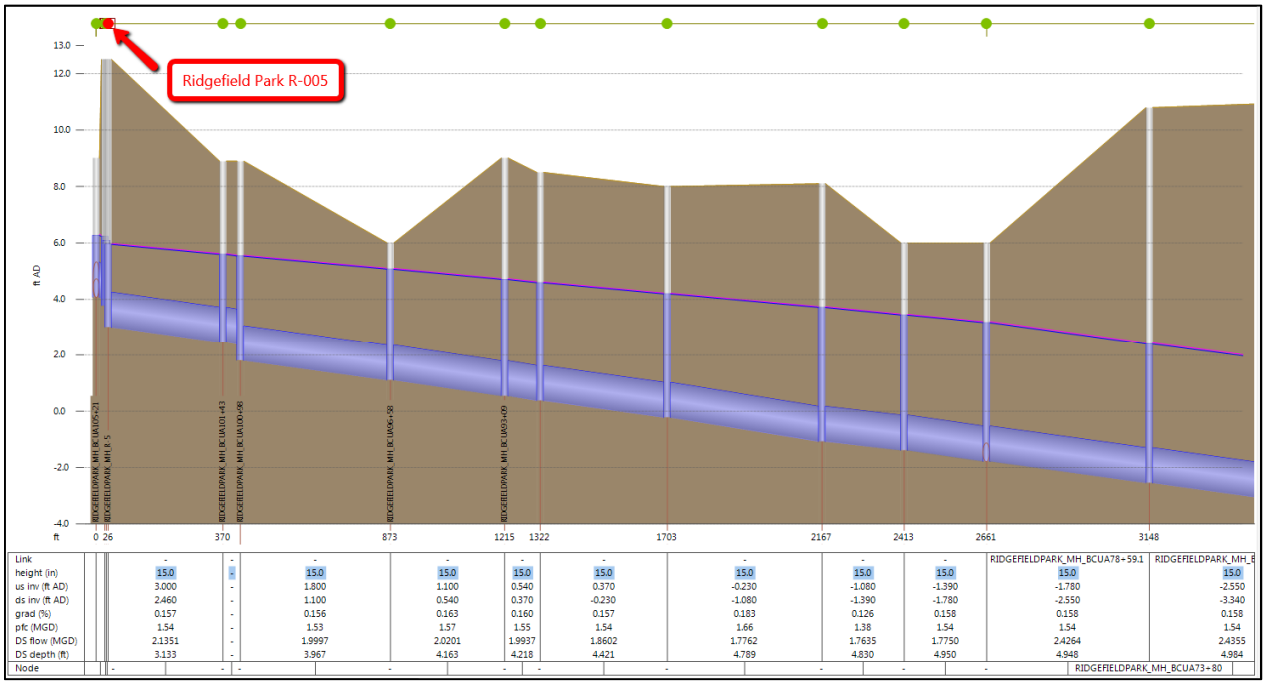


Figure 8-15: RPBIS Regulator 005

Profiles in Figure 8-16 (OVRS) and Figure 8-17 (OVTS) illustrate segments of the Overpeck Valley Trunk Sewers in the region of Junction Chamber 6, and the connection of the Fort Lee Branch Intercepting Sewers. As illustrated, it appears that the Overpeck Valley Trunk Sewer is free flowing and that the flow in the Trunk Sewer increases from Fort Lee. Please note that the information provided in the profile is a snapshot in time based on the peak flows predicted within each reach separately throughout the one-year period and that the occurrence of that peak flow may not correspond in time with any other reach. Accordingly, the flows as seen entering the Trunk Sewer from any one connection way not be the peak flow entering the sewer during the modeling period. These profiles are meant to give a general picture of conditions within the Trunk and Relief sewers during the typical year analysis.

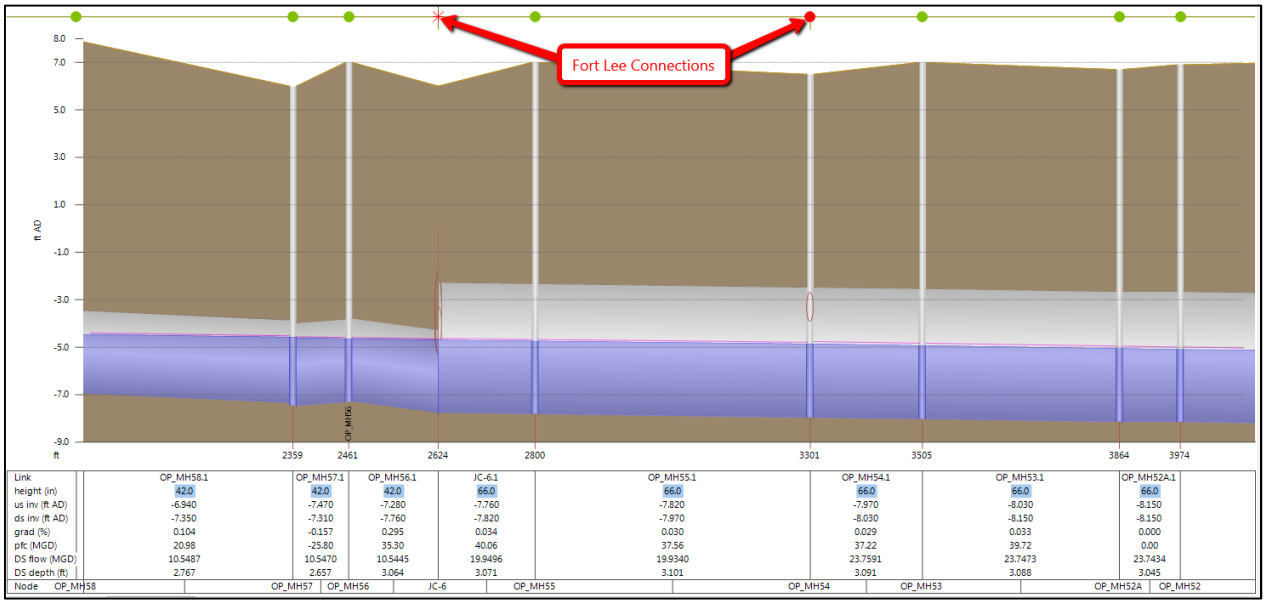


Figure 8-16: OVRS Fort Lee Connections

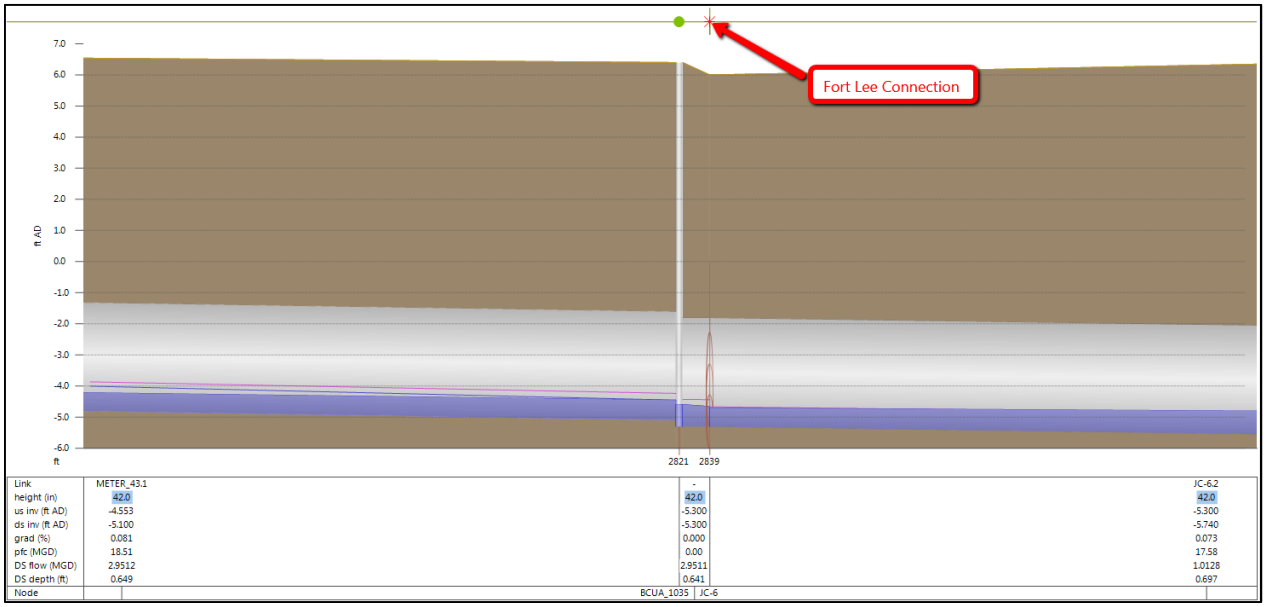


Figure 8-17: OVTS Fort Lee Connections

B. BCUA Transport System Maximum Flow Capacity Review

As noted above, the 2004 run of the BCUA InfoWorks Model indicated that the BCUA Trunk Sewer System does not surcharge in the lower reaches downstream of the combined sewer communities during the typical year. While the flow capacity of various segments of the BCUA Trunk System were provided, Manning's Equation only evaluates the impact of pipe geometry on maximum pipe flow of a single reach.

The InfoWorks model overcomes these difficulties by using hydraulics to understand the interaction between sewer reaches, and thus provides a more realistic evaluation of the capacity of the Trunk Sewer System.

The BCUA transport system capacity was evaluated using the InfoWorksICM model to evaluate how the capacity of the system might change when the hydraulic grade within the sewer reaches the crown in any portion of the trunk sewer. The model was thus setup with a theoretical rainfall event (storm) by increasing the rainfall intensity over the entire district by 0.05 inches per hour. This method continues to simulate the impacts of wet weather within the separate and combined sewer systems. This could be viewed as a theoretical look at how the transport system may react to a hurricane, when large rainfall volumes are recorded during a relatively short period.

The general result of this evaluation has been broken down by trunk sewer segment (Main Trunk, Overpeck Valley Trunk, and Overpeck Valley Relief Sewers.

Main Trunk Sewer

The BCUA InfoWorks model includes limited sewer information on tributary sewers and thus it is important that surcharging of a branch sewer or interceptor be considered in determining the peak flow that could be transported to the WPCF. While the surcharging of a branch connection does not necessarily mean that piping within the drainage system is surcharged, it does warrant additional investigations to assure that any surcharge does not have a detrimental impact on the upstream drainage basin. Accordingly, the peak flow analysis of the main has been broken down into several specific intervals that occurred during the computer simulation including the surcharge of the incoming sewer, or surcharge along the trunk sewer at any one or more relations. The segments illustrated extend from north of Hackensack to the WPCF.

The first point of interest on the Main Trunk Sewer occurred at 8:15 of the run as illustrated in Figure 8-18.

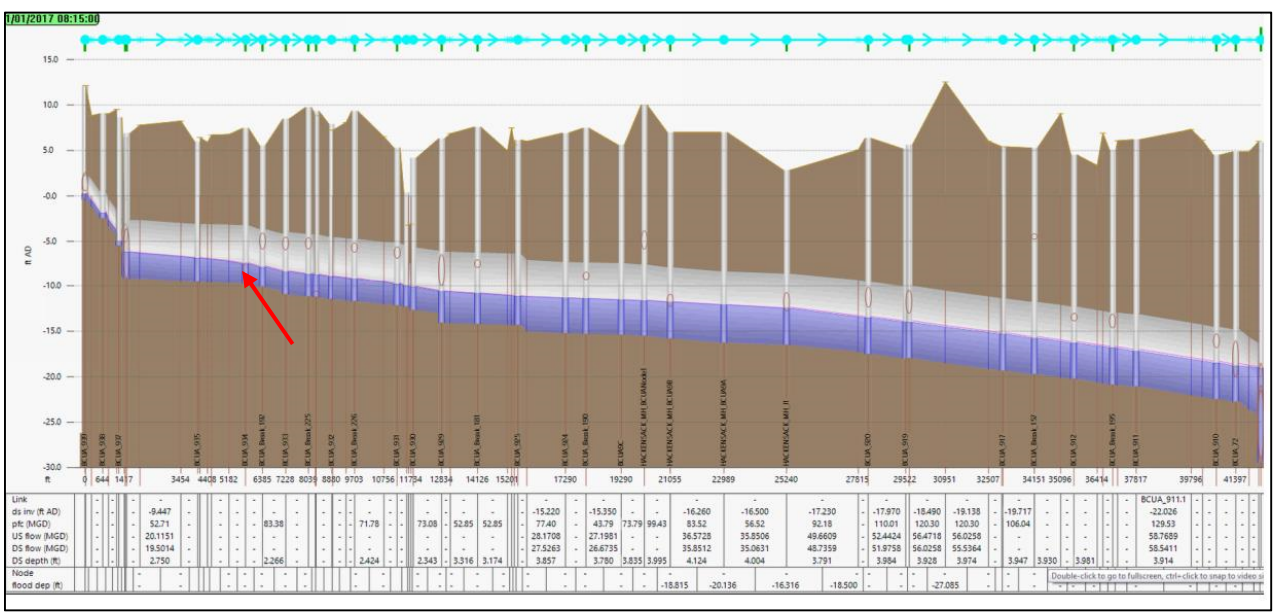


Figure 8-18: Main Trunk Sewer Hydraulic Grade at Step 8:15

As noted above, the crown of the connection at BCUA MH-934 is below the water surface elevation within the Main Trunk. While not all data is illustrated below the profile due to the scale being presented, from the data visible it appears that at this point in time the Main Trunk Sewer is operating at about 50% of its full pipe capacity along this segment of the sewer.

The next time step of interest is when segments of the Main Trunk Sewer reach a surcharged condition as illustrated below in Figure 8-19. As shown in Figure 8-20, it appears that the surcharged condition transfers both up and downstream, except of the segment of the Main Trunk Sewer near the junction with the Southwestern Trunk Sewer. Surcharging of the Main Trunk Sewer does not significantly increase the transport capacity of the sewer, but could have detrimental impacts upstream. Accordingly, any modifications to the overall system to control CSOs needs to be completed without surcharging the trunk sewer.

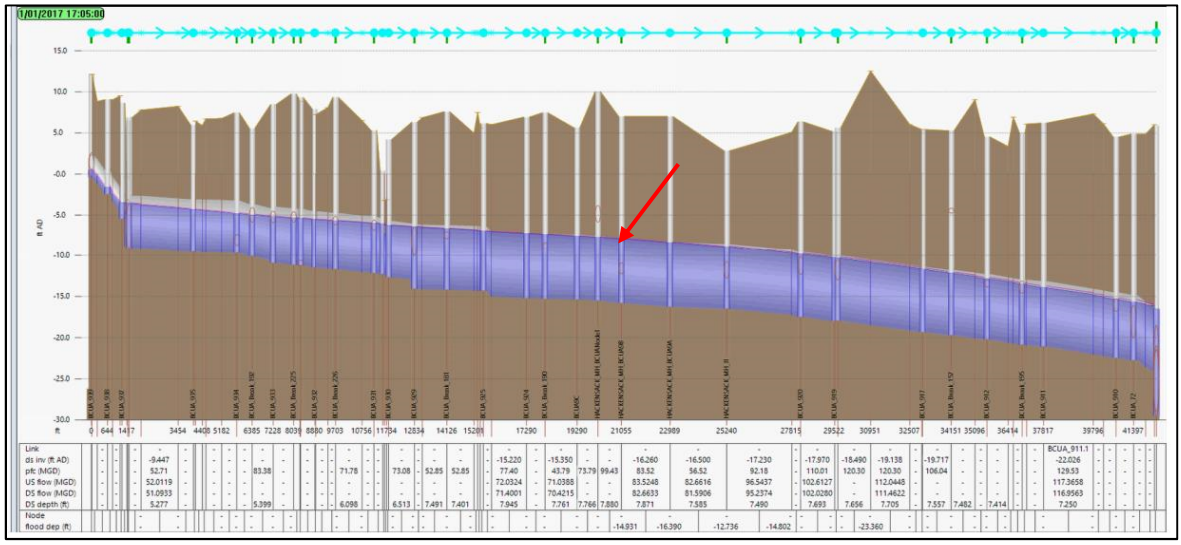


Figure 8-19: Main Trunk Sewer Hydraulic Grade at Step 17:05

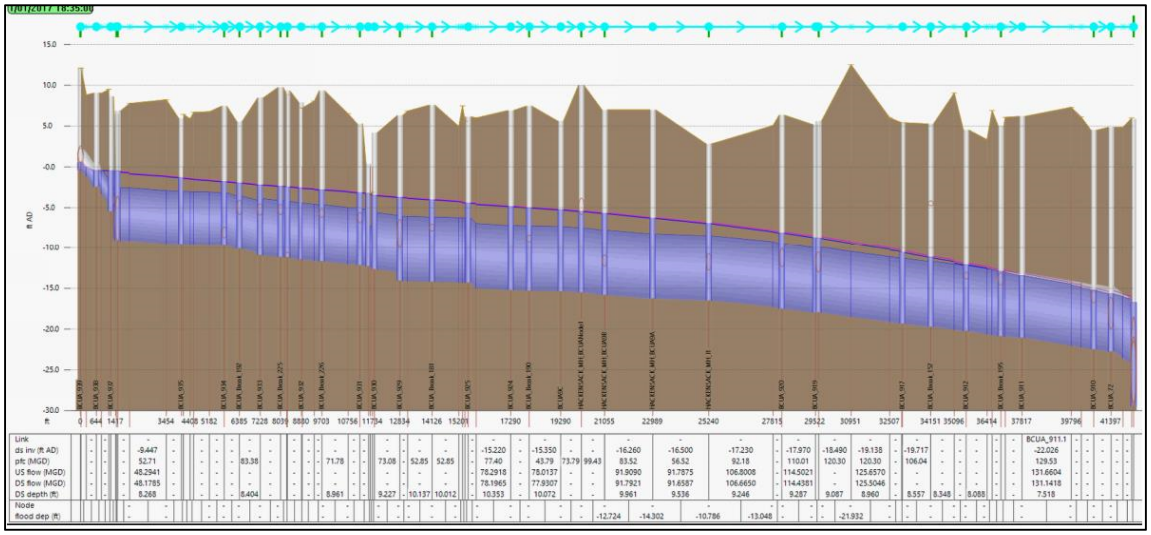


Figure 8-20: Main Trunk Sewer Hydraulic Grade at Step 18:30

Overpeck Valley Trunk and Relief Sewers

The BCUA finished construction of the Overpeck Valley Relief Sewer in 2010 to eliminate capacity issues during significant wet weather events and sanitary sewer overflows within the Overpeck Valley Trunk Sewer. The Relief Sewer is a relatively recent addition to the BCUA Transport System and thus it was anticipated that this portion of the BCUA Transport System has more than adequate capacity. Nevertheless, the model was exercised using the rainfall step analysis on the Overpeck Valley Trunk/Relief Sewers to determine how these segments of the system respond to wet weather events within the region.

As previously noted, the first surcharging of the Main Trunk in the area of the combined sewer systems occurred at Step 17:05 in the computer run. Figure 8-21 illustrates the hydraulic grade in the Overpeck Valley Trunk Sewer at approximately the same point in time. As shown, the model indicates that the Overpeck Trunk Sewer should be free flowing at approximately half of its full pipe capacity at this point in time.

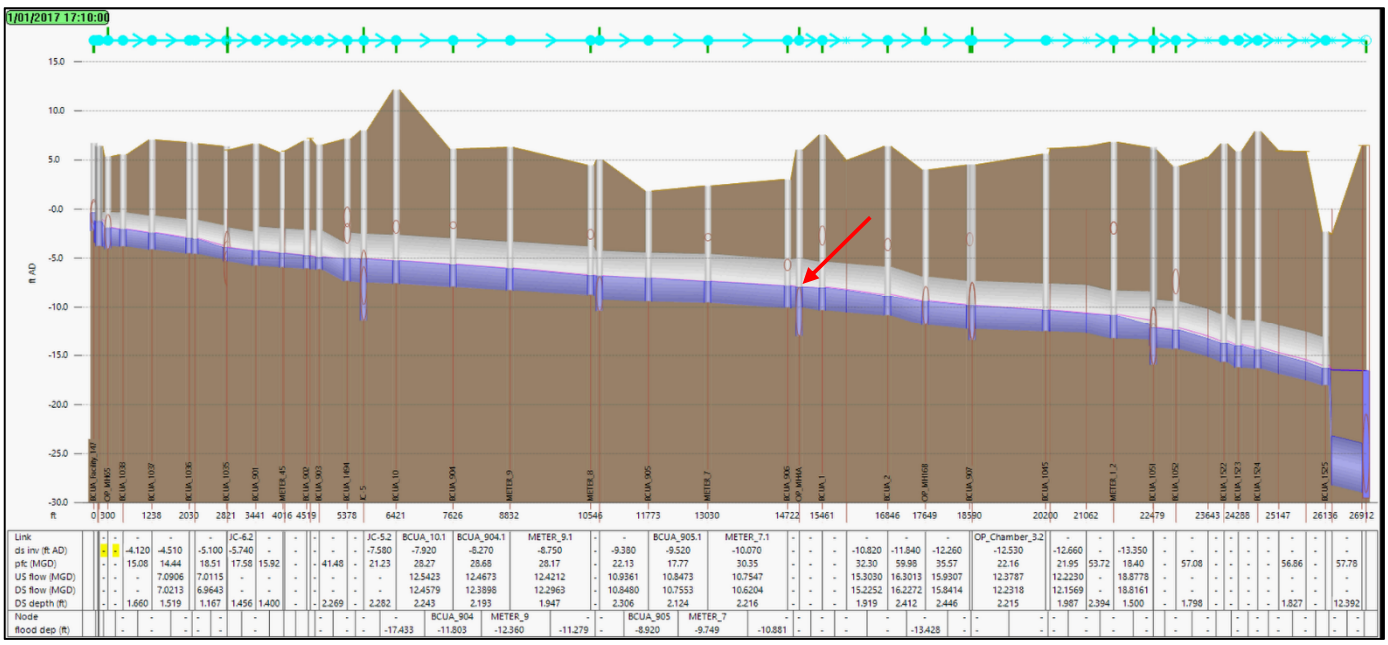


Figure 8-21: Overpeck Valley Trunk Sewer at Step 17:10

The Overpeck Valley Relief Sewer was designed to relieve surcharging issues within the Trunk. The Relief sewer is a 72” diameter pipe near the WPCF and was designed to have a lower invert that the 60 inch Trunk sewer and therefore should, in theory carry the bulk of the dry and wet weather flows to the WPCF. Figure 8-22 illustrates the Overpeck Valley Relief Sewer at Step 17:10. As previously noted, the lower reaches operate under surcharge due to the hydraulic head at the influent screens and pumping stations. The middle segment of the Relief Sewer is flowing approximately 60% full, but the upper reaches are flowing surcharged at the same point where the OVTS Sewer surcharges. It appears that the upper reaches of the Trunk sewer are the controlling factor and that any wet weather flows within the Relief Sewer above the peak flow of approximately 60 MGD as noted in the lower regions could result in additional surcharging upstream, however, additional modeling will be required to evaluate this further.

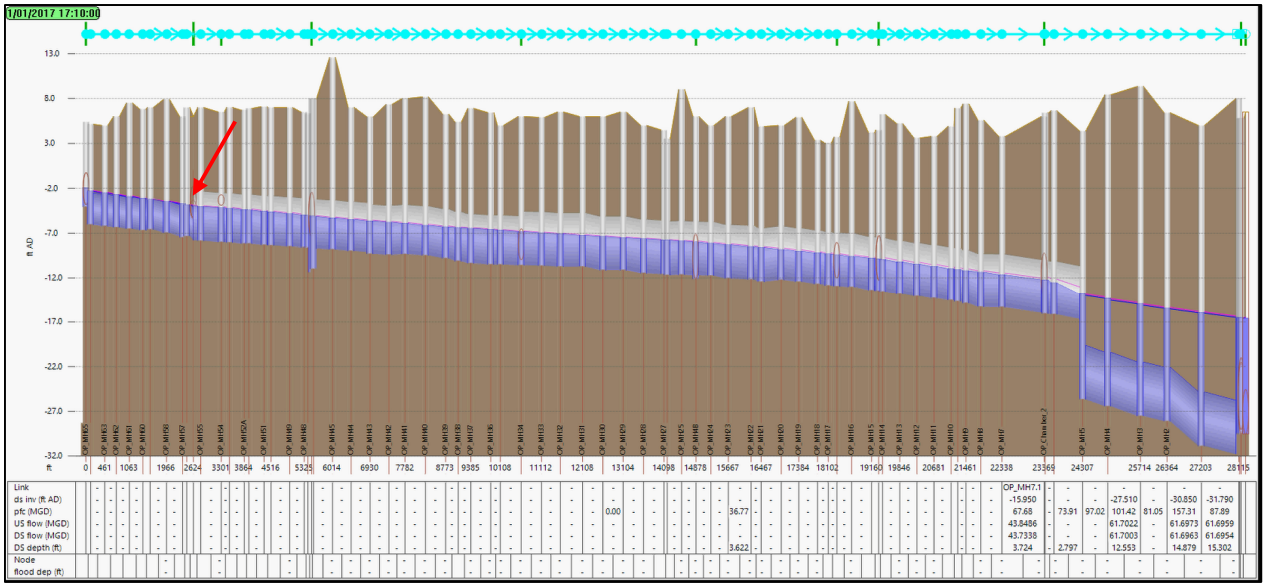


Figure 8-22: Overpeck Valley Relief Sewer at Step 17:10

8.1.1.3 Summary of Interceptor Capacity

As the above analysis demonstrates, the performance of the BCUA interceptors is complex and depends on many interrelated factors such as level control on wet well, what the critical points in the system are defined to be, and the distribution of flow entering the interceptor. There is no one fixed number to represent the capacity of an interceptor system, as the flows vary throughout the interceptors and the downstream boundary conditions vary as well. A conservative approach to determine the capacity was developed by compiling the theoretical pipe capacity, the typical year analysis and the maximum flow capacity analysis. Capacity was selected as the highest flow that was not associated with surcharging the interceptor or connecting pipes. This gives a conservative number for capacities, particularly for the OVRS due to its interconnection with the OVTS and upstream surcharging.

The theoretical flows as determined through Manning's Equation indicate that the Main Trunk Sewer has a capacity of approximately 120 MGD, and that the joint Overpeck Valley Trunk and Relief Sewers have a combined capacity of approximately 143 MGD. Together these sewers should theoretically be able to transport approximately 265 MGD to the WPCF, which is consistent with historic peak flows observed at the WPCF. Notwithstanding the theoretical calculation, the InfoWorksICM model step rainfall analysis indicates that the maximum flow capacity transfer through the Overpeck Valley Trunk Sewers is controlled by the upstream components of these sewers. Accordingly, the flows introduced downstream must be such that they do not negatively impact the upstream hydraulic grades to an extent that surcharging and potential backups are created upstream. Table 8-9 notes the peak transport flow to the WPCF based on the step rainfall analysis indication that a surcharged condition is predicted in some region of the Trunk Sewer.

Table 8-9: Peak Transport Flow to the WPCF

Trunk Sewer Description	Max Flow (MGD)
Main Trunk Sewer	128
Overpeck Valley Trunk Sewer	62
Overpeck Valley Relief Sewer	17
Total Flow to WPCF	207

8.1.2 Analysis of the WPCF

The BCUA retained Arcadis, their Special Consulting Engineer, to undertake the investigations needed to bypass the secondary treatment unit processes and/or providing additional treatment capacity at the Little Ferry WPCF. Arcadis provided the BCUA with a technical memorandum titled "BCUA Little Ferry WPCF CSO Treatment Alternatives Evaluation. This memo served as the basis for the Bypass Alternative.

8.2 Preliminary Control Program Alternatives

8.2.1 Control Program 1 – Expansion of WPCF Capacity

In addition to the limitations on the capacity of the interceptor system, the treatment capacity at the BCUA WPCF is limited. The BCUA WPCF has excess pumping capacity and hydraulic capacity, however the unit treatment processes cannot accommodate unlimited flows. Thus, while more flow could be sent through the plant, it may not receive adequate treatment and could result in washing out the plant's biological processes. Arcadis calculated the plant's treatment capacity as 133 MGD under ideal condition, however, when accounting for actual plant data, New Jersey Administrative Code (N.J.A.C.) 7:14A-23, and 10 States Standards, the wet weather treatment capacity of the plant was estimated to be 120 MGD. To provide secondary treatment for additional wet weather flows to accept additional combined sewage, the capacity of plant would need to be expanded. The feasibility of expanding the treatment capacity of the BCUA WPCF was investigated in 2007 as part of the prior LTCP. During that investigation, increases in full treatment capacity were evaluated by replicating the existing treatment batteries A through D, which provide 109 MGD of treatment capacity under the Ten States Standards. The 2007 evaluation investigated the required facilities and costs to increase the treatment capacity by:

- 29 MGD,
- 58 MGD,
- 86 MGD and
- 115 MGD

This alternative consists of providing additional full treatment capacity by constructing:

- A diversion from the plant headworks to the new facilities
- Grit removal system
- Primary settling tanks
- Aeration tanks

- Final settling tanks
- Chlorine contact tanks
- Sludge storage and handling facilities
- Support buildings for:
    - Electrical equipment
    - Chemical storage
    - Blower building
- Ancillary site improvements and utility relocations

There is no room to expand the existing facilities by constructing additional treatment batteries parallel to the existing ones. The most likely site for additional facilities would be to the southwest of the existing plant. Flows will need to be pumped from the plant headworks to the new treatment facilities. Conceptual layouts developed during the 2007 work are depicted below in Figure 8-23 through Figure 8-26. These facilities would only be needed intermittently during wet weather and the ability and costs of maintaining the biological treatment process between wet weather events are uncertain. Accordingly, other forms of treatment such as ballasted flocculation may be more appropriate. However, in either case it is anticipated that the costs for expansion of the WPCF will be prohibitive, but will be reviewed in more detail.

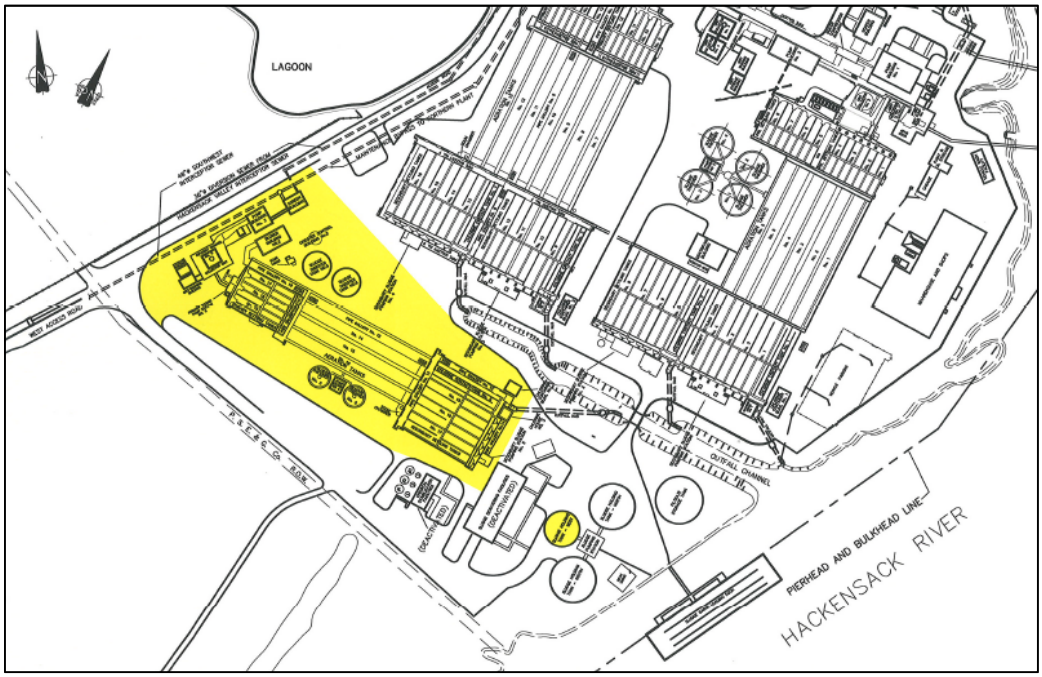


Figure 8-23: Facilities for 29 MGD plant expansion

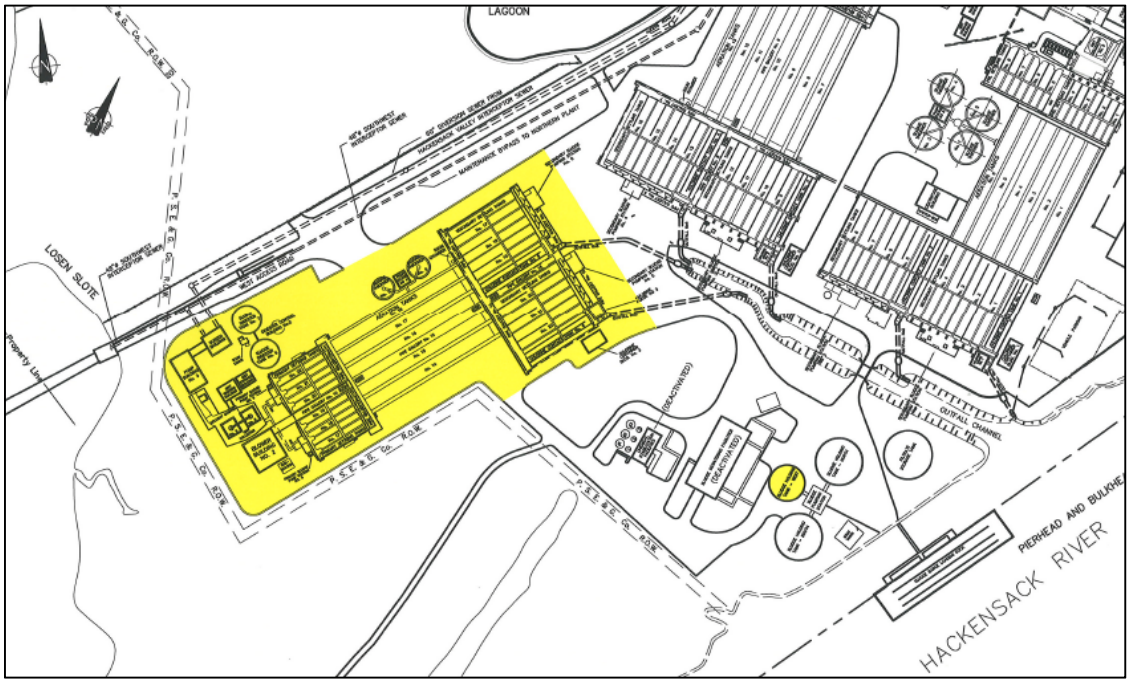


Figure 8-24: Facilities for 58 MGD plant expansion

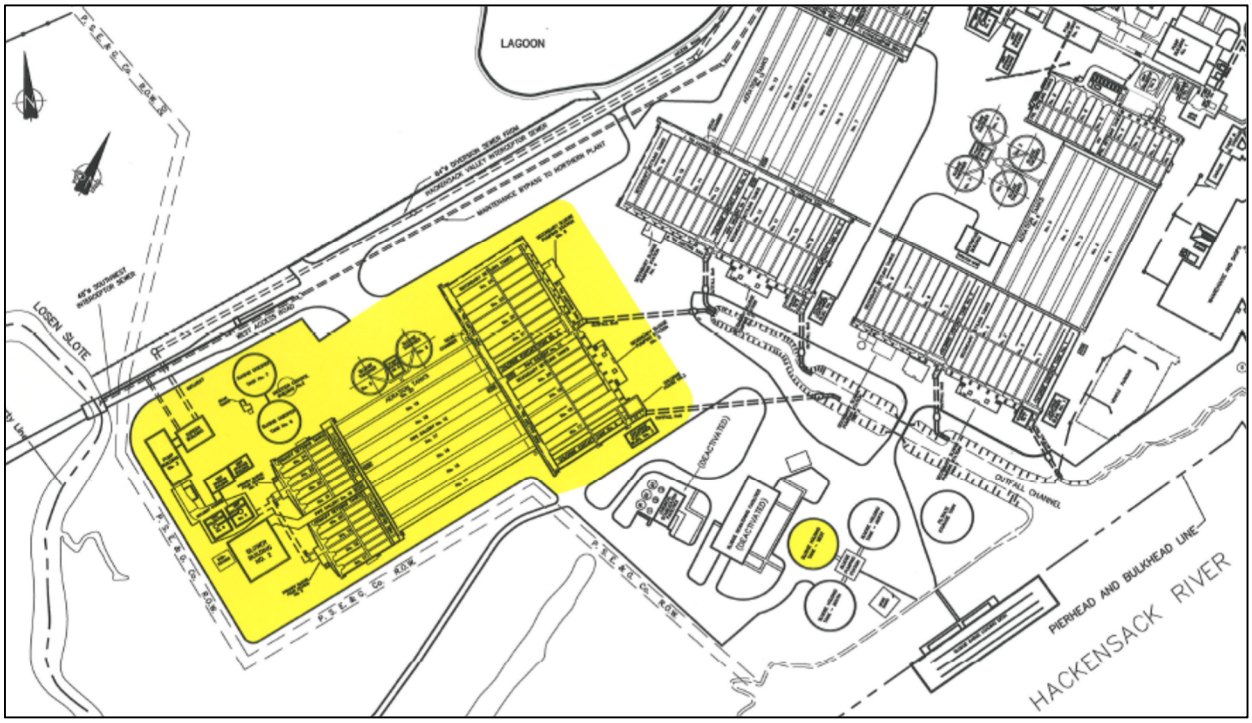


Figure 8-25: Facilities for 86 MGD plant expansion

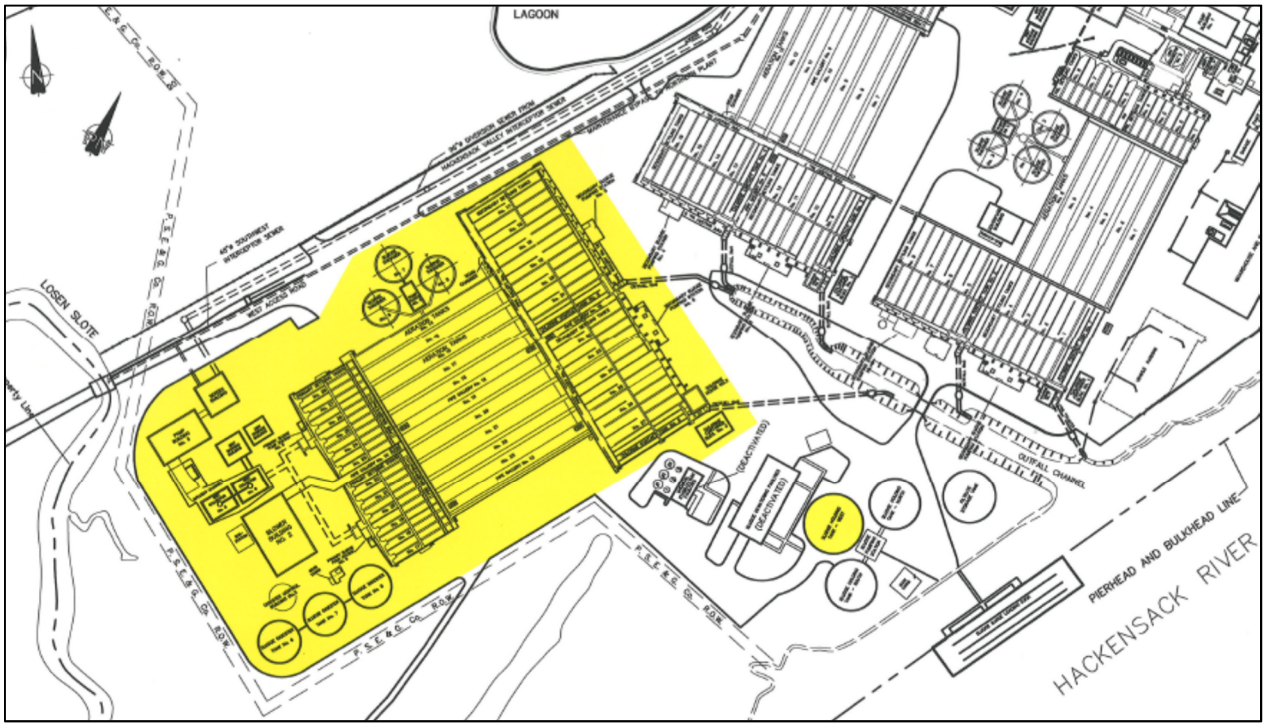


Figure 8-26: Facilities for 115 MGD plant expansion

8.2.1.1 Siting

On a preliminary basis, it appears feasible to site any future plant expansion within the available area. It is noted that one of the parcels required is currently owned by the Town of Little Ferry and this parcel would need to be acquired. The potential site is also encumbered by existing utility easements.

8.2.1.2 Institutional Issue

The future plant expansion, as noted, is on undeveloped land adjacent to several watercourses. It is expected that the site would be environmentally constrained and accordingly, the following permits are anticipated:

- Waterfront Development
- Flood Hazard Area
- Wetlands
- Stormwater Management
- Tidelands
- Local permits

These are common permits and are normally not too difficult to obtain. However, it is noted that given the extents at the site, a wetlands individual permit may be required and the BCUA may be required to provide extensive mitigation.

8.2.1.3 Implementability

Implementing this alternative will be challenging. The treatment facilities are large heavy structures and likely will require deep piles to support their weight in what are likely mucky soils. There are existing utilities easements that may require relocation of large existing utilities including a 138-kV electrical transmission line. The construction would also impact BCUA facilities requiring the relocation of pipelines and temporary facilities to achieve the necessary tie ins.

8.2.1.4 Public Acceptance

The location of the expansion is generally out of the public view and in areas not utilized for recreation. However, the facilities will be visible from the water and there may be objections from recreational boaters. It is likely any significant plant expansion would have high environmental impacts to which public opposition could be expected.

8.2.1.5 Costs

Costs were developed in 2007 for the plant expansion, these costs were indexed to the December 2006 Engineering News Record (ENR) Construction Cost Index (CCI). The costs presented in Table 8-10 were updated and are indexed to the January 2019 ENR CCI (11,206)

Table 8-10: Cost summary for WPCF expansion

Plant Capacity Expansion	Total Plant Wet Weather Capacity	Capital Costs	O&M Costs	O&M Present Worth (20-year)	Total Present Worth (20-year)
29 MGD	149 MGD	$192,000,000	$7,400,000	$113,000,000	$305,000,000
58 MGD	178 MGD	$286,000,000	$11,000,000	$167,000,000	$453,000,000
86 MGD	206 MGD	$373,000,000	$14,400,000	$219,000,000	$592,000,000
115 MGD	235 MGD	$462,000,000	$17,800,000	$271,000,000	$733,000,000

Class 5 Costs (-50%+100%)

8.2.2 Control Program 2 – Wet Weather Blending

As discussed earlier, the analysis of the blending alternative was provided by Arcadis in their plant capacity memo. Two alternatives were explored, the first was to allow the bypass of 90 MGD around the final aeration tanks and final settling tanks to achieve a total blended flow of 210 MGD which is approximately the same as the capacity of the BCUA interceptors. The second was to allow a bypass of 180 MGD to provide a blended flow of 320 MGD which is approximately the hydraulic (not treatment) capacity of the primary settling tanks. Each alternative was investigated for processes to increase the primary treatment capacity. These two alternatives were chemically enhanced primary treatment (CEPT) and ballasted flocculation (BF). These four combinations are depicted in the schematics of the plant unit processes in Figure 8-27 through Figure 8-30.

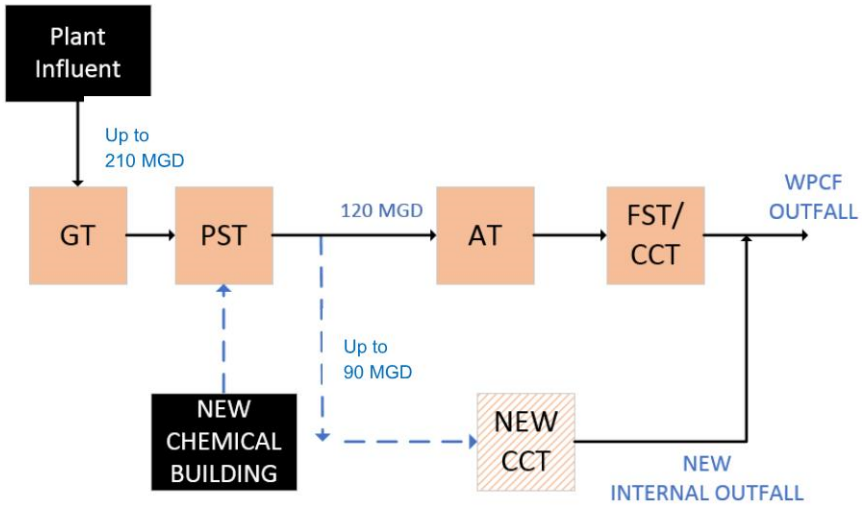


Figure 8-27: BCUA Schematic for total blended flow of 210 MGD using CEPT

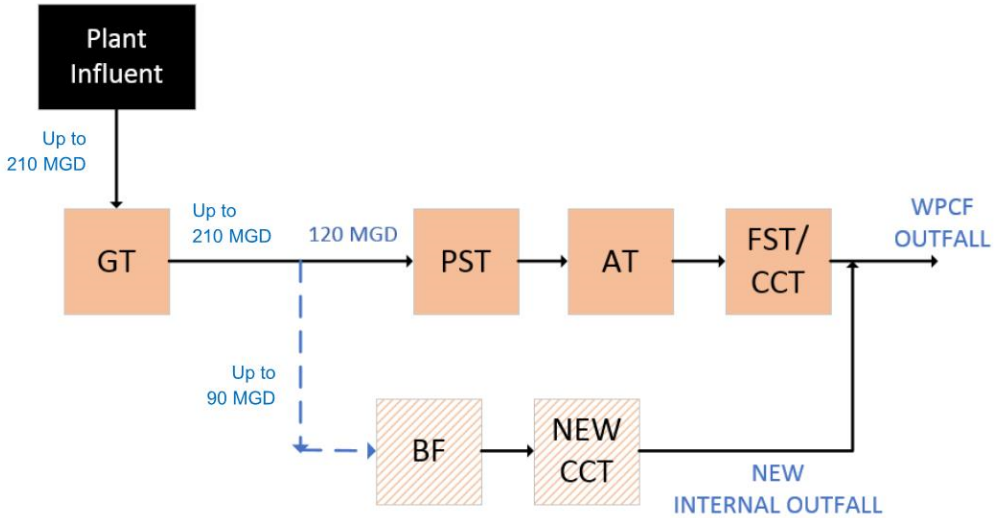


Figure 8-28: BCUA Schematic for total blended flow of 210 MGD using BF

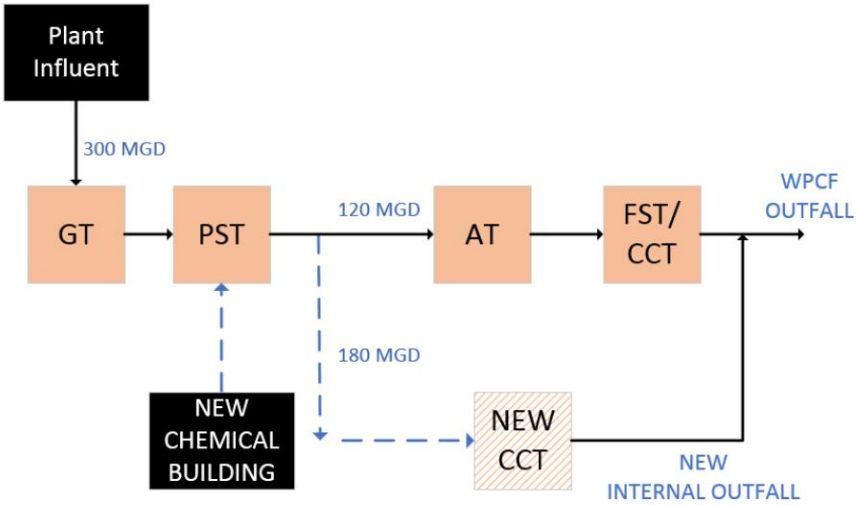


Figure 8-29: BCUA Schematic for total blended flow of 300 MGD using CEPT

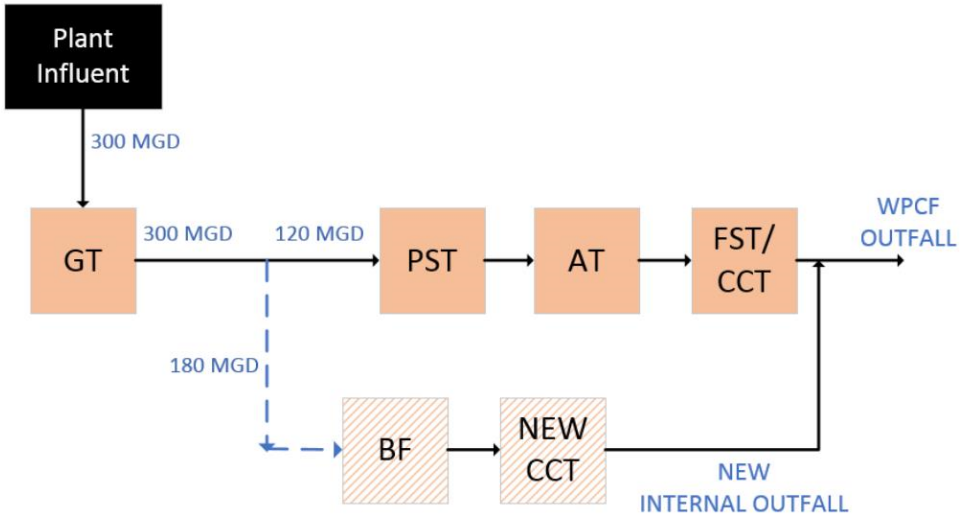


Figure 8-30: BCUA Schematic for total blended flow of 300 MGD using BF

8.2.2.1 Siting

It appears feasible to site plant expansions within the available area. It is noted that one of the parcels required is currently owned by the Town of Little Ferry and this parcel would need to be acquired. Potential locations for the CEPT and BF systems are shown below in Figure 8-31 and Figure 8-32, for full size depictions of CEPT and BF facilities see Section 11. The figures are for 90 MGD systems to bring the total treated to 210 MGD, roughly equivalent to the interceptor capacity.

Figure 8-31: Layout for 210 MGD blended flow using CEPT

Figure 8-32: Layout for 210 MGD blended flow using BF

8.2.2.2 Institutional Issue

The plant expansions are sited on undeveloped land adjacent to several watercourses. It is expected that the site would be environmentally constrained, and accordingly, the following permits are anticipated:

- Waterfront Development
- Flood Hazard Area
- Wetlands
- Stormwater Management
- Tidelands
- Local permits

These are common permits and are normally not too difficult to obtain. However, it is noted that given the extents of the impacts to the site, a wetlands individual permit may be required, and the BCUA may be required to provide extensive mitigation.

8.2.2.3 Implementability

Implementing this alternative will be challenging. The treatment facilities are large heavy structures and likely will require deep piles to support their weight in what are likely mucky soils. There are existing utilities easements that may require relocation of large existing utilities including a 138-kV electrical transmission line. The construction would also impact BCUA facilities requiring the relocation of pipeline and temporary facilities to achieve the necessary tie ins.

8.2.2.4 Public Acceptance

The location of the expansion is generally out of the public view and in areas not utilized for recreation. However, the facilities will be visible from the water and there may be objections from recreational boaters. It is likely any significant plant expansion would have high environmental impacts to which public opposition could be expected.

8.2.2.5 Costs

Costs were developed by Arcadis based on 2018 costs, using the annual average ENR CCI for 2018 of 11,062, these costs were updated to January 2019 (ENR CCI 11,206) to be consist with other costs presented in this report.

Table 8-11: Cost summary for Wet Weather Blending

Blended Flow and Technology	Total Wet Weather Treatment Capacity	Capital Costs	O&M Costs	O&M Present Worth (20-year)	Total Present Worth (20-year)
90 MGD CEPT	210 MGD	$64,500,000	$850,000	$12,900,000	$77,700,000
90 MGD BF	210 MGD	$111,500,000	$1,220,000	$18,600,000	$129,800,000
180 MGD CEPT	300 MGD	$90,200,000	$850,000	$12,900,000	$103,300,000
180 MGD BF	300 MGD	$161,100,000	$1,220,000	$18,600,000	$179,300,000

8.2.3 Control Program 3 – Regional Storage

Rather than increase the treatment capacity at the WPCF to treat the peak inflow, storage could be provided to equalize the flow entering the plant, also referred to as peak shaving. This storage could be provided in the form of a tunnel or tank. Since the goal is to site the additional facilities at the BCUA plant, only site tank(s) will be considered. Typically, but depending on the volume to be stored, tunnel storage is costlier when cost per gallon of storage is considered.

This alternative consists of providing wet weather equalization storage by constructing:

- A diversion from the plant headworks to the new facilities
- New low lift pumps
- Conveyance piping to the tank site
- Storage tanks
- Dewatering pumping station
- Ancillary site improvements and utility relocations

8.2.3.1 Analysis

To evaluate this alternative the 2050 (future) baseline time series data for the Typical Year precipitation was analyzed to determine the volume of flow that would have to be stored, based on the WPCF treatment capacity. Table 8-12 below summarizes the volume required to provide full treatment for the entire flow reaching the WPCF. Note the flow rates and thus the treatment rates were reported at a 5 min time step.

Table 8-12: BCUA WPCF Treatment Rate versus Storage Volume

Treatment Rate [5-min] (MGD)	Required Storage Volume (MG)
120	40
140	7.9
160	0.5
180	0

8.2.3.2 Institutional Issue

The storage tanks are sited on undeveloped land adjacent to several watercourses. It is expected that the site would be environmentally constrained. The following permits are anticipated:

- Waterfront Development
- Flood Hazard Area
- Wetlands
- Stormwater Management
- Tidelands
- Local permits

These are common permits and are normally not too difficult to obtain. However, it is noted that given the extents of the impacts to the site, a wetlands individual permit may be required, and the BCUA may be required to provide extensive mitigation.

8.2.3.3 Implementability

Figure 8-33: WPCF Transport and Treatment Facilities

Implementing this alternative will be challenging. Ideally the site of an equalization type storage tank is near the head end of the WPCF so that hydraulic grades between the wet well and storage unit can be matched, otherwise wastewater flows may need to be pumped. As illustrated in Figure 8-33, the head end of the BCUA Little Ferry WPCF (blue dot) is surrounded on the west side by treatment units and on the east side by water severely limiting the land available for a gravity system. In, addition, the storage facility(s) are large, deep, and heavy structures that will necessitate extensive dewatering facilities during construction, and likely will require deep piles to support their weight in what are likely mucky soils. At the same time, the tanks may need to be ballasted to protect against floatation. There are existing utilities easements that may require relocation of large existing utilities. The construction may also impact BCUA facilities requiring the relocation of pipeline and temporary facilities to achieve the necessary tie ins.

8.2.3.4 Public Acceptance

The location of the expansion is generally out of the public view and in areas not utilized for recreation. However, the facilities will be visible from the water and there may be objections from recreational boaters. It is likely any significant plant expansion would have high environmental impacts to which public opposition could be expected.

8.2.3.5 Costs

Costs were developed using the 2018 Technical Guidance Manual developed by Greely and Hansen for the NJ CSO Group. The manual was dated January 2018 (ENR CCI = 10,878) these costs were updated to January 2019 (ENR CCI 11,206).

Table 8-13: Cost summary for Storage Tanks

Tank Size	Capital Costs	O&M Costs	O&M Present Worth (20-year)	Total Present Worth (20-year)
40 MG	$217,000,000	$3,800,000	$58,000,000	$269,000,000
7.9 MG	$56,000,000	$1,200,000	$18,000,000	$72,000,000
0.5 MG	$9,000,000	$430,000	$6,500,000	$15,500,000

8.2.4 Control Program 4 – Utilize Inline Storage in Interceptor for CSO

During the evaluation of the transport capacity, it was observed that there appears to be available storage within the interceptors. It may be possible to use the storage within the interceptors to store additional combined sewer and then treat the flow at the WPCF as the interceptor drains down. The first step this process was to evaluate the storage volume available. This was accomplished by exporting the pipe dimensions and typical year peak hydraulic grades for the interceptors for the reaches of pipe between the WPCF and the CSO communities. The geometric properties of the pipe were used to determine cross sectional area occupied by sewage at each end of the pipe. These areas were then subtracted from the full pipe cross sectional area to determine the area available for storage. The average end area methodology was applied to determine the volume of storage available by multiplying the average available area for each pipe by the length of the pipe. The individual pipe volumes were then summed to determine a total volume of 6.1 MG, see Table 8-14.

The hydraulic condition within the interceptors is dependent on the tailwater at the BCUA headworks. In the model, the tailwater was fixed at the average of normal operating range which typically varies with a 5-6 foot band. To estimate the impact of a higher water level in the wet well, the storage volumes were calculated when 2 feet was added to the water levels. In this case, the storage volume dropped to 1.3 MG, see Table 8-14.

Table 8-14: Theoretical storage volume in BCUA interceptors

Interceptor	Storage volume at max depth (MG)	Storage volume at max depth +2 ft (MG)
Hackensack Trunk Sewer	2.8	0.2
Overpeck Trunk Sewer	2.5	0.7
Overpeck Relief Sewer	1.8	0.4
Total	6.1	1.3

The BCUA operates its system to maximize conveyance to the treatment plant and the use of inline storage. However, taking advantage of inline storage inherently raises the water level in the interceptor and thus the risk of adverse impacts to basement flooding or surface flooding. While the model can give insight into what is happening within the system, it cannot account for unforeseen conditions that may occur in the “real” world. There is a potential tradeoff between keeping the interceptor level drawn down to provide storage and the impact of pumping additional flow through the plant. As was observed in Control Program 3, under the current flows, storage would be useful to attenuate the peak flows to enhance the treatment provided at the WPCF. Thus, the seeming available storage is already contributing to attenuating flows to the plant and is not available to hold additional CSO. However, this volume would become available if additional treatment capacity was added to the plant.

8.3 Preliminary Selection of Alternative

The BCUA does not own any CSO outfalls, but has agreed to work cooperatively with the municipal permitees, who will be responsible for bearing the costs for any expansion of transport and treatment facilities to accommodate additional combined flow conveyed to and treated by the BCUA. Accordingly, the municipal permittees will need to weigh the costs of CSO controls within the municipality against the costs to convey and treat the flow at the BCUA WPCF. Therefore, the selection of alternatives acceptable to the BCUA lies with the municipal permittees. Notwithstanding this fact, the proceeding evaluation would indicate a preference for certain alternatives over others. The cost of blending is significantly less than full expansion of the treatment plant. Likewise, storage costs are relatively high when compared to blending costs. It is noted BCUA will need to agree to any changes in flow, transport or treatment capacity.

9 Summary

While the members of the BCUA CSO Group have agreed to work cooperatively on the Development and Evaluation of Alternative Control Report. Most of the work for the Selection and Implementation of Alternatives Report will be completed separately and then coordinated and integrated through group meetings into a regional submission through the BCUA. Three different consultants were engaged in the development of Regional Report. The Borough of Fort Lee retained HDR to complete its individual Report, the City of Hackensack retained Arcadis to complete its individual Report, while the Village of Ridgefield Park and BCUA both retained Mott MacDonald to complete their Reports. As part of the Regional Sewer System Characterization Report (June 2018), Mott MacDonald developed a comprehensive model of the entire BCUA District using the InfoWorksICM modeling software covering the forty-four separately sewered municipalities, with special emphasis on the combined sewer systems within the Borough of Fort Lee, the City of Hackensack, and the Village of Ridgefield Park. The models for the combined sewer areas were developed by the respective municipal permittees and imported into the overall BCUA model.

The CSO Long Term Control Plan (LTCP) for the BCUA Service District is due to the NJDEP no later than June 1, 2020. While the Development and Evaluation of Alternatives Report provide preliminary data on the alternatives being considered, it is not the LTCP. Additional investigations and coordination between the members of the BCUA CSO Group will take place over the next eleven months to work towards a Regional LTCP that is acceptable to all members and which complies with the individual NJPDES permits issued to individual members. The information provided herein is a summary of the investigations and cost analyses conducted by each individual member of the BCUA CSO Group. More detailed information is available within the individual reports from each member that has been referenced within this report. Copies of the municipal permitees reports have been appended to this report as Sections 13 through 15. The results will be summarized by each BCUA CSO Group member.

The BCUA undertook and investigation of its transport and treatment facilities to review the alternative controls that potentially could be undertaken within BCUA facilities, by members of the BCUA CSO Group to reduce the frequency and volume of overflows from the permittees with CSO Outfalls. As per previous discussions with members of the BCUA CSO Group the cost of any improvements undertaken to BCUA facilities will need to be funded by those directly benefiting from the project.

Transport Facilities

The performance of the BCUA interceptors is complex and depends on many interrelated factors such as level control on wet well, what the critical points in the system are defined to be, and the distribution of flow entering the interceptor. The theoretical flows as determined through Manning's Equation indicate that the Main Trunk Sewer have a capacity of approximately 120 MGD, and that the joint Overpeck Valley Trunk and Relief Sewers have a combined capacity of approximately 143 MGD. Together these sewers should theoretically be able to transport approximately 265 MGD to the WPCF. Notwithstanding the theoretical calculation, the InfoWorksICM model step rainfall analysis indicates that the maximum flow capacity transferred through the Overpeck Valley Trunk Sewers is controlled by the upstream components of these sewers. Accordingly, the flows introduced downstream must be such that they do not negatively impact the upstream hydraulic grades to an extent that surcharging and potential backups are created upstream. Overall the modeling conducted indicates that approximately 210 MGD can be transported to

Trunk Sewer Description	Max Flow (MGD)
Main Trunk Sewer	128
Overpeck Valley Trunk Sewer	62
Overpeck Valley Relief Sewer	17
Total Flow to WPCF	207

the WPCF safely without hydraulically impacting upstream sewer systems as indicated in the above table. When flows to the WPCF exceed 210 MGD it is likely that upstream in the system surcharging conditions exist.

Treatment Facilities

Arcadis was retained by the BCUA to evaluate the existing design capacity of their WPCF as well as improvements that will be needed to accept additional wet weather flow. Arcadis prepared a memorandum that states: "Based on a hydraulic capacity assessment of the BCUA LF WPCF, as much as 220 MGD can flow through the plant under hydraulic control. If hydraulic control is not necessary, as in a major storm event, as much as 325 MGD can be accepted by the plant, however existing influent trunk capacity is limited to 210 MGD. Based on process modeling and a review of New Jersey and 10 States design standards, the process capacity of the WPCF is estimated at 120 MGD. In order to accommodate flow beyond 120 MGD (up to 210 MGD) during wet weather events, it is proposed that either CEPT or BF be used."

Arcadis determined that improvements to the WPCF would be needed, and that either chemically-enhanced primary treatment (CEPT) or a new ballasted flocculation (BF) unit would be required to increase wet weather flows up to 210 MGD and that the cost of construction and annual operation and maintenance would be as follows:

Option	EPOCC	OPEX
chemically-enhanced primary treatment	$32MM – $127MM	$0.80MM
ballasted flocculation	$55MM - $220MM	$1.20MM

Arcadis did note that blending will require a relaxation of the current permit limits for BOD and TSS during wet weather events since the WPCF's existing limitations would not be consistently met. Wet weather discharge limitations have been allowed and acceptable by the USEPA in other regions of the county. In addition, the above referenced construction and operational costs do not consider, nor include any increased sludge treatment of storage facilities that may be required.

10 References

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http://www.cleanwaternashville.org/content/projects/central-wwtp-capacity-improvements-and-cso-reduction

11 CEPT and BF Plant Modifications

CHLORINE CONTACT TANK
HYPOCHLORITE BUILDING
PUMP STATION
CHEMICAL BUILDING
500 ft
N
Google Earth
© 2018 Google

CHLORINE CONTACT TANK
BALLASTED FLOCCULATION TANK
HYPOCHLORITE BUILDING
PUMP STATION
500 ft
N
Google Earth
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12 NJ CSO Group Updated TGM (Attached)

PVSC retained Greeley and Hansen/CDM Smith to update the 2007 Technical Guidance Manual as originally developed by Mott MacDonald. The updated Technical Guidance Manual developed in January 2018 was provided by PVSC to all members of the NJ CSO Group for use in the development of the CSO Long Term Control Plan so that the costs associated with the LTCP would be consistent. These costs have been updated to 2019 by means of the ENR Construction Cost Index as appropriate for use within this report. The Technical Guidance Manual has been reproduced in its entirety in this section.

Passaic Valley Sewerage Commissioners

CSO Long Term Control Plan

Updated Technical Guidance Manual

January 2018

GREELEY AND HANSEN

CDM Smith

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Appendices

Appendix A – Climber Screens Installation List

Appendix B – ROMAG Installation List

Appendix C – Storm King Vortex Separator Installation List

Appendix D = HYDROVEX FluidSep Vortex Separator Installation List

Appendix E – SanSep Installation List

Appendix F – ACTIFLO Ballasted Flocculation Unit Installation List

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Appendix H – FlexFilter Installation List

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Section 1
Introduction

The combined sewer systems (CSS) in the State of New Jersey are owned by a mix of municipal governments and authorities that are responsible for the State's 210 permitted outfalls. These collection systems are serviced by nine publicly owned treatment works (POTW) wastewater treatment facilities. The New Jersey Department of Environmental Protection has issued NJPDES permits to each of the CSS owners and POTWs requiring that the nine hydraulically connected systems develop and submit a Long Term Control Plan (LTCP) for reducing the impact of combined sewer overflow (CSO) to their receiving waters.

The Passaic Valley Sewerage Commission (PVSC) is one of the nine permitted POTW facilities and is coordinating the LTCP for its eight combined sewer communities: Bayonne, East Newark, Harrison, Jersey City, Kearny, Newark, North Bergen, and Paterson. The North Bergen Municipal Utility Authority also operates one of the nine permitted POTW facilities with its Woodcliff Wastewater Treatment plant, which services parts of North Bergen and Guttenberg. While a separate LTCP will be developed for that system, PVSC and NBMUA have agreed that PVSC would coordinate that LTCP development process as well.

The LTCP development process requires that the permittees each evaluate a variety of CSO control alternatives and submit an Evaluation of Alternatives Report. Although the PVSC and NBMUA hydraulically connected communities will submit system-wide LTCPs, each permittee will be responsible for evaluating the alternatives within their community.

To assist in the communities in performing their alternatives evaluations, PVSC has updated this Technical Guidance Manual (TGM) that was originally developed in 2007.

1.1 Background

In 2004, the NJDEP issued a General Permit (GP) for combined sewer systems that, in part, required combined sewer system owners to initiate the CSO LTCP development process and undergo a Cost and Performance Analysis for Combined Sewer Overflow Point Operation. That analysis required the permittees to evaluate alternatives at each CSO point that would provide continuous disinfection prior to discharge. To assist their communities in performing the analysis, PVSC developed a Technical Guidance Manual that provides an overview of various screening, pretreatment, disinfection, and storage technologies along with guidance on costs. The original TGM was released in 2007.

The New Jersey Pollutant Discharge Elimination System (NJPDES) permits issued in 2015 require the permittees to continue the CSO LTCP development process and perform a complete CSO control alternatives evaluation that will lead to a selected alternative and eventual implementation. While much of the information in the original TGM is still viable, a decade has passed since it was developed. To assist their permittees with the current permit, PVSC has updated the TGM to reflect new information, updated costs, and new permit requirements such as the evaluation of green infrastructure.

1.2 Purpose of the Technical Guidance Manual

The Technical Guidance Manual is intended as a guidance document to assist the individual permittees in performing their LTCP alternatives evaluations. The information and costs provided throughout the document are for planning purposes only, and the individual permittees should verify all of the assumptions and information contained herein.

Section 2

Treatment Technology

Treatment technologies are intended to reduce the pollutant loads to receiving waters by treating wet weather flows prior to discharging to the environment. Specific technologies can address different pollutant constituents, such as settleable solids, floatables, or bacteria. To satisfy CSO treatment objectives, treatment technologies for each unit processes of screenings/ pretreatment/ disinfection alternatives have been evaluated, including the following:

- Screenings - mechanical bar screens, fine screens, band and belt screens, and drum screens.
- Pretreatment - vortex/swirl Separation (Storm King® Vortex Separator, HYDROVEX® Fluidsep Vortex Separator, and SANSEP Process), ballasted flocculation (ACTIFLO® Ballasted Flocculation Process and DensaDeg Ballasted Flocculation), and compressible media filtration (FlexFilter Process)
- Disinfection – chlorination, peracetic acid, ozonation, and, UV disinfection.

CSOs are intermittent in nature and are characterized by highly variable flow rates relative to base sewage flow. Bacterial and organic loadings from the collection system also vary greatly, both within and between storm events. The screenings/pretreatment/disinfection system must be able to handle variable pollutant loadings and large fluctuations in flow that can change drastically. Where treatment facilities are to be considered, provisions for the handling, treatment, and ultimate disposal of sludge and other treatment residuals shall also be included.

2.1 Treatment Technology Evaluation Criteria

In the evaluation of each treatment technology as included in subsequent sections, the following description outlines the process used to evaluate each technology:

1. **Description of Process**: includes a verbal and graphical description of the treatment process and pertinent components.
2. **Applicability**: evaluates the applicability of technology for CSO control. Equipment manufacturers/vendors have been contacted to gather information on installation list for CSO applications, technology evaluation and case study. If determined not applicable for CSO control, no further evaluation will be performed.
3. **Performance**: Each process has been evaluated on a preliminary basis for its performance under similar conditions to CSO, particularly where flow and loading rates varied significantly. Individual processes have a different ability to handle varying loading rates and still maintain a reasonably consistent removal rate, or disinfection rate. The inability to maintain a required level of performance over varying hydraulic loadings may eliminate the process, or require that limitations to its use be considered.

4. **Hydraulics**: The screenings/ pre-treatment/ disinfection alternatives will need to be physically located between the CSO control facility and the receiving waters. In many locations, there may be limited difference in elevation between the water surface level in the regulator and the receiving water level. This will be particularly true wherein the receiving water elevations are affected by tides. Head loss within an individual control process will vary from negligible to as much as 8 feet. The total head loss for a treatment train consisting of screenings, pre-treatment, and disinfection may be as much as 10 feet. For this reason, the evaluation will identify the need for intermediate pumping. Screw pumps, which are capable of efficiently handling large flows under low head conditions, can be utilized for this purpose.

5. **Generation of Waste Streams:** Most if not all screening and pretreatment processes produce waste streams that must be contained and disposed of; however, none of the disinfection processes produce appreciable waste streams. Waste streams for the screening processes consist of the storing and/or disposal of collected screening materials. For the pre-treatment process, the waste streams are more varied. The vortex units produce underflow containing the solids removed by the process, which can be as much as 10% of the design flow of the vortex unit. Ballasted flocculation units produce waste sludge as part of the process. In addition, there is a startup period (approximately 20 minutes) for the ballasted flocculation system during which time the process effluent is of poor quality, and filtration processes produce filter backwash water. When these processes are located at a WWTP or along an interceptor sewer with available capacity, the waste streams can be discharged and treated. However, in remote locations, such as those envisioned for CSO treatment facilities, there is typically no place to dispose of the waste stream. While the permittees that own and operate the CSO conveyance systems will be evaluating the feasibility of increasing wet weather flows to the WWTP, most interceptor sewers during wet weather events are currently at capacity or surcharged. As a result, ancillary tankage must be provided to store the volume of the waste stream produced until such time that it can either be introduced into the process, or discharged to the interceptor sewer for treatment at the WWTP. Where applicable, the need for ancillary tanks must be included in the evaluation of the process.

6. **Complexity**: This portion of the evaluation will identify the level of complexity of the process, whether it is capable of functioning unmanned in a remote setting, and the level of instrumentation that would be needed to operate the system during the overflow events.

7. **Limitations**: Different processes can have limitations on the hydraulic and pollutant loading conditions that it can operate within, which can include both lower and upper limits. Any such limitation must be considered when determining the configuration of unit sizes for that process as needed to handle the variable flow/pollutant loading conditions. Limitations for each process are discussed in subsequent sections and have been considered in development of the evaluation process.

8. **Construction Costs**: This portion of the evaluation will provide preliminary report level construction cost estimates, which includes budgetary equipment costs as provided by the manufacturer, installation costs, building costs, and contingency for design flow ranging from 10 MGD to 450 MGD.

9. **Operation and Maintenance Costs**: Information on the operation and routine maintenance requirements was obtained from each of the equipment manufacturers and included in this section. Annual operation costs have been prepared based on power requirements for operation of the equipment, the estimated cost of power, and the estimated annual hours of operation of the equipment. In addition, annual maintenance costs reflecting those recommended by the equipment manufacturer, as well as the manpower required for anticipated post-overflow event clean up and service has been included.

10. **Space Requirements**: Due to the proximity of the regulators to the receiving water body, in most cases it is unlikely that there will be sufficient existing open land available to construct the screenings/pre-treatment/disinfection facilities. Therefore, it will likely be necessary for the Permittee to purchase land. The evaluation of the respective process shall include an evaluation of the space needed for the process. This area is not limited to the process or tank area but includes a small buffer for roadways and access base.

In the process of preparing this TGM, technology users were contacted to gather information on their experience with using the technology for CSO treatment.

2.1.1 Bayonne Wet Weather Demonstration Project

The Bayonne Wet Weather Flow Treatment and Disinfection Demonstration Project (Bayonne MUA Pilot Study) was conducted over a two-year period at the Oak Street facility in Bayonne, NJ which receives the CSO from Bayonne City. The project was sponsored by the Bayonne Municipal Utilities Authority (BMUA), with grants and collaboration from New Jersey Department of Environmental Protection (NJDEP) and the United States Environmental Protection Agency (USEPA). The primary focus of the Bayonne MUA Pilot Study was to verify the performance of selected technologies to treat CSO discharges for solids removal and disinfection under field conditions as suitable for remote satellite locations.

The treatment technologies evaluated included high rate solids removal (i.e., vortex and plate settler units) and enhanced high rate solids treatment (i.e., a compressed media filter). Three types of disinfection units were also included, namely chemical disinfection (i.e., Peracetic acid, PAA), and ultraviolet (UV) disinfection (low and medium pressure units). The evaluation results of the pilot study are discussed in the corresponding sections of the TGM.

2.2 Screenings

Screening technologies can either represent minimal treatment of a CSO before disinfection or can be used to remove larger particles upstream of vortex/swirl separation, ballasted flocculation, or compressed media filtration before high rate disinfection processes. The screening technologies and their related clearances, reviewed for this Technical Guidance Manual, are as follows:

- Mechanical Bar Screens 0.25" to 2" (6-50 mm) bar spacing
- Fine Screens 0.125" to 0.5" (3-13 mm) bar spacing
- Band and Belt Screens 0.08" to 0.4" (2-10 mm) openings
- Drum screens 0.0004" (0.01 mm) openings

As indicated above, screening technology will remove large material or particles as small as 0.0004" from the waste stream. The choice of a particular screening technology is a function of the general purpose of the screen, and what additional treatment process or equipment lies downstream. Screens with smaller openings, such as belt and micro screens, typically require pretreatment with a mechanical bar screen to prevent damage from large objects. Screenings equipment which are not continuously cleaned, such as manually cleaned bar screens, were eliminated from this evaluation due to the potential for backup and surcharging of the collection system. In general, screening systems are very effective in removing floatable and visible solids, but do not remove a significant amount of TSS, fecal coliform, enterococci, BOD, COD, NH3, TKN, total phosphorous, and total nitrogen.

The following sections describe the types of screens and equipment, as well as its capability to remove the various pollutants of concern. At the end of the section a summary of performance, operation, and environmental impacts will be presented. Based upon this summary some of the screening technologies will be eliminated from further consideration.

2.2.1 Mechanical Bar Screens

Description of Equipment

The three most common types of mechanically cleaned bar screens are: (1) chain driven, (2) climber type rake, and (3) catenary. Chain driven mechanical raking systems consist of a series of bar rakes connected to chains on each side of the bar rack. During the cleaning cycle, the rakes travel continuously from the bottom to the top of the bar rack, removing material retained on the bars and discharging them at the top of the rack. A disadvantage of chain-driven systems is that the lower bearings and sprockets are submerged in the flow and are susceptible to blockage and damage from grit and other materials. Climber-type systems employ a single rake mechanism mounted on a gear driven rack and pinion system. The gear drive turns cog wheels that move along a pin rack mounted on each side of the bar rack. During the cleaning cycle, the rake mechanism travels up and down the bar rack to remove materials retained on the bars. Screenings are typically discharged from the bars at the top of the rack. This type of bar screen has no submerged bearings or sprockets and is less susceptible to blockages, damage and corrosion. Catenary systems also employ chain drive rake mechanisms, but all sprockets, bearings, and shafts are located above the flow level in the screen channel. This in turn reduces the potential for damage and corrosion and facilitates routine maintenance. During the cleaning cycle, the rakes travel continuously from the bottom to the top of the bar rack to remove materials retained on the bars. Screenings are typically discharged from the bars at the top of the rack. The cleaning rake is held against the bars by the weight of its chains, allowing the rake to be pulled over large objects that are lodged in the bars and that might otherwise jam the rake mechanism.

Bar screens will remove essentially 100% of all rigid objects of which the minimum dimension is more than the spacing between the bars. Removing screenings from CSOs essentially does not remove any dissolved solids, or nutrients such as TKN, total nitrogen and total phosphorous. Screenings removed from overflows can however contain some larger rigid materials that reflect a BOD loading. Solids, such as fecal material, can also be contained within screenings collected on the bar screen, however the velocity between the bars increases with increasing flow, thus this material can be broken up and pass through the bars. Therefore, it is difficult to quantify on a consistent basis any BOD loading, fecal coliform and enterococci count, and TSS concentrations removed by

the screening technologies. Nevertheless, some removal estimates, as provided by the manufacturer, have been included within the analysis procedure for further consideration.

For the purposes of the Technical Guidance Manual, the mechanical bar screen evaluation is based on the use of Climber Screens® since these have been found to be more reliable and significantly lower in operation and maintenance requirements than others. Figure 2-1 shows photos of typical climber screens. The Technical Guidance Manual analysis is based on mechanical bar screens with a maximum velocity between the bars of 4.5 feet per second (fps) and a peak velocity of approach of 3.0 fps. These are the standard criteria for designing bar screens for use in wastewater treatment plants, where flow is continuous and the diurnal patterns more predictable. Since CSOs are intermittent, with widely varying flow rates, these standards are more likely to be violated for short periods of time. The mechanical bar screen selections are also based upon an anticipated head loss of less than one foot, a peak flow level of six feet under peak flow conditions, with an operating floor located twelve feet above the water surface. For CSO applications where heavy debris loadings are likely, the minimum bar spacing should be approximately 1 inch.

Figure 2-1 - Photos of Typical Climber Screens

(Source: Infilco Degremont, Inc.)

Applicability to The Project

Mechanical bar screens have proven to be a relatively simple and inexpensive means of removing floatables and visible solids. They are typically the screen of choice in treatment facilities, and are used at a many CSO treatment facilities. There have been hundreds of Climber Screens® installed in CSO applications across the US. A list is provided in Appendix A focused on Type IIS and IIIAS installations in NJ, NY, and PA since 2000.

Performance Under Similar Conditions

As stated above, mechanical bar screens are already installed in many CSO facilities and operate successfully to remove floatables and visible solids over the fluctuations in flow rates seen in CSOs. Slight removal of TSS, total phosphorous, and total nitrogen (typically 5%, 3%, and 2%, respectively) can be achieved with the solids removal.

Hydraulics

Hydraulic losses through bar screens are a function of approach velocity, and the velocity through the bars. The head loss across the bar screen increases as the bar screen becomes clogged, or blinded. Instrumentation provided with mechanically cleaned screens is typically configured to send a signal to the cleaning mechanism so the head loss across the screen is limited to 6 inches.

Generation of Waste Streams

As screenings are removed from the CSO flows they generate a waste stream for disposal. Studies have found that the average CSO screenings loads vary from approximately 0.5 to 11 cubic feet per million gallons, with peaking factors based upon hourly flows ranging from 2:1 to greater than 20:1. These screenings must be either transferred to the interceptor sewer for ultimate disposal at the WWTP, or removed and stored in a container for onsite removal at a convenient time. The collection of screenings can be performed using conveyors, screenings compactors, or pumps. Any enclosure around the screenings equipment should provide space for a container and odor control.

Complexity

Mechanical bar screens are able to function intermittently, at remote locations with a minimum level of instrumentation. A level detector is needed to determine when a CSO is occurring and to activate the screen. Differential head sensors located upstream and downstream of the screen will detect head loss and initiate a cleaning cycle. During periods where there are no overflows, a timer can be utilized to periodically exercise the screen, so it is ready for use.

Limitations

When mechanical bar screens are installed in a WWTP, the flows vary within an anticipated range which is predetermined so the screens can be sized for the necessary peak flows, and redundant units can be provided. In CSO installations there are wide variations in flow rates that can pass through the screens, but the high flow rates are usually of short duration. Due to the intermittent nature of CSOs, it is not considered cost effective, nor necessary to provide redundancy. Nevertheless, providing multiple units in separate channels is a means of handling equipment out of service. The quickness with which CSO flows can increase however can lead to problems in getting units in other channels into operation quickly enough given the operating speeds of motor operated sluice gates. A review of the pollutant removal rates as reported by the manufacturer indicates that only about 5% of the TSS is removed by the screen. While screening of solids may be adequate for the lower treatment objects (50%, 85%, and 95% removals) where TSS levels are not as critical, the literature does not indicate that screening alone will remove adequate solids to provide for consistent and reliable disinfection at higher treatment objectives.

Construction Costs

Table 2-1 presents the preliminary planning level construction cost estimates of Climber Screens® for design flows ranging from 10 MGD to approximately 450 MGD. It includes equipment cost,

installation cost, general contractor (GC) field general conditions, GC overhead & profit (OH&P), and contingency. This cost estimates assume that the Climber Screens® will be installed in existing CSO channels. If the existing CSO channel does not provide adequate channel width to maintain velocities below 3 fps, a new or modified chamber will be required at an additional cost. The installation cost is assumed at 50% of the equipment cost based on the complexity of the installation. Budgetary equipment pricing information for Climber Screens® was gathered from equipment manufacturer Suez, formerly Infilco Degremont, Inc. The estimated total construction costs for the Climber Screens® are plotted against flowrates from 10 MGD to approximately 450 MGD in Figure 2-2.

Climber Screens® pricing is primarily determined by channel size which is dictated by the flow and plant specific parameters or design. Therefore, the Type IIS is suitable for channels up to 7'-0" wide. Pricing provided by the manufacturer is based on assumed channel dimensions of 5'-0" wide by 10'-6" deep. A single unit of this model of Climber Screen® would be suitable for up to 50 MGD or larger depending on channel dimensions. The Type IIIAS is suitable-for channels 6'-6" to 12'-0" wide. The pricing provided by the manufacturer is accurate up to the 8'-0" wide and 10'-6" deep dimensions. For the large 450MGD flow, multiple units each designed for a peak flow of 112 MGD are recommended. Capacity can be adjusted based on channel dimensions, bar rack clear spacing, and number of units desired.

Operation and Maintenance

Costs associated with operation include the electrical cost for operating the motor(s) on the mechanical bar screens. Regular maintenance requires visits to the site after each storm to inspect the screens for damage, remove any large material in the channels, clean up any screenings on the floor or equipment, and general wash down of the area. Regular maintenance also includes routine lubrication and maintenance of the tracks, racks, drives, and gear boxes. It is important to keep the pin racks and carriage bearings greased and oiled. It is also important to inspect the bearings for excessive wear. The Type IIS and IIIAS carriage assemblies utilize self-greasing/oiling canisters which are easily replaced at the recommended intervals. The follower shaft bearings and carriage drive bearings are replaced utilizing access points built into the side frames (i.e. carriage does not need to be removed). It is recommended to perform periodic visual inspections to ensure proper operation, lubrication and bearing wear.

Estimated annual operation costs for the Climber Screen® are presented on Table 2-2 containing factors for calculation of operating costs; while estimated annual maintenance labor cost including cost factors are included on Table 2-3.

Space Requirements

The space required for mechanical bar screens consists of the building and area on the exterior of the building for access to remove the screenings container.

Case Study

New York City utilized TypeIIIAS Climber Screens® at their Manhattan and Bronx Grit Chambers from 1986 until 2016. These chambers deliver combined sewage to the Wards Island WWTP, which has a total plant flow of approximately 500 MGD. After the first 6 years of using the Climber Screens®, the shaft bearings were beyond their useable life. Although initially designed for 5HP per

motor based on the average weight of debris, it was later found that 7.5 HP was required to handle the harsher conditions imposed by the combined sewage.

Table 2-1 - Preliminary Construction Cost Estimates for Climber Screens

Flow Range	System	Width x Depth	Budgetary Equipment Price	Install Cost(1)	GC General Conditions (2)	GC OH&P(3)	Contingency(4)	Total
10 MGD to 50 MGD	(1) Type IIS	5'-0" x 10'-6"	$305,000	$152,500	$45,750	$45,750	$274,500	$823,500
50 MGD to 112 MGD	(1) Type IIIAS	8'-0" x 10'-6"	$465,000	$232,500	$69,750	$69,750	$418,500	$1,255,500
112 MGD to 224 MGD	(2) Type IIIAS	8'-0" x 10'-6"	$465,000	$232,500	$69,750	$69,750	$418,500	$1,255,500
224 MGD to 336 MGD	(3) Type IIIAS	8'-0" x 10'-6"	$1,900,000	$950,000	$285,000	$285,000	$1,710,000	$5,130,000
336 MGD to 448 MGD	(4) Type IIIAS	8'-0" x 10'-6"	$1,900,000	$950,000	$285,000	$285,000	$1,710,000	$5,130,000

Notes:
(1) Installation cost is assumed at 50% of the equipment cost.
(2) GC general conditions are estimated at 10% of the total direct cost.
(3) GC OH&P are estimated at 10% of the total direct cost.
(4) 50% of Contingency is used for the planning level of cost estimates.

Figure 2-2 - Total Estimated Construction Cost of Climber Screens

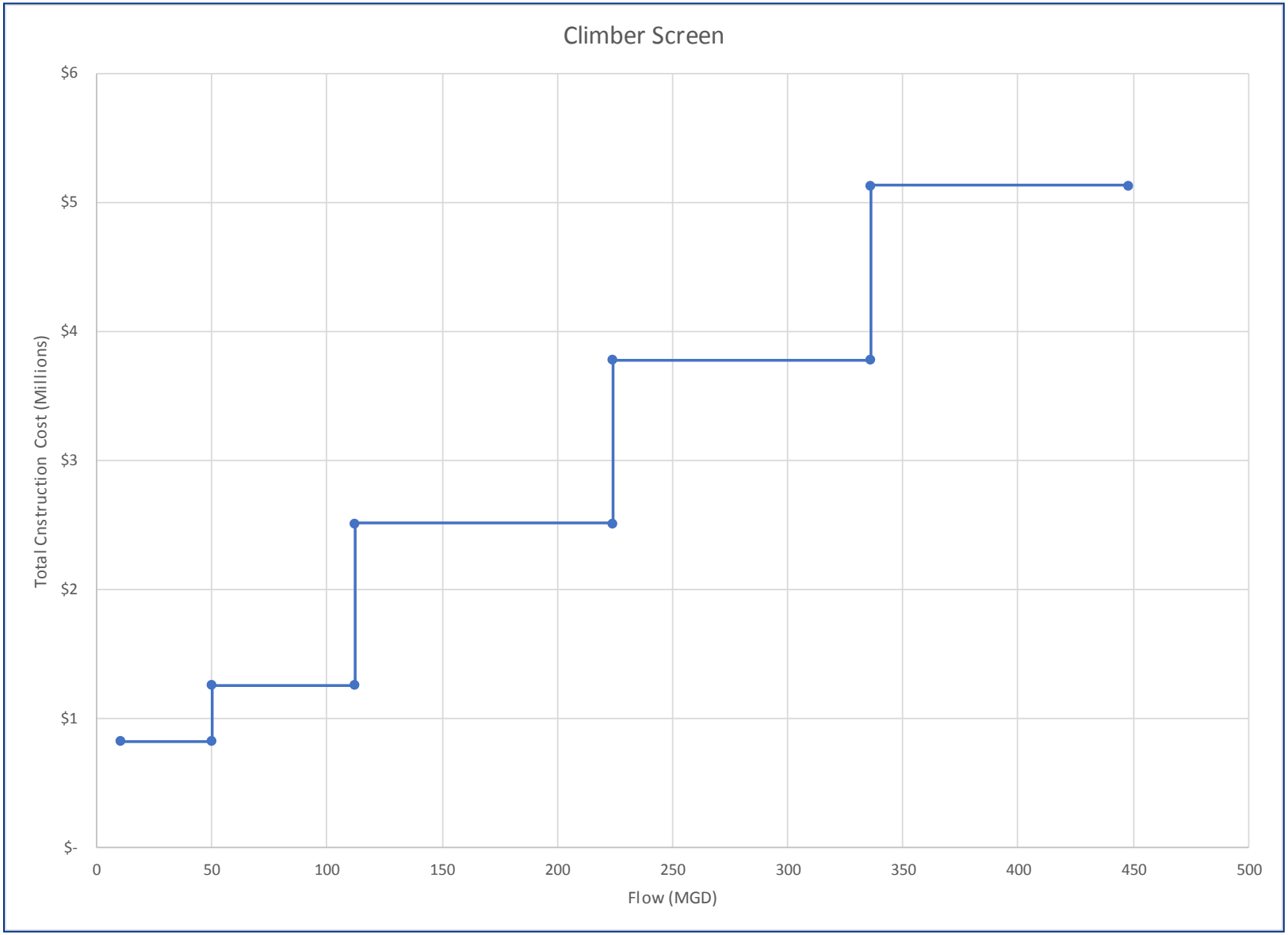


Table 2-2 - Annual Operation Costs of Climber Screens

Flow Range	System	Total Horsepower (HP)	Total Power (kW)(1)	Annual Energy Usage (kW-hr)(2)	Annual Cost(3)
10 MGD to 50 MGD	(1) Type IIS	3	2	1,119	$157
50 MGD to 112 MGD	(1) Type IIIAS	5	4	1,864	$261
112 MGD to 224 MGD	(2) Type IIIAS	10	7	3,729	$522
224 MGD to 336 MGD	(3) Type IIIAS	15	11	5,593	$783
336 MGD to 448 MGD	(4) Type IIIAS	20	15	7,457	$1,044

Notes:
(1) HP x 0.7457
(2) Assumes 500 hours of annual operation
(3) Assumes energy costs of $0.14/kW-hr

Table 2-3 - Annual Maintenance Labor Costs of Climber Screens

Maintenance Frequency	Parts	Description	Estimated Man-Hours	Annual Cost(1)(2)
Monthly	Cam Tracks and Pin Racks	Grease and inspection	0.5	$900
Bi-annually	Automatic Lubricators	Grease	0.5	$150
Annually	Automatic Lubricators	Oil	0.5	$75
2-3 years	Carriage Drive Shaft Bearing	Replace	1	$75
3-5 years	Follower Shaft Bearing	Inspect - replace as necessary	2	$100
5 years	Gear Box	Change fluid	2	$60
After Each CSO Event	Screens	Inspection and cleanup	2	$30,000
Total Annual Maintenance Labor Cost				$31,360

Notes:
(1) Assumes 100 events per year
(2) Assumes labor rate of $150/hour

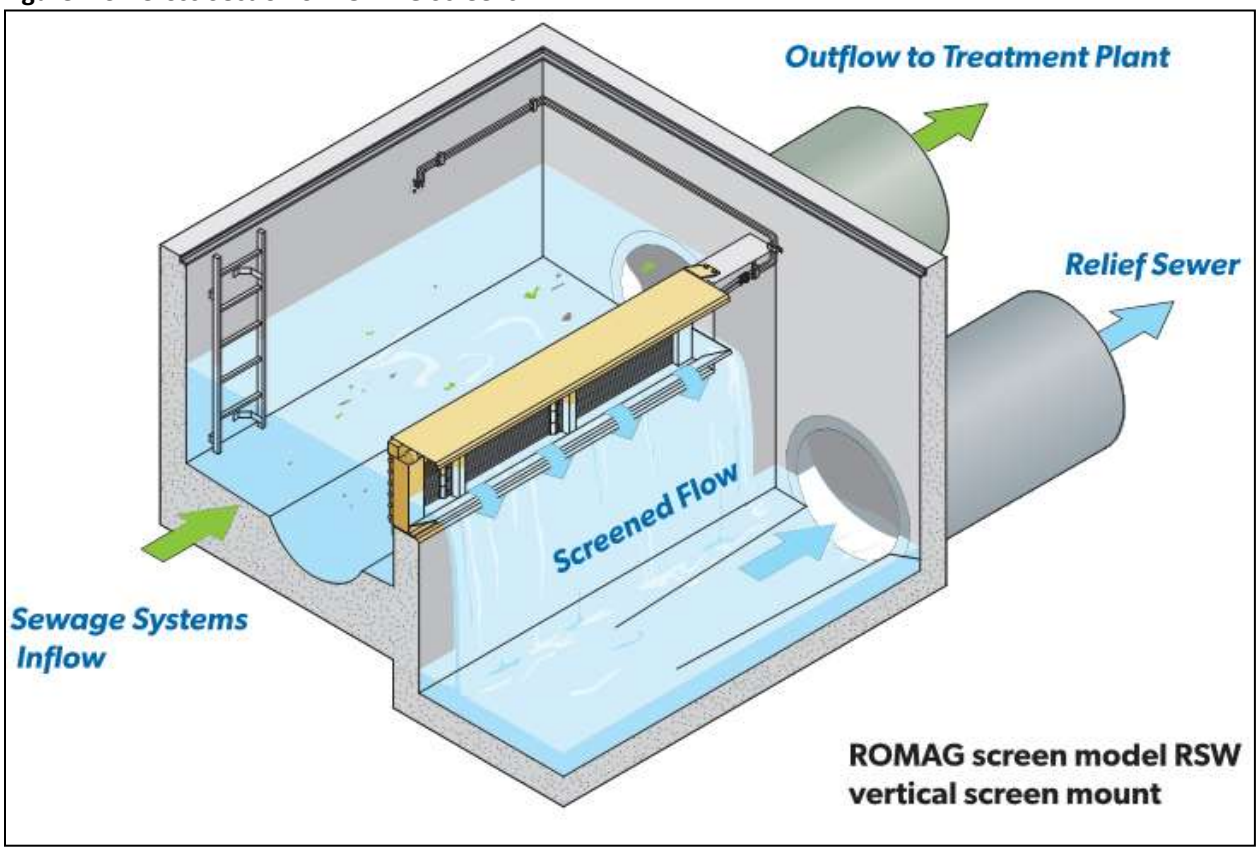
2.2.2 Fine Screens

Description of Process

These screens have openings ranging from 1/8" to 1/2", and will capture suspended and floatable material with smaller dimensions. The equipment evaluated under this category of screenings technology includes ROMAG™ Screens as manufactured by WesTech Engineering, Inc.

The ROMAG™ Screens consist of parallel bars similar to a bar screen, with spacing varying from 0.16" to 0.47". The screens are cleaned by combs, which extend through the rack and are attached to a hydraulically driven mechanism on the downstream side of the screen. The hydraulic unit is located above grade in an enclosure. The material collected on the upstream side of the screen is cleaned off the face of the screen by the combs and kept in the flow in the interceptor. They are not removed or collected, but continue toward the wastewater treatment plant for removal. As the flow increases beyond the capacity of the screens, the upstream water surface rises and overflows a baffle that is part of the screen assembly, discharging directly to the outfall. All the fine screens of this category are located such that the solids are retained on one side of the screen and transported to the interceptor or other facility for ultimate disposal. **Figure 2-3** shows the cross section of vertical mount ROMAG™ Screens.

Figure 2-3 - Cross Section of ROMAG Screens



(Source: WesTech Engineering, Inc.)

Applicability to the Project

Fine screens have proven to be a relatively simple and inexpensive means of removing floatables and visible solids where the overflow is controlled by a weir. They are typically constructed in the regulator, sometimes requiring modifications to the regulator, such as moving the weirs, and extending the weir lengths. The required screening capabilities for the maximum flow rate would need to be provided, since flows exceeding the capacities of the screens will continue to overflow unscreened. See Appendix B for a list of installation of ROMAG™ Screens for CSO application.

Performance Under Similar Conditions

As stated above, fine screens are typically installed in CSO regulators and operate successfully to remove floatables and visible solids over the fluctuations in flow rates seen in CSOs. Slight removal of TSS, total phosphorous, and total nitrogen (typically 10%, 8%, and 5%, respectively) can be achieved with the solids removal.

Hydraulics

The typical head loss reported through the unit is 4 inches, while additional freeboard from the maximum flow through the screens to the baffle height is typically 2 inches. The total head loss through the screen is typically about 6 inches at the design flow.

Flows exceeding the capacity of the screens would overflow the baffle and by-pass the screen. Usually additional weir length is needed so that the existing upstream water surface elevations are maintained after the screen is installed

Generation of Waste Streams

Fine screens are located in the regulator with flow passing up and through the screen, overflowing the weir and going out the outfall. Since the flow direction is up through the screen, the screened material is kept on the interceptor side of the screen, and remains in the interceptor when the cleaning mechanism cleans the face of the screen. Since the screenings remain in the interceptor, there is no collection at the screen and therefore no waste stream. Nevertheless, the limitation is that there be adequate flow and solids transport within the interceptor sewer system. The additional screening material that remains in the interceptor will find its way to any downstream regulators, and eventually to the WWTP.

Complexity

Fine screens can function intermittently, at remote locations with the minimum of instrumentation. A level detector is needed to determine when a CSO is occurring and to activate the screen. Differential head sensors located upstream and downstream of the screen will detect head loss and initiate a cleaning cycle. During periods where there are no overflows, a timer can be utilized to periodically exercise the screen, so it is ready for use.

Limitations

Fine screens would need to be installed on regulators with side overflow weirs. Other types of regulators would require the construction of a weir, at which point the use of a mechanical bar screen may be preferable. Also, any regulators where the fine screens would be installed would need to be accessible for routine inspection and maintenance of the screens. A review of the pollutant removal rates as reported by the manufacturer indicates that only about 10% of the TSS is removed by the screen. While screening of solids may be adequate for the lower treatment

objectives (50%, 85%, and 95% removals) where TSS levels are not as critical, the literature does not indicate that screening alone will remove adequate solids to provide for consistent and reliable disinfection at higher treatment objectives. The higher TSS removal rates of fine screens versus mechanical bar screens (10% vs 5% respectively) may result in TSS levels acceptable for disinfection at lower treatment objectives.

Construction Costs

The preliminary planning level construction cost estimates are provided in Table 2-4 for ROMAG™ Screens of design flow ranging from 10 MGD to 450 MGD. It includes equipment cost, installation costs, GC field general conditions, GC OH&P, and contingency. This cost estimates assume that the ROMAG™ Screens will be installed in existing regulators. The costs for modifying a side overflow regulator to accommodate the installation of the screen is included in the installation cost. If the existing regulator cannot be modified to accommodate the ROMAG Screen and side overflow, a new and larger regulating chamber will be required at an additional cost. The installation cost is assumed at 50% of the equipment cost based on the complexity of the installation. Budgetary equipment pricing information for ROMAG™ Screen was gathered from equipment manufacturer WesTech Engineering, Inc. Based on vendor provided information, the largest individual screen can potentially handle up to 100 MGD, and in the case of higher demand multiple screens would be applied side by side. Velocities should be restricted to 5 ft/s. The equipment cost includes the controls, hydraulic power pack and everything needed to operate.

The estimated total construction costs for the ROMAG™ Screens are plotted against flowrate from 10 MGD to 450 MGD in

Figure 2-4.

Operation and Maintenance Costs

The operating costs include the electrical cost for operating the hydraulic power pack and an in-tank (hydraulic fluid) heater (700W-120V). The hydraulic pack operates the cleaning comb action across the screen. Each single ROMAG™ Screen has a hydraulic power pack that consists of a 5HP motor to drive the hydraulic pump. An 1HP in-tank heater for each screen is used to keep the hydraulic fluid at right temperature. Routine maintenance of the ROMAG™ Screens includes visits to the site after each storm to inspect the screens for damage, remove any large material in the channels, and cleanup of any screenings on the floor or equipment, and general wash-down of the area. Routine maintenance also includes the monthly maintenance of the screen such as replacing combs, repairing leaks in the hydraulic lines, maintaining the oil level in the hydraulic drive, and cleaning any level sensors, etc.

Estimated annual operation costs for the ROMAGTM Screens are presented on Table 2-5 containing factors for calculation of operating costs; while estimated annual maintenance labor cost including cost factors are included on Table 2-6.

Table 2-6Space Requirements

Since the fine screens would be installed in the regulators, which would probably be located in the street or existing easement, it is anticipated that there would be no additional space requirements for the fine screens.

Case Studies

Chattanooga, Tennessee utilizes ROMAG™ Screens at their downtown CSO treatment facility. Two RSW 8x7 screens were installed in 2000 and are still in use treating approximately 180 MGD. The maintenance of the screens was reported as minimum, and the automatic cleaning function had been working well with the exception of one instance where the screens became stuck.

The City of Binghamton, NY, has been using CSO screens for floatable control at four CSO locations since 2003. According to conversations with the site supervisor, the screens have been trouble-free. Both sides of the screens can be observed without entering the channel, and weekly inspection takes approximately 5 minutes. Typically, operators hose down the screens to remove residual debris after a storm event. Binghamton operators check the tension of the bars annually, and change hydraulic oil and filters per the Operations and Maintenance manual. No parts have required replacement to date.

Chattanooga, Tennessee utilizes ROMAG™ Screens at their downtown CSO treatment facility. Two RSW 8x7 screens were installed in 2000 and are still in use treating approximately 180 MGD. The maintenance of the screens was reported as minimum, and the automatic cleaning function had been working well with the exception of one instance where the screens became stuck.

Table 2-4 - Preliminary Construction Cost Estimates for ROMAG Screens

Flow	System	Length x Depth	Budgetary Equipment Price	Install Cost(1)	GC General Conditions(2)	GC OH&P(3)	Contingency(4)	Total
10 MGD	(1) Model RSW 4x3/4	9'-10" x 1'-9"	$252,000	$126,000	$37,800	$37,800	$226,800	$680,400
25 MGD	(1) Model RSW 7x4/4	13'-2" x 2'-8"	$305,000	$152,500	$45,750	$45,750	$274,500	$823,500
50 MGD	(1) Model RSW 12x4/4	13'-2" x 4'-3"	$393,000	$196,500	$58,950	$58,950	$353,700	$1,061,100
75 MGD	(1) Model RSW 14x5/4	16'-5" x 4'-11"	$450,000	$225,000	$67,500	$67,500	$405,000	$1,215,000
100 MGD	(1) Model RSW 14x6/4	19'-8" x 5'-1"	$475,000	$237,500	$71,250	$71,250	$427,500	$1,282,500
450 MGD	(6) Model RSW 14x5/4	98'-5" x 4'-11"	$2,700,000	$1,350,000	$405,000	$405,000	$2,430,000	$7,290,000

Notes:

Note:
(1) Installation cost is assumed at 50% of the equipment cost.
(2) GC general conditions are estimated at 10% of the total direct cost.
(3) GC OH&P are estimated at 10% of the total direct cost.
(4) 50% of Contingency is used for the planning level of cost estimates.

Figure 2-4 - Total Estimated Construction Cost of ROMAG Screens

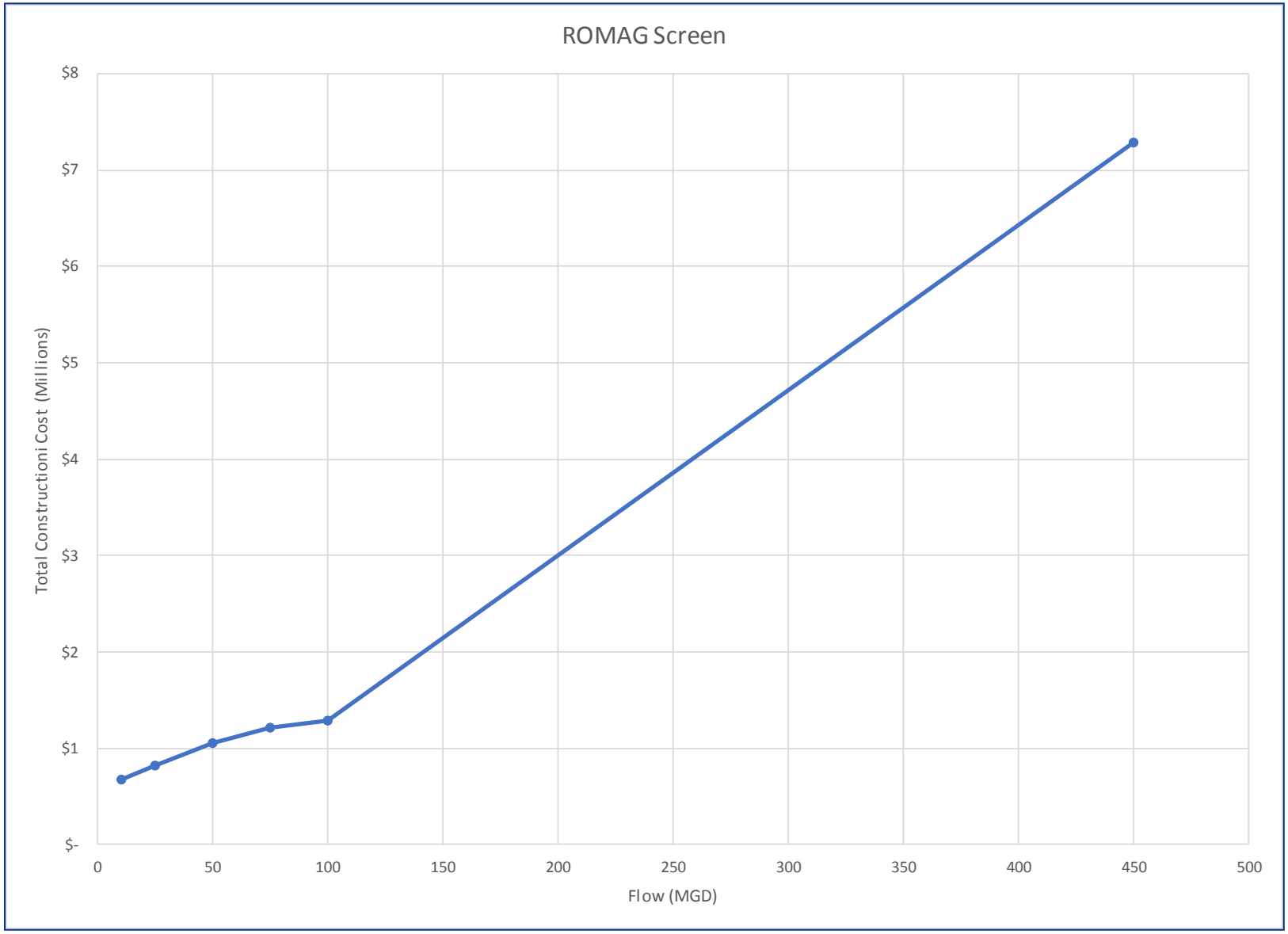


Table 2-5 - Annual Operation Costs of ROMAG Screens

Flow	System	Total Horsepower (HP)	Total Power (kW)(1)	Annual Energy Usage (kW-hr)(2)	Annual Cost(3)
10 MGD	(1) Model RSW 4x3/4	6	4	2,237	$313
25 MGD	(1) Model RSW 7x4/4	6	4	2,237	$313
50 MGD	(1) Model RSW 12x4/4	6	4	2,237	$313
75 MGD	(1) Model RSW 14x5/4	6	4	2,237	$313
100 MGD	(1) Model RSW 14x6/4	6	4	2,237	$313
450 MGD	(6) Model RSW 14x5/4	30	22	11,186	$1,566

Notes:
(1) HP x 0.7457
(2) Assumes 500 hours of annual operation
(3) Assumes energy costs of $0.14/kW-hr

Table 2-6 - Annual Maintenance Labor Costs of ROMAG Screens

Maintenance Frequency	Parts	Description	Estimated Man-Hours	Annual Cost(1)(2)
Every 100 Operational Hours	Fasteners	Check for tightness	0.5	$375
Monthly	Screen bars	Check for clogging	0.5	$900
Monthly	Cleaning carriage	Check for proper operation	0.25	$450
Monthly	Piston rod locking nut	Check for tightness	0.25	$450
Monthly	Power pack oil level	Check for proper level and Check lines and piston rod for major fluid loss	0.5	$900
Monthly	Oil filter	Replace filter if necessary	0.25	$450
Annually	Screen Bars	Confirm tension with torque wrench	0.5	$75
Annually	Oil Temperature Probe	Check for proper operation and send sample to oil supplier; replace if required	0.5	$75
Annually	Motor	Lubricate	0.5	$75
After Each CSO Event	General Visual Inspection	Check for proper operation	1	$15,000
Total Annual Maintenance Cost				$18,750

Notes:
(1) Assumes 100 events per year
(2) Assumes labor rate of $150/hour

2.2.3 Band and Belt Screens

Description of Process

The common characteristic of these screens is that they contain stainless steel perforated elements forming a continuous band traveling either parallel or perpendicular to the flow stream. In the case where the band is parallel to the channel, flow enters the center of the screen, turns 90 degrees and passes through the sieve elements, exiting through the sides of the unit. Where the band is perpendicular to the channel flow passes through the screen, with the screened flow continuing down the channel.

Figure 2-5 shows a photo of Finescreen Monster, manufactured by JWC Environmental. These screens utilize either stainless steel, or UHMW sheets with perforations between 0.08" to 0.4" mm in diameter.

Figure 2-5 - Photo of Finescreen Monster

(Source: JWC Environmental)

Applicability for the Project

These screens are typically used for polishing wastewater treatment flows. Their perforated panels are very prone to clogging from fibrous materials and are not easily cleaned. To protect these screens from larger objects that could damage or clog them, the manufacturers recommend installing ¾ inch screens upstream of them. However, that ¾ inch screen upstream of the belt and band screen would have the same pollutant removal efficiency and thus the belt and band screen would be ineffective. Accordingly, it does not appear to be practical to utilize these types of screens in a CSO application. There currently are no known installations on CSO discharges.

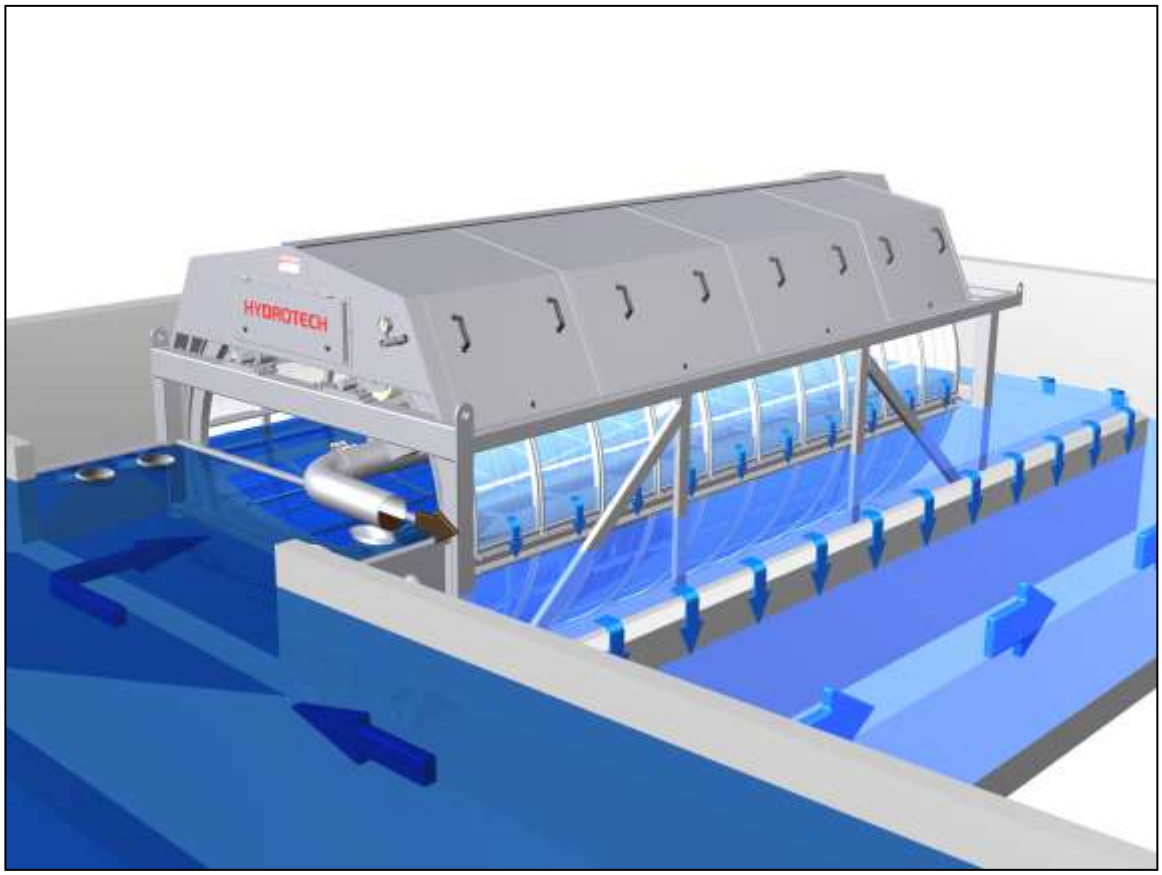
These screens are not considered applicable for CSO treatment and not further evaluated.

2.2.4 Drum Screens

Description of Process

A drum screen is a fine filter with openings from 10 to 1000 microns. The filter cloth is made of acid proof steel or polyester. Three, four, or five filter elements are placed in sections over a rotating drum, depending upon the drum diameter. The drum rotates in a tank. The liquid is filtered through the periphery of the slowly rotating drum. Assisted by the filter elements special cell structure, the particles are carefully separated from the liquid. Separated solids are rinsed off the filter cloth into the solids collection tray and discharged. The operation of the drum can be continuous or automatically controlled. The unit evaluated for this application was the HydroTech Drumfilter by Veolia Water Technologies. Figure 2-6 shows a cross section HydroTech Drumfilter.

Figure 2-6 - Cross Section of HydroTech Drumfilter



(Source: Veolia Water Technologies)

Applicability for the Project

Drum filters are currently used as a polishing unit at WWTPs. The disc media is polyethylene and the size openings are 10 microns for wastewater. The hydraulic loading for drum filters is 50 to 100 gpm/ft^2, based upon an influent TSS concentration of 20 mg/L. The manufacturer expects an influent TSS concentration of 10 to 100 mg/L upstream of the unit. Accordingly, significant TSS removal equipment would be needed upstream of the screen. There currently are no known installations on CSO discharges.

These screens are not considered applicable for CSO treatment and not further evaluated.

2.2.5 Evaluation of Screening Technology

The above sections evaluated each of the screening processes considered for pretreatment of CSO flow relative to criteria on cost, performance, limitations, and ancillary facilities. Each process was rated from 1 to 5, with 5 being the most effective, for approximately twenty different items and totaled. While somewhat subjective, this method does provide a mechanism for comparing each screening unit in relationship to each category and subcategory. The results of the evaluation are illustrated on Table 2-7.

Based upon the evaluation results in Table 2-7, fine screens received the highest results followed by mechanical bar screens, band and belt screens, and drum screen. requirements, which is reflected in their rating. Fine screens and mechanical bar screens should be considered as part of this TGM. Drum screens and band and belt screens were not considered applicable, and did not undergo further consideration.

Table 2-7 - Evaluation of Screening Technology

Criteria	Mechanical Bar Screens	Fine Screens	Band and Belt Screens	Drum Screens
Applicability	5	5	1	1
Performance				
TSS	1	3	4	4
Solids and Floatables	1	2	4	4
Hydraulics	4	4	1	1
Waste streams	3	5	1	1
Complexity	5	5	1	1
Limitations	2	2	1	1
Construction Cost	4	2	1	1
Operations	4	4	1	1
Maintenance	4	3	1	1
Space Requirements	3	2	1	1
Total	31	32	16	16

2.3 Pretreatment Technology

Pretreatment technology is used to remove floatable and total suspended solids (TSS) prior to high rate disinfection in CSO applications. The pretreatment technology evaluated for the TGM includes vortex/swirl separation technology, ballasted flocculation, and compressed media filtration.

The choice of a pretreatment technology is a function of construction costs, space requirements, and type of disinfection treatment process downstream. In general, pretreatment is very effective in removing floatable and TSS. It can also remove certain amount of fecal coliform, enterococci, BOD, COD, NH3, TKN, total phosphorous, and total nitrogen, which is attached to the TSS.

The following sections describe the types of pretreatment technology, as well as its capability to remove the various pollutants of concern. At the end of the section a summary of performance, operation, and environmental impacts will be presented.

2.3.1 Vortex/Swirl Separation Technology

Vortex/swirl separation technology utilizes naturally occurring forces to remove solids and floatable material. Flow enters a circular tank tangentially causing the contents to rotate slowly about the vertical axis. The flow spirals down the perimeter allowing the solids to settle out. This process is aided by rotary forces, shear forces, and drag forces at the boundary layer on the wall and base of the vessel. The internal components direct the main flow away from the perimeter and back up the middle of the vessel as a broad spiraling column, rotating at a slower velocity than the outer downward flow. Per manufacturer claims, by the time the flow reaches the top of the vessel it is virtually free of settleable solids and is discharged to the outlet channel. The collected solids are then discharged by gravity or pumped out from the base of the unit to the interceptor sewer or auxiliary storage tank if interceptor capacity is not available.

Conventional vortex separators such as Storm King®, manufactured by Hydro International, and the HYDROVEX® FluidSep manufactured by John Meunier were reviewed for this Technical Guidance Manual. A variation of the typical vortex/swirl separation process - the SanSep equipment from PWTech is evaluated as well.

The following provides a discussion of each of the above referenced unit processes, as well as its reported capability to remove the various pollutants of concern. A summary of performance, operation, and limitations or constraints, is provided at the end of this section.

2.3.1.1 Storm King® Vortex Separator

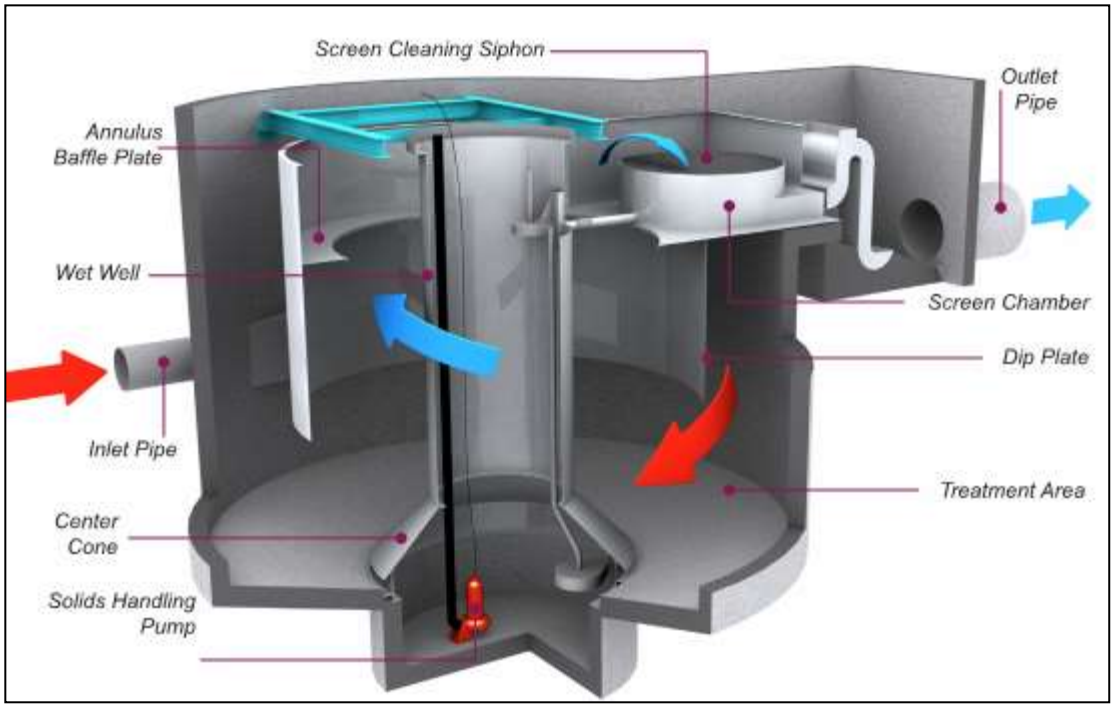
Description of Process

Flow is introduced tangentially into the side of the Storm King®, causing the contents to rotate slowly about the vertical axis. The flow spirals down the perimeter allowing the solids to settle out. This process is aided by rotary forces, shear forces, and drag forces at the boundary layer on the wall and base of the vessel. The internal component directs the main flow away from the perimeter and back up the middle of the vessel as a broad spiraling column, rotating at a slower velocity than the outer downward flow. A dip plate locates the shear zone, the interface between the outer downward circulation and the inner upward circulation, where a marked difference in velocity encourages further solids separation. Settled solids are directed to the helical channel located under the center cone and are conveyed out of the main chamber through the underflow outlet. The

flow passes down through the Swirl Cleanse screen which captures floatables and neutrally buoyant material greater than 4mm in diameter. The air regulated siphon provides an effective backwash mechanism to prevent the screen from blinding. Screened effluent is discharged into a receiving watercourse, a storage facility, or continues on to receive further treatment. The collected solids are then discharged by gravity or pumped out from the base of the unit to the sanitary sewer.

Typical design loading rates are from 7 to 44 gpm/sf. This loading rate is based on the flow coming in and the horizontal surface area of the circular vortex unit. Cross section of a Storm King® Vortex Separator in full operation is provided in Figure 2-7.

Figure 2-7 - Cross Section of Storm King Vortex Separator



(Source: Hydro International)

Applicability to the Project

Based on manufacturer publications, Storm King® units have been used for floatables control, primary treatment equivalency of CSOs and wet weather induced flows. The first installation of Storm King® units for CSO application was in mid-1995 in Hartford CT. See Appendix C for a list of Storm King® installation in the US for CSO application.

The units have been installed in remote locations, away from treatment plants and reportedly performed well. There are no moving parts within the vortex unit itself. Underflow from the unit can be discharged by gravity to sewers or continuously pumped to an ancillary tank where it would be stored until there is capacity in the interceptor sewer system. Underflows from the unit run approximately 10% of the design flow and thus the volume from the underflow can be significant.

Performance

The Storm King® vortex separator is most effective in removing heavier settleable solids, floatable material, and inorganic solids. The performance information provided by the manufacturer

indicates that the percent removal of TSS, BOD and COD drops off as the hydraulic loading rate increases. TSS removal ranges from 35-50%, and BOD removal is typically 15-25%. Vortex units achieve removal by two means: the consolidation of solids material; and flow separation, which is accomplished by the underflow removal. When the vortex unit operates under low hydraulic loading rates, and there is a significant amount of settleable solids, both removal mechanisms are operating. As the hydraulic loading rate increases, or the settleable solids concentration decreases, there is less consolidation and the vortex unit functions more as a flow separator. At the highest hydraulic loading rates recommended, the unit functions strictly as a flow separator. The vortex units, the Storm King included, usually have an underflow that is 10% of the design capacity of the unit. So even under the worst conditions, when there is no consolidation of solids taking place, they would theoretically remove 10% of the pollutants. While this would hold true for the soluble portion of pollutants, in the case where the pollutant was associated with fine particles, the removal would be less. The reason for this decrease is that since fine particles weigh less, more of these particles would be carried out in the effluent especially at higher hydraulic loading rates. Some of the removals associated with these units are for lower volume storms when the volume associated with the unit acts as a storage system.

In the Bayonne MUA Pilot Study, the Storm King® units experienced operating issues due to their screens clogging with materials that appeared to be primarily toilet paper. Performance issues of less than 10% TSS removals were experienced when Volatile Suspended Solids (VSS) accounted for a high percent of the influent TSS. The TSS removal efficiencies improved when evaluating the inorganic component of TSS, or Fixed Suspended Solids (FSS). The FSS removal efficiencies for Storm King® units averaged around 17%, with the maximum removal efficiencies of 45.2%. The low removal of VSS (or inorganic) fraction of TSS indicated that the Storm King® units will be ineffective on their own with UV disinfection due to low ultraviolet light transmittance of the effluent.

Hydraulics

Vortex units are hydraulically efficient. The head loss through the unit consists of the losses through the inlet to the unit, and the head loss over the effluent weir. The losses in the lower hydraulic loading rates will be limited to less than six inches. At higher hydraulic loading rates, the losses will increase significantly, possibly up to a couple of feet, unless diverted upstream.

Generation of Waste Streams

As discussed under the description of the process and the performance: 10% of the design flow must continuously be removed as underflow. In many cases this flow will need to be pumped from the vortex unit due to the depth of the underflow pipe. While permittees with conveyance facilities must evaluate means of increasing conveyance to the WWTP, it is doubtful that the underflow can be consistently and constantly transported to the interceptor. In locations where interceptor capacity is not available during the overflow, the underflow must be stored in ancillary tanks. The capacity of these ancillary tanks is based upon the underflow flow rate and the duration of the overflow event. Once the event is over the contents of the storage tank can be pumped back into the interceptor. Floatable material captured in the tank is removed at the end of the overflow event as the tank is emptied, and is also sent back into the interceptor.

Complexity

The vortex/swirl separator is a simple process, especially since there are no moving parts within the unit. Removals are achieved using natural forces and no adjustment of equipment is necessary. The only controls that are needed are in the flow coming to the unit to ensure that the unit operates within its hydraulic loading rates. This can be accomplished using sluice gates or overflow weirs. The other area requiring instrumentation would be the control of the underflow sump where underflow is pumped out. The control of the pumping units would be by floats, bubblers, or ultrasonic level sensors.

Limitations

As previously indicated, the hydraulic loading rate is key to the performance of the vortex/swirl separator. Therefore, the limitation to this process occurs for the more stringent treatment objectives. Since a required and consistent effluent TSS must be achieved for the disinfection process to be effective, the variations in flows, particularly above the required hydraulic loading rate, result in a reduced removal of TSS and a corresponding decrease in the efficiency of the disinfection process. If the excess flows are by-passed around the vortex unit, going directly to disinfection, as required by the NJPDES requirement for complete disinfection, the higher TSS concentrations will again result in decreased disinfection efficiency. This represents a limitation on the process for the higher treatment objectives.

Construction Costs

Budgetary equipment pricing information for Storm King® vortex separator was obtained from equipment manufacturer Hydro International, Inc. Table 2-8 presents preliminary planning level construction cost estimates for flows ranging from 10 MGD to 450 MGD. It includes equipment cost, concrete cost associated with the construction of the tank containing the vortex structure, cost for ancillary tank for underflow storage, installation costs, GC field general conditions, GC OH&P, and contingency. Budgetary equipment pricing provided by the equipment manufacturer Hydro International includes only the fabricated stainless-steel vortex structures inside. Cost for outside concrete tank enclosure were estimated based on the sizes of the vortex units. Construction costs for excavation, sitework, soil support, and dewatering, as well as the underflow wet well and the pumps are included in the installation costs. The estimated total construction costs for the Storm King® Vortex Separator are plotted against flowrate from 10 MGD to 450 MGD in Figure 2-8.

Operation and Maintenance

The operating costs for the Storm King® vortex separator are associated with the power of the underflow pump. The horsepower of the pumps required increases as the size of the vortex separator, and corresponding underflow, increases. Regular maintenance required for the Storm King® unit includes inspection of the vortex separator after each rainfall event, replacement of the underflow pumps every 6 months for overhaul and sharpening of the cutter blades, and vacuuming out the floatable material that will accumulate in the underflow wet well.

Estimated annual operation costs for the Storm King® vortex separator are presented on Table 2-9 containing factors for calculation of operating costs; while estimated annual maintenance labor cost including cost factors are included on Table 2-10.

Space Requirements

The space requirements of the Storm King® vortex separator shall be based upon a square area utilizing the diameter of the tank and a buffer of 5 feet on each side.

Case Studies

According to literature obtained from Hydro International, Bucksport, ME, has been using Storm King® since 2008 as a solution to CSO related flooding caused by the nearby Penobscot River. The installation of satellite treatment within the collection system saved the city from expanding the capacity of their wastewater treatment plant. Solids which settle out from the Storm King® are fed via gravity from the base of the unit to the sewage treatment plant. Additionally, the system is used as a chlorine contact and mixing chamber for the reduction of fecal coliforms before effluent is discharged into the Penobscot River. Since the system was commissioned, all rain events the system has handled have been treated in accordance with regulatory requirements

The 18' (5.5 m) diameter Storm King® system was constructed in a park and is housed within a building which may resemble a restaurant. Residents are impressed with the installation. Bucksport has designed the facility such that a Swirl-Cleanse screening component may be added in the future which will allow capture of all floatables and neutrally buoyant material greater than 4 millimeters in diameter.

According to literature obtained from Hydro International, Saco, ME, has been using a 22-ft diameter Storm King® since November 2006. Sedimentation and screening are followed by disinfection using sodium hypochlorite (NaClO) in the flow tank. A Swirl-Cleanse screen is installed in this system which captures all floatables and neutrally buoyant material greater than 4 millimeters in diameter. Influent Total Suspended Solids (TSS) levels are in the range of 300 mg/L. Treated effluent TSS is typically 60mg/L or lower. Treated effluent is discharged directly into the Saco River, while the collected screenings and settleable solids are pumped back to the wastewater treatment plant for processing.

Engineers who worked on the Saco Sewer Project have been impressed with the performance of the Storm King® even in storms much larger than the set design criteria. The system requires maintenance crews to perform a quick wash down the tank after a storm. Additional maintenance is minimal.

Table 2-8- Preliminary Construction Cost Estimates for Storm King Vortex Separator

Flow	System	Diameter	Budgetary Equipment Price	Concrete Structure Cost	Auxiliary Tank Cost(1)	Install Cost(2)	GC General Conditions (3)	GC OH&P(4)	Contingency(5)	Total
10 MGD	(1) StormKing 10 MGD	28’	$739,000	$82,000	$871,200	$1,269,150	$296,135	$296,135	$1,776,810	$5,330,430
25 MGD	(1) StormKing 25 MGD	38’	$1,403,000	$181,000	$1,573,000	$2,367,750	$552,475	$552,475	$3,314,850	$9,944,550
50 MGD	(2) StormKing 25 MGD	38’	$2,797,000	$291,500	$2,300,000	$4,041,375	$942,988	$942,988	$5,657,925	$16,973,775
75 MGD	(2) StormKing 37 MGD	42’	$3,831,000	$291,500	$3,040,000	$5,371,875	$1,253,438	$1,253,438	$7,520,625	$22,561,875
100 MGD	(3) StormKing 35 MGD	42’	$5,733,000	$359,000	$3,720,000	$7,359,000	$1,717,100	$1,717,100	$10,302,600	$30,907,800
450 MGD	(10) StormKing 45 MGD	44’	$23,463,000	$718,000	$10,890,000	$26,303,250	$6,137,425	$6,137,425	$36,824,550	$110,473,650

Notes:
(1) Auxiliary Tank costs derived from quotation from Mid Atlantic Storage System on Aquastore Glass Fused to Steel Storage Tank of 150,000 gal
(2) Installation cost is assumed at 75% of the equipment cost.
(3) GC general conditions are estimated at 10% of the total direct cost.
(4) GC OH&P are estimated at 10% of the total direct cost.
(5) 50% of Contingency is used for the planning level of cost estimates.

Figure 2-8 - Total Estimated Construction Cost of Storm King Vortex Separator

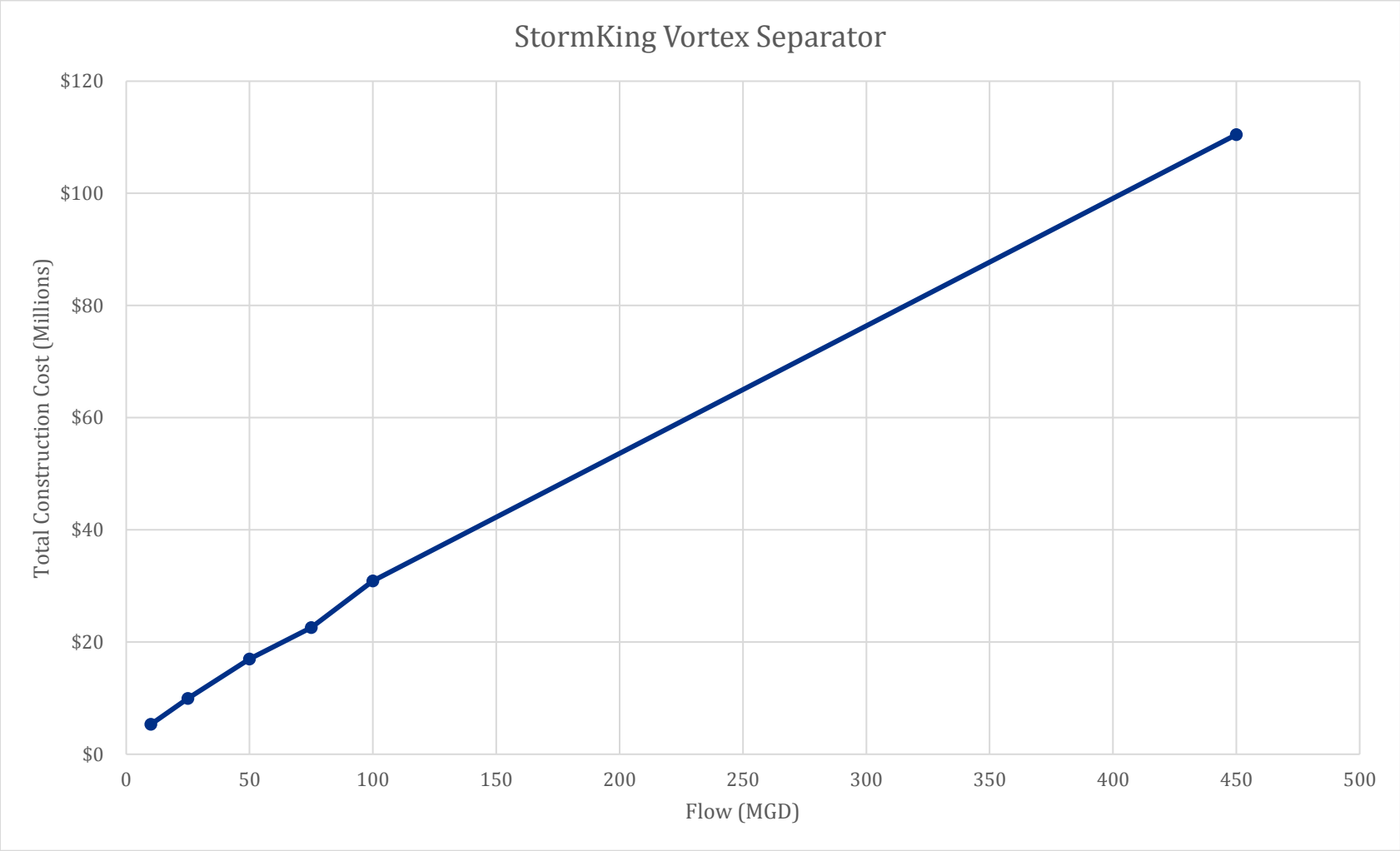


Table 2-9 - Annual Operation Costs of Storm King Vortex Separator

Flow	System	Total Horsepower (HP)	Total Power (kW)(1)	Annual Energy Usage (kW-hr)(2)	Annual Cost(3)
10 MGD	(1) StormKing 10 MGD	14	10	1	$731
25 MGD	(1) StormKing 25 MGD	35	26	4	$1,827
50 MGD	(2) StormKing 25 MGD	70	52	7	$3,654
75 MGD	(2) StormKing 37 MGD	104	78	11	$5,429
100 MGD	(3) StormKing 35 MGD	139	104	15	$7,256
450 MGD	(10) StormKing 45 MGD	625	466	65	$32,624

Notes:
(1) HP x 0.7457
(2) Assumes 500 hours of annual operation
(3) Assumes energy costs of $0.14/kW-hr

Table 2-10 - Annual Maintenance Labor Costs of Storm King Vortex Separator

Maintenance Frequency	Parts	Description	Estimated Man-Hours	Annual Cost(1)
Biannually	Valve inlet and outlet	Visual check and removal of coarse debris	1	300
Biannually	Underflow pumps	Visual check	1	300
Every three years	Underflow pumps	Replacement of underflow pumps	8	400
Total Annual Maintenance Cost				$1,000

Notes:

(1) Assumes labor rate of $150/hour

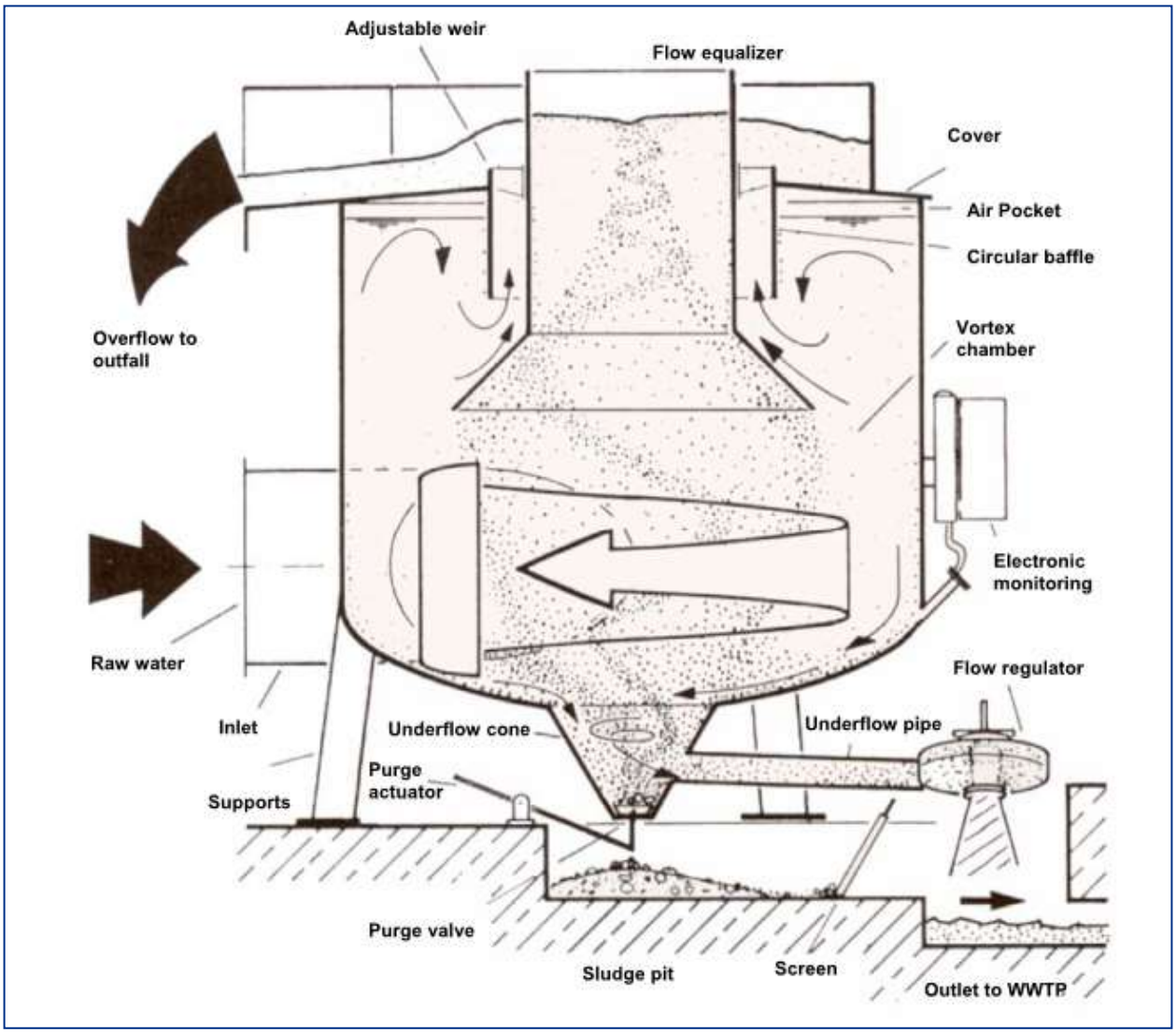
2.3.1.2 HYDROVEX® FluidSep Vortex Separator

Description of Process

In CSO installations, the dry weather flow that enters the HYDROVEX® FluidSep Vortex Separator passes by freely on the sloped bottom towards the central cone of evacuation and then through a flow regulator. During a storm event, the incoming flow becomes greater than the regulated outflow. This will effectively start the filling of the vortex separator. Many minor events can be fully intercepted and contained inside the vortex separator volume without actual overflow. For more intense or more durable storm events, the HYDROVEX® FluidSep Vortex Separator starts overflowing through its central annular overflow weir. This weir is made of two plunging cylindrical treatment baffles providing a double crown arrangement. The overflow water is evacuated through the ring-shaped opening formed by these two treatment baffles. The overflow is fixed in the circular opening of the top cover of the vortex separator structure. The overflowed water falls from the weir on the upper chamber of the separator and is then evacuated, either towards an additional treatment system or directly to the outfall. Due to its tangential inlet port, the incoming water brings the mass of retained water into a rotational movement inside the tank. The resulting flow pattern is non-turbulent and very favorable to the separation of suspended solids. These particles can readily settle and are furthermore pulled by the centrifugal currents towards the wall of the separator. Once the particles are caught on the limit layer along the walls, they fall to the structure bottom and are finally brought to the unit's evacuation cone. From there, they are carried out with the underflow water through the regulator. When the HYDROVEX® FluidSep Vortex Separator is filled, an air pocket is formed under the unit's cover, imprisoned by the baffle partition arrangement. The floatables entering the separator will be caught there and will simply circulate around until the unit progressively gets back to dry time flow conditions. The lower surface of the cover always remains free of water, due to the captured air pocket.

The proper selection of the HYDROVEX® FluidSep implies that the unit operating size is efficient for all flows up to the design flow. When flows higher than the design flow are received, the unit will operate at a lesser efficiency level. The collected solids are then discharged by gravity or pumped out from the base of the unit to the sanitary sewer. Loading rates vary from 3 gpm/sf to 21 gpm/sf. Cross section of a HYDROVEX® FluidSep Vortex Separator in full operation is shown in Figure 2-9.

Figure 2-9 - Cross Section of a HYDROVEX® FluidSep Vortex Separator



(Source: John Meunier, Inc.)

Applicability

The HYDROVEX® FluidSep Vortex Separator was developed in 1985 by a German firm, Umwelt-und Fluid-Technik (UFT) as a tool in the treatment of CSO and stormwater. The first HYDROVEX® Fluidsep unit was installed in 1987 in the City of Tengen near Schaffhausen in Germany. The units are still operating successfully. A special research program that ended in the summer of 1990 supplied evidence of CSO treatment efficiency of the HYDROVEX® FluidSep (H. Brombach, *et al.*, 1993). The program was based on the qualitative evaluation of sampling campaigns performed at the installation.

HYDROVEX® FluidSep is currently in full operation in Germany, France, Canada, and the United States of America. John Meunier Inc./Veolia Water Technologies designs and manufactures HYDROVEX® FluidSep units for the North America under license from UFT. See Appendix D for an installation list of HYDROVEX® FluidSep units in the North America. All the installations included on the list are for CSO applications. HYDROVEX® FluidSep Vortex Separator are most effective on

removing settleable solids and floatable material. The units have been installed in remote locations, away from treatment plants and have performed well. There are no moving parts within the vortex unit itself. Underflow from the unit can be discharged by gravity to sewers or continuously pumped to an ancillary tank where it would be stored until there is capacity in the interceptor sewer system.

Performance

The performance of HYDROVEX® FluidSep Vortex Separator is similar to that described above for the Storm King® Vortex Separator in terms of contaminants removal since they use similar mechanism for solids removal.

Hydraulics

Vortex units are hydraulically efficient. The head loss is comparable to that described above for the Storm King® Vortex Separator.

Generation of Waste Streams

As discussed under the description of the process and the performance, 10% of the design flow will continuously be removed as underflow. This flow must be pumped from the vortex unit, and since the interceptor is full, no capacity will exist in the interceptor during an overflow event. Therefore, the underflow must be stored in ancillary tanks. The capacity of the ancillary tanks is based upon the underflow flow rate and the duration of the overflow event. Once the event is over the contents of the storage tank can be pumped back into the interceptor. Floatable material captured in the tank is removed at the end of the overflow event as the tank is emptied, and is also sent back into the interceptor.

Complexity

The vortex/swirl separator is a simple process. Hydraulic loading rates can be controlled using sluice gates or overflow weirs. Floats, bubblers, or ultrasonic level sensors would be used to control the underflow sump similar to the Storm King® Vortex Separator.

Limitations

The limitations of the HYDROVEX® FluidSep Vortex Separator are similar to those described above for the Storm King® Vortex Separator.

Construction Costs

Table 2-11 presents preliminary planning level construction cost estimates for flows ranging from 10 MGD to 450 MGD. It includes equipment cost, concrete cost associated with the construction of the tank containing the vortex structure, cost for ancillary tank for underflow storage, installation costs, GC field general conditions, GC OH&P, and contingency. Budgetary equipment pricing provided by the equipment manufacturer Veolia Water Technologies includes only the fabricated stainless-steel vortex structures inside. Cost for outside concrete tank enclosure were estimated based on the sizes of the vortex units. Construction cost for excavation, sitework, soil support, and dewatering, as well as the underflow wet well and the pumps are included in the installation costs. The estimated total construction costs for the HYDROVEX® FluidSep Vortex Separator are plotted against flowrate from 10 MGD to 450 MGD in Figure 2-8.

Operation and Maintenance

The operating costs for the HYDROVEX® FluidSep Vortex Separator are the power costs for the underflow pump. The horsepower of the pumps increases as the size of the vortex separator, and correspondingly the underflow, increase. Maintenance costs for the HYDROVEX® FluidSep unit include inspection of the vortex separator and removal of coarse debris (if any) after first heavy rainfall event and then every six months. Once every year, a full inspection of the unit is recommended, including cleaning of the area, visual inspection for abnormalities, like leaks, cracks in the unit's tank and pipe works. Perform visual inspection of all anchors and bolted assemblies. During visual inspection, all normal safety procedures are recommended to be used to prevent any kind of injury. Underflow pumps are recommended to be replaced every six months for overhaul and sharpening of the cutter blades.

Estimated annual operation costs for the HYDROVEX® FluidSep Vortex Separator are presented on Table 2-12 containing factors for calculation of operating costs; while estimated annual maintenance labor cost including cost factors are included on Table 2-13.

Space Requirements

The space requirements of the HYDROVEX® FluidSep Vortex Separator shall be based upon a square area utilizing the diameter of the tank and a buffer of 5 feet on each side.

Case Study

In 2016, Mattoon, IL installed a HYDROVEX® FluidSep Vortex Separator at their Riley Creek satellite CSO treatment facility. As of September 2017, the unit has not been in service yet. The Riley Creek facility is in a remote location and designed for 15 MGD. The application required a 12" gravity underflow line (at 2 ft/s flow) for 3 or 4 MGD of underflow, which will get pumped back to the wastewater treatment plant. This large amount of underflow requires having almost one pump dedicated to pumping it back to the WWTP.

Table 2-11 - Preliminary Construction Cost Estimates for HYDROVEX Fluidsep Vortex Separator

Flow	System	Diameter x Depth	Budgetary Equipment Price	Concrete Structure Cost	Auxiliary Tank Cost(1)	Install Cost(2)	GC General Conditions (3)	GC OH&P(4)	Contingency(5)	Total
10 MGD	(1) Type 1	20'-0" x 20'-0"	$60,000	$82,000	$871,200	$759,900	$177,310	$177,310	$1,063,860	$3,191,580
25 MGD	(1) Type 2	35'-0" x 19'-6"	$81,000	$181,000	$1,573,000	$1,376,250	$321,125	$321,125	$1,926,750	$5,780,250
50 MGD	(1) Type 2	45'-0" x 24'-6"	$85,700	$291,500	$2,300,000	$2,007,900	$468,510	$468,510	$2,811,060	$8,433,180
75 MGD	(1) Type 2	45'-0" x 24'-5"	$85,700	$291,500	$3,040,000	$2,562,900	$598,010	$598,010	$3,588,060	$10,764,180
100 MGD	(1) Type 2	50'-0" x 27'-5"	$113,900	$359,000	$3,720,000	$3,144,675	$733,758	$733,758	$4,402,545	$13,207,635
450 MGD	(4) Type 2	50'-0" x 27'-5"	$455,600	$718,000	$10,890,000	$9,047,700	$2,111,130	$2,111,130	$12,666,780	$38,000,340

Notes:
(1) Auxiliary Tank costs derived from quotation from Mid Atlantic Storage System on Aquastore Glass Fused to Steel Storage Tank of 150,000 gal
(2) Installation cost is assumed at 75% of the equipment cost.
(3) GC general conditions are estimated at 10% of the total direct cost.
(4) GC OH&P are estimated at 10% of the total direct cost.
(5) 50% of Contingency is used for the planning level of cost estimates.

Figure 2-10 - Total Estimated Construction Cost of HYDROVEX FluidSep Vortex Separator

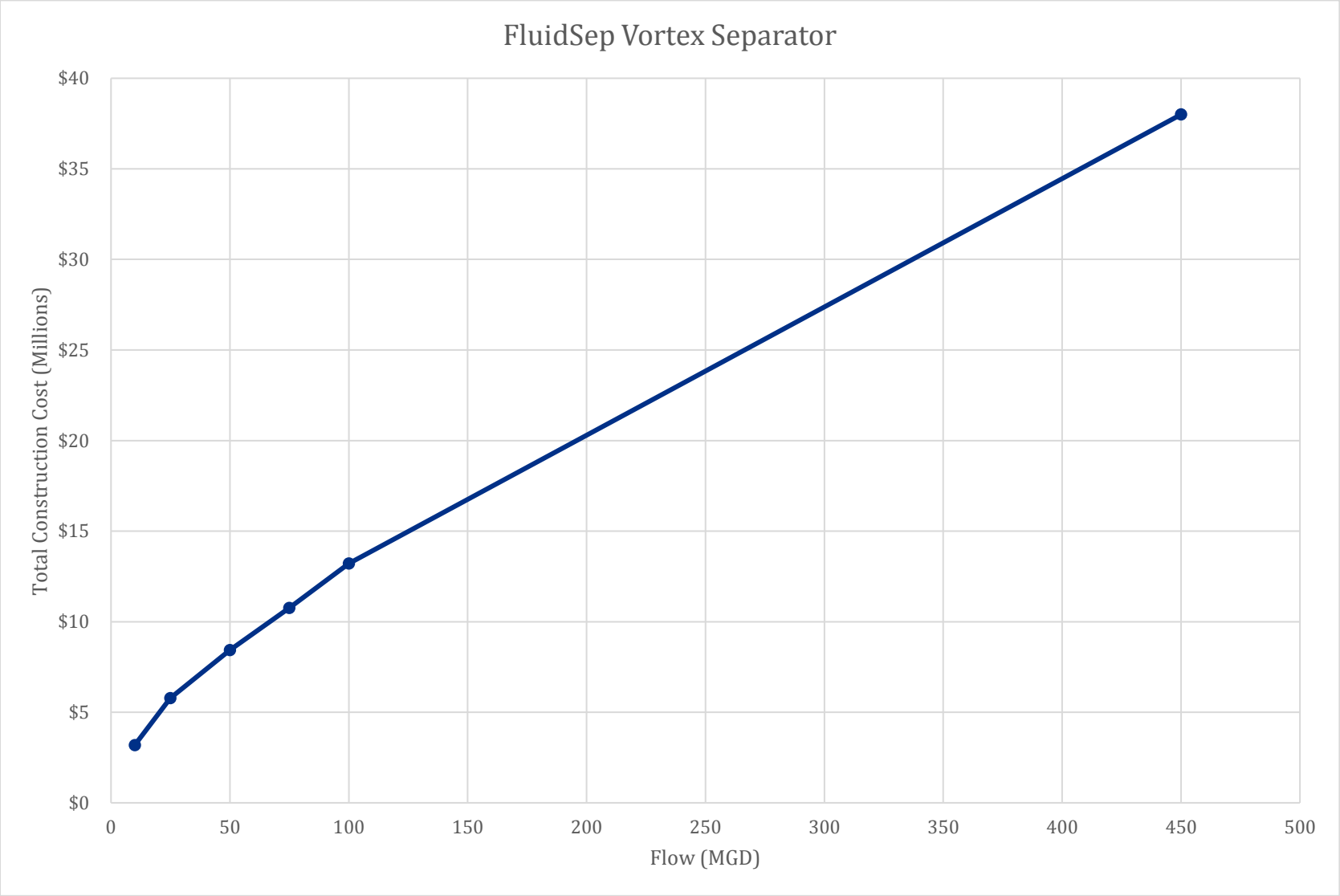


Table 2-12 - Annual Operation Cost of HYDROVEX Fluidsep Vortex Separator

Flow	System	Total Horsepower (HP)	Total Power (kW)(1)	Annual Energy Usage (kW-hr)(2)	Annual Cost(3)
10 MGD	(1) Type 1	14	10	1	$731
25 MGD	(1) Type 2	35	26	4	$1,827
50 MGD	(1) Type 2	70	52	7	$3,654
75 MGD	(1) Type 2	104	78	11	$5,429
100 MGD	(1) Type 2	139	104	15	$7,256
450 MGD	(4) Type 2	625	466	65	$32,624

Notes:
(1) HP x 0.7457
(2) Assumes 500 hours of annual operation
(3) Assumes energy costs of $0.14/kW-hr

Table 2-13 - Annual Maintenance Labor Cost of HYDROVEX Fluidsep Vortex Separator

Maintenance Frequency	Parts	Description	Estimated Man-Hours	Annual Cost(1)
Biannually	Tank and pipe	Visual check and removal of coarse debris (if any)	1	300
Annually	Full Inspection	Cleaning, check for leaks/cracks in unit tank and pipes; visual inspection of all anchors and bolted assemblies	2	300
Biannually	Underflow pumps	Replacement of underflow pumps	8	400
Total Annual Maintenance Cost				$1,000

Notes:
(1) Assumes labor rate of $150/hour

2.3.1.3 SANSEP

Description of Process

The SanSep process is a variation of the typical vortex/swirl separation process, in that it utilizes a screen at the mid-depth of the tank where the treated flow exits the tank. Using the patented non-blocking screen, all gross solids larger than 0.04" and finer sediments down to below 0.004" are captured and retained inside the unit. The settleable solid pollutants settle into the lower catchment chamber while the floatables are retained at the surface of the upper chamber. A flow of liquid is maintained across the face of the screen producing a "washing" effect that keeps the solids moving while the fluid passes through the screen. The SanSep is typically automated with an underflow pump, which periodically removes the solids and returns them to the interceptor sewer. The non-blocking screen operates continuously at its maximum design flow. Cross section of a SanSep unit is shown in Figure 2-11.

Figure 2-11 - Cross Section of a SanSep Unit

(Source:PWTech.)

Application to the Project

SanSep was initially developed in Australia as a stormwater treatment system by the corporate predecessor of PWTech (CDS Technologies). The system was introduced in the US in the mid 90's and first used for CSO applications in Louisville Kentucky. Three units have been in continuous operation there since the late 90s. SanSep units have been installed on CSO applications in Cohoes, New York since 2004, and in in Akron, OH and in Weehawken, NJ. since 2004. See Appendix E for an installation list for SanSep for CSO applications in the US, Europe and the Pacific Rim.

Performance

The SanSep unit is more efficient in removal of solids and other pollutants than conventional vortex/swirl separation units due to the use of the screen. The unit removes all solids larger than 1 mm, including organic debris such as vegetation and coarse sediments, fine organic sediments, and significant amounts of BOD and Phosphorus associated with the organic material and fine sediments captured. The SanSep units are also capable of operating at high separation efficiency, over a larger range of hydraulic loading rates than the conventional vortex/swirl separation units. Hydraulic loading rates for conventional units are based upon the horizontal area of the vortex unit, whereas the hydraulic loading rate for the SanSep units are based upon the area of the screen. The screening area, which is greater than the horizontal surface area, and the continuous cleaning action of the flow across the screen enables the SanSep unit to maintain the higher removal rates than conventional units over a wider range of hydraulic loading rates. The performance information from the manufacturer show that there is light drop in removal of TSS as the hydraulic loading rate increases. TSS removal can drop from approximately 70% to 50% as loading rate increases to about 60 gpm/sf.

Hydraulics

Vortex units are hydraulically efficient. The head loss through the unit consists of the losses through the inlet to the unit, and the head loss through the screen. The losses in the lower hydraulic loading rates will be limited to less than six inches. At higher hydraulic loading rates, the losses will increase.

Generation of Waste Stream

The SanSep process has a reduced underflow of 2-3% of the design flow which will continuously be removed as underflow, compared to conventional vortex units with an underflow of 10%. This flow must be pumped from the vortex unit, and since no or limited capacity will exist in the interceptor during an overflow event, the underflow must be stored in ancillary tanks. The capacity of the ancillary tanks is based upon the underflow flow rate and the duration of the overflow event. Once the event is over the contents of the storage tank can be pumped back into the interceptor. Floatable material captured in the tank is removed at the end of the overflow event as the tank is emptied, and is also sent back into the interceptor.

Complexity

The vortex/swirl separator is a simple process, especially since there are no moving parts within the unit. Removals are achieved using natural forces and no adjustment of equipment is necessary. The only controls that are needed are in the flow coming to the unit, in order to ensure that the unit operates within its hydraulic loading rates. This is typically accomplished using sluice gates or overflow weirs. The other area requiring instrumentation would be the control of the underflow sump where underflow is pumped out. The control of the pumping units would be by floats, bubblers, or ultrasonic level sensors.

Limitations

As stated above, the hydraulic loading rate is key to the performance of the vortex/swirl separator. However, since the SanSep unit is able to maintain high removal rates over a wider range of hydraulic loading they perform better in removing TSS, and as a result enable the downstream disinfection processes to be more effective.

Construction Costs

The preliminary report level construction cost estimates provided in Table 2-14 include the equipment, installation, building, land, and contingency for SanSep of design flow ranging from 10 MGD to 100 MGD. Budgetary equipment pricing information for SanSep was gathered from equipment manufacturer Echelon Environmental. Flowrate higher than 100 MGD was considered impractical to use the SanSep unit by the equipment manufacturer. Installation costs are estimated at 150% of the equipment cost per manufacture recommendation. The estimated total construction costs for the SanSep are plotted against flowrate from 10 MGD to 100 MGD in **Figure 2-**12.

Operation and Maintenance

The operating costs for the SanSep vortex separator are the power costs for the underflow pump. The horsepower of the pumps increases as the size of the vortex separator, and correspondingly the underflow, increase. Regular maintenance required for SanSep unit includes inspection of the vortex separator after each rainfall event. After each event, the PLC for the unit initiates a cleaning and wash-down cycle. During this cycle, the underflow pumps empty the unit, followed by a wash-down with clean water directed at the screen through a series of water jets. If a clean water source is not available, the wash-down can also be accomplished using the spray from a vactor truck. The screen should also receive a periodic inspection from the surface to ensure that the cleaning cycle is removing accumulated debris. Unless large debris is accumulating in the structure, it shouldn't be necessary to enter the unit. If it is ever necessary to enter the unit, confined space entry regulations would apply. The underflow pumps are recommended to be replaced every 6 months for overhaul and sharpening of the cutter blades.

Estimated annual operation costs for the SanSep separator are presented on Table 2-15 containing factors for calculation of operating costs; while estimated annual maintenance labor cost including cost factors are included on Table 2-16.

Space Requirements

The space requirements of the SanSep vortex separator shall be based upon a square area utilizing the diameter of the tank and a buffer of 5 feet on each side.

Case Study

The Fort Wayne, Indiana Public Utilities installed the SanSep unit in 2009 at one of their CSO locations to catch floatables half and inch and larger. Prior to the installation, a pilot study was completed in which baskets were installed to observe the types of materials collected. The pilot study showed that the unit was able to capture fine materials. According to the CSO Program Manager, the unit was in use until about 2015 at which point the CSO location was almost entirely eliminated due to Consent Decree regulations. During its operation, there had been no plugging or washdown of the system needed and maintenance consisted of the general routine maintenance. There was also a small pump station which pumps debris back into the wastewater treatment plant. Overall the CSO Program Manager was satisfied with the product.

Table 2-14 - Preliminary Construction Cost Estimates for SanSep

Flow	System	Length X Width	Budgetary Equipment Price	Auxiliary Tank Cost	Install Cost(1)	GC General Conditions (2)	GC OH&P(3)	Contingency(4)	Total
10 MGD	(1) Model 80_80	23'-0" x 25'-6"	$300,000	$420,000	$1,080,000	$180,000	$72,000	$1,026,000	$3,078,000
25 MGD	(2) Model 80_80	42'-0" x 25'-6"	$430,000	$680,000	$1,665,000	$277,500	$111,000	$1,581,750	$4,745,250
50 MGD	(3) Model 80_80	42'-0" x 38'-6"	$560,000	$1,000,000	$2,340,000	$390,000	$156,000	$2,223,000	$6,669,000
75 MGD	(4) Model 80_80	42'-0" x 51'-0"	$690,000	$1,300,000	$2,985,000	$497,500	$199,000	$2,835,750	$8,507,250
100 MGD	(4) Model 80_80	42'-0" x 51'-0"	$690,000	$1,570,000	$3,390,000	$565,000	$226,000	$3,220,500	$9,661,500

Notes:
(1) Installation costs are estimated at 150% of the equipment cost per manufacture recommendation.
(2) GC general conditions are estimated at 10% of the total direct cost.
(3) GC OH&P are estimated at 10% of the total direct cost.
(4) 50% of contingency is used for the planning level of cost estimates.

Figure 2-12 - Total Estimated Construction Cost of SanSep

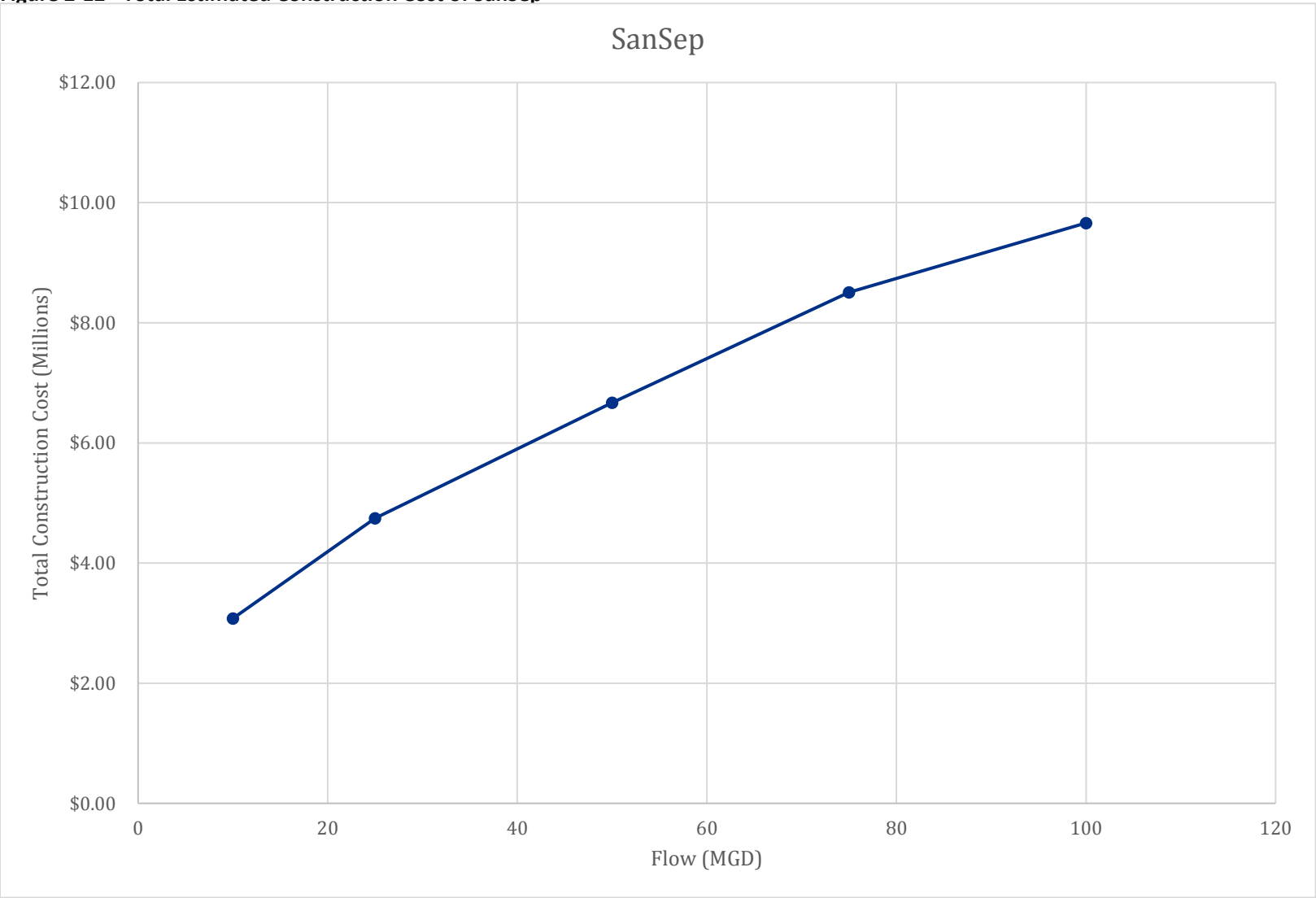


Table 2-15 - Annual Operation Cost of SanSep

Flow	System	Total Horsepower (HP)	Total Power (kW)(1)	Annual Energy Usage (kW-hr)(2)	Annual Cost(3)
10 MGD	(1) Model 80_80	6	4	1	$313
25 MGD	(2) Model 80_80	10	7	1	$522
50 MGD	(3) Model 80_80	10	7	1	$522
75 MGD	(4) Model 80_80	15	11	2	$783
100 MGD	(4) Model 80_80	20	15	2	$1,044

Notes:
(1) HP x 0.7457
(2) Assumes 500 hours of annual operation
(3) Assumes energy costs of $0.14/kW-hr

Table 2-16 - Annual Maintenance Labor Cost of SanSep

Maintenance Frequency	Parts	Description	Estimated Man-Hours	Annual Cost(1)
Biannually	Tank and pipe	Visual check and removal of coarse debris (if any)	1	$300
Annually	Full Inspection	Cleaning, check for leaks/cracks in unit tank and pipes; visual inspection of all anchors and bolted assemblies	2	$300
Biannually	Underflow pumps	Replacement of underflow pumps	8	$400
Total Annual Maintenance Cost				$1,900

Notes:
(1) Assumes labor rate of $150/hour

2.3.2 Ballasted Flocculation

Ballasted flocculation, also known as high rate clarification, is a physical-chemical treatment process that uses microsand, or sludge and a variety of additives to improve the settling properties of suspended solids through improved floc bridging. The objective of this process is to form floc particles with a specific gravity of greater than two. Faster floc formation and decreased particle settling time allow clarification to occur up to ten times faster than with conventional clarification, allowing treatment of flows at a significantly higher rate than allowed by traditional unit processes. Ballasted flocculation units function through the addition of a coagulant, such as ferric chloride; an anionic polymer; and a ballast material such as microsand, a microcarrier, or chemically enhanced sludge. When coupled with chemical addition, this ballast material has been shown to be effective in reducing coagulation-sedimentation time.

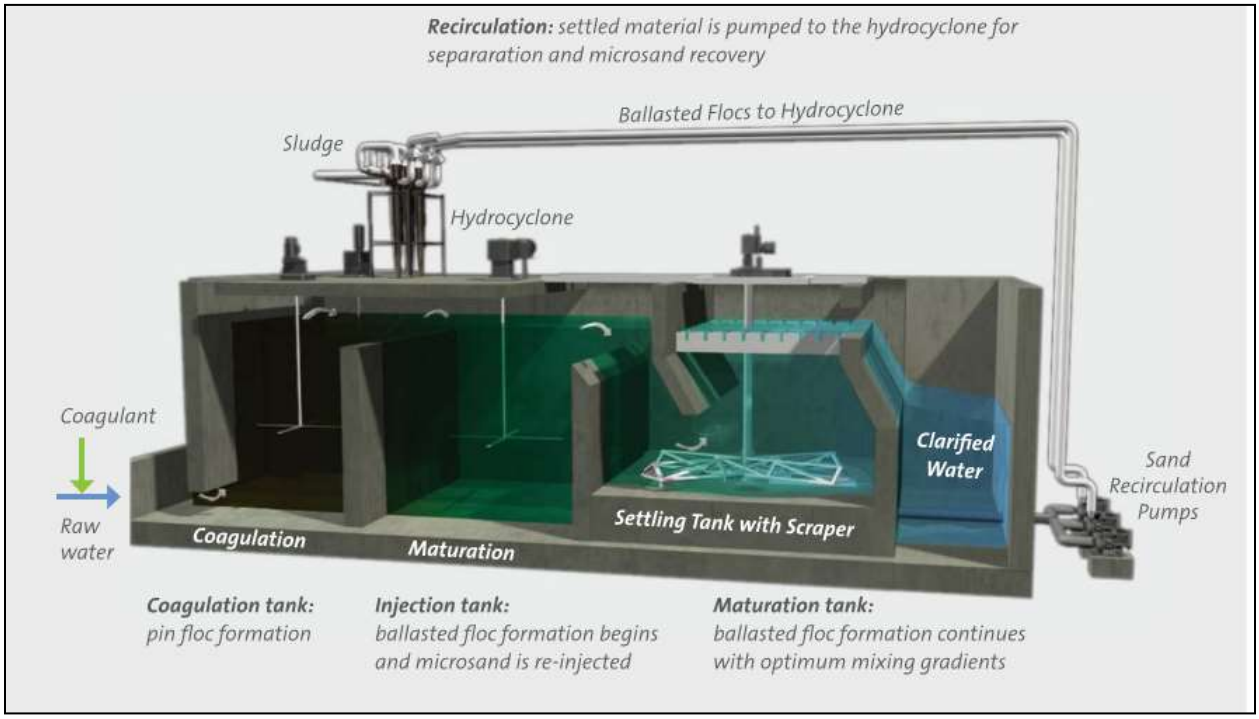
The ballasted flocculation processes, using chemical addition as a critical part of their operation, have higher removal percentages than vortex/swirl separation processes for virtually all the pollutants with the exception of total nitrogen and NH_3. The compact size of ballasted flocculation units can significantly reduce land acquisition and construction costs. This technology has been applied both within traditional treatment trains and as overflow treatment for peak wet weather flows. Several different ballasted flocculation systems are discussed in more details in sections below.

2.3.2.1 ACTIFLO® Ballasted Flocculation Process

Description of Process

ACTIFLO® is a microsand ballasted clarification process that may be used to treat water or wastewater. The process begins with the addition of a coagulant, such as an iron or aluminum salt, to destabilize suspended solids. The flow enters the coagulation tank for flash mixing to allow the coagulant to rapid mix with the flow after which it overflows into the injection tank where microsand is added. The microsand serves as a seed for floc formation, providing a large surface area for suspended solids to bond to, and is the key to the ACTIFLO® process. The larger flocculation particles allow solids to settle out more quickly, thereby requiring a smaller footprint than conventional clarification. Polymer may either be added in the injection tank or at the next step, the maturation tank. Mixing is slower in the maturation tank, allowing the polymer to help bond the microsand to the destabilized suspended solids. Finally, the settling tank effectively removes the floc with help from the plate settlers. The plate settlers allow the settling tank size to be reduced. Clarified water exits the process by overflowing weirs above the plate settlers. The sand and sludge mixture is collected at the bottom of the settling tank with a conventional scraper system and pumped back to a hydrocyclone, located above the injection tank. The hydrocyclone converts the pumping energy into centrifugal forces to separate the higher-density sand from the lower density sludge. The sludge is discharged out of the top of the hydrocyclone while the sand is recycled back into the ACTIFLO® process for further use. Screening is required upstream of ACTIFLO® so that particles larger than 0.1 - 0.25 mm do not clog the hydrocyclone. Cross section of ACTIFLO® unit is shown in Figure 2-13.

Figure 2-13 - Cross Section of ACTIFLO® Unit



(Source: Veolia Water Technologies)

Applicability to the Project

High rate clarification (HRC) was traditionally used for water treatment until in the late 1990s when HRC demonstration testing programs were performed to verify whether HRC technology would be able to be used for wastewater and CSO treatment. The results of the demonstration programs indicated that HRC can be used for CSO treatment and the effluent quality produced during pilot-testing surpassed CSO treatment standards, making it amenable to subsequent UV disinfection.

The ACTIFLO® system, as one type of HRC that uses ballasted flocculation, can be installed at the treatment plant or at a satellite facility within the collection system. The Actiflo process can be fully automated and the process train(s) can sit idle for extended periods of time and still be fully operational within 15 minutes of start-up. Installations at the WWTP also enable the sludge produced by the unit to be processed with existing systems. When installing the ACTIFLO® unit in a remote CSO location, the flows will vary widely, and the sludge must be stored in ancillary tanks so it can be put back into the interceptor during periods of low flow. Appendix F summarizes ACTIFLO® installations in the USA. The table lists only installations used for wastewater treatment operations. System applications include Primary WW, Primary WW/CSO, Primary WW/ Tertiary WW, CSO, CSO/Tertiary WW, and Tertiary WW treatment operations.

Performance

The ACTIFLO® ballasted flocculation process is sized for the peak hour or day flow to prevent flow from exceeding the capacity of the unit. The units are designed for a surface-loading rate of 60 gallons per minute per square foot, at a peak hydraulic loading rate of 150%. When starting up the

unit it takes between 15-30 minutes for the process to reach steady state conditions. Accordingly, the initial 15-30 minutes of operation receives only little or partial treatment. The ACTIFLO® ballasted flocculation process is very effective in removing most of the pollutants; especially since the addition of flocculants and polymers helps remove smaller particles. Performance for removal of pollutants is reportedly constant up to for a surface-loading rate of 60 gallons per minute per square foot. See Table 2-17 for manufacturer provided performance efficiency. Performance deteriorates quickly for higher surface loading rates than 60 gallons per minute per square foot.

Table 2-17 - Anticipated Performance Efficiency

Parameter	Removal Rate
TSS	80 - 95%
COD	50 - 70%
Total BOD	50- 80%
Soluble BOD	10 - 20%
Total P	80 - 95%
TKN	15 -20%
Heavy Metals	85 -100%
Oils & Grease	50 -80%
Fecal Coliform	85 -95%

Hydraulics

The head loss through the units at peak flow rates are reported at less than two feet.

Generation of Waste Streams

As previously noted, the initial 15-30 minutes of operation of the unit provides no or only partial treatment. Since the disinfection process requires consistent pretreatment removals of TSS, the discharge of this partially treated flow will result in only partial disinfection. One potential means of eliminating this problem would be to provide ancillary tanks for storage of the initial discharge. This storage can then be reintroduced to the treatment process once the unit is fully operational. Under the description of the process, sludge is produced and separated in a hydrocyclone unit. The solids percentage of the waste sludge will vary depending on the concentration of the influent TSS and the coagulant dosage. In most cases the solids concentrations will vary from 0.1 to 1.0% with an average of 0.3%. Sludge from the ACTIFLO® process is easily treated and dewatered. When the ACTIFLO® process is located at the WWTP the sludge is sent back to the head of the plant or primary clarifiers, in some cases it is sent to intermediate gravity thickeners and then on to centrifuges or belt thickeners for final processing. The sludge production is approximately 4.8% of the design capacity of the unit.

Complexity

The ACTIFLO® ballasted flocculation process is more complex than the vortex/swirl separator process. The ACTIFLO® ballasted flocculation process consists of chemical addition, which must be controlled by the flow rate, mixers and flocculators, sludge pumps and a hydrocyclone, which separates the sludge from the microsand.

Limitations

The startup time for the ACTIFLO® process of from 15 to 30 minutes is a limitation in that for stringent treatment objectives the flow from the unit during this time period must be stored and fed back into the system later. For some drainage areas, this startup period may correspond to the first flush when the loading is the greatest. Also, the ACTIFLO® process has 4:1 turndown ratio, which means the minimum flow through the unit is 25% of the unit's capacity. Flows lower than this result in process problems. There is a maximum TSS limit on the ACTIFLO® process at the higher loading rate of 60 gpm/sf, of between 500 to 1000 mg/L TSS. This value is high and should not provide a routine problem in the operation of the unit. In remote locations, the ACTIFLO® process will see intermittent operation which will make operation more challenging.

Construction Costs

The preliminary planning level construction cost estimates are provided in Table 2-18 for ACTIFLO® Ballasted Flocculation Unit of design flow ranging from 10 MGD to 450 MGD. It includes equipment cost, installation costs, GC field general conditions, GC OH&P, and contingency. Budgetary equipment pricing information for ACTIFLO® Ballasted Flocculation Unit was gathered from equipment manufacturer Veolia Water Technologies. The equipment price includes engineering and project management time. Cost for concrete structure and auxiliary tank for waste sludge storage were also estimated based on equipment sizing and design flowrate. Installation cost was assumed at 115% of equipment cost based on equipment manufacturer's recommendations. The installation cost includes assembly of the ACTIFLO® ballasted flocculation unit, excavation and backfilling, and the cost of the Chemical Building and the chemical feed equipment. The estimated total construction costs for the ACTIFLO® Ballasted Flocculation Unit are plotted against flowrate from 10 MGD to 450 MGD in **Figure 2-**14.

Operation and Maintenance

Operating costs for the ACTIFLO® Ballasted Flocculation unit consists of the power and chemical costs. Power costs are based upon the horsepower of the mixers, flocculators, chemical feed equipment and pumps. Chemical costs are based on usage of coagulant and polymer. Regular maintenance includes routine lubrication and maintenance of the mixers, scrapers, pumps, hydrocyclones and other mechanical components. Weekly inspections and preventive maintenance are important to keep an intermittent-use facility ready to operate at a moment's notice. When the unit will be offline for more than 8 hours, the units will be completely drained and all equipment stopped.

Estimated annual operation costs for the ACTIFLO® system are presented on Table 2-19 containing factors for calculation of operating costs; while estimated annual maintenance labor cost including cost factors are included on Table 2-20.

Space Requirements

The space requirements of the ACTIFLO® units consist of the size of the tanks and a buffer of 5 feet around the unit for access and maintenance.

Case Study

The Water Environment Federation's (WEF) February 2012 issue of Water Environment and Technology (WE&T) provided a case study on the use of HRC in the city of Bremerton, Washington. Bremerton adopted a proprietary high rate compact clarification process to reduce its CSO discharges. Followed by an ultraviolet disinfection treatment, the HRC process was piloted by CDM Smith in 1999. The pilot testing determined effluent capable of being discharged into sensitive waterways would be produced by the HRC process and that a UV disinfection treatment could be added to the process. This project received the 2002 Grand Award in Small Projects by the American Academy of Environmental Engineers (Annapolis, MD).

The process takes wet weather flow that cannot be handled by the wastewater treatment plant, and puts it through a flash mixing tank with polymer added, and a maturation tank before it is sent through a clarifier. Reduction of BOD5 and TSS is typically 60-65% and 90-95%, respectively. Sludge from the clarifier is pumped back to the hydrocyclone and then either to the solids processing plant, or through a microsand filter and into the flash mixing tank. The facility utilizes a 10 MGD nominal capacity with a maximum hydraulic capacity of 20MGD. Additionally, flow to the facility is minimized by a 100,000-gallon storage tank, which has reduced overall CSO occurrences by 80% in the surrounding collection system. The HRC facility only receives flow when the storage tank fills over a weir wall.

Weekly inspection and maintenance is required to ensure the facility is ready to operate when the next rainfall occurs. Additionally, a small flow (less than 3 gal/min) of chlorinated potable water is discharged into the injection tank during periods of dry weather to eliminate the chance of biofouling on lamella tubes and other components. The facility has had issues with UV ballast burnout due to short durations of high intensity operation. Since installation, operators have adjusted the coagulant injection point to increase flocculation time. Additionally, the discharge was relocated from the hydrocyclone to the far side of the storage tank to reduce sand loss and resuspension of separated solids. Operators spent several years altering the chemical dosing to meet permitted discharge requirements as there are very few events each year which trigger the HRC.

Table 2-18 - Preliminary Construction Cost Estimates for ACTIFLO Ballasted Flocculation Unit

Flow	System	Length X Width of ACTFLO Unit	Auxiliary Tank Volume	Budgetary Equipment Price	Concrete Cost	Auxiliary Tank Cost	Install Cost(1)	GC General Conditions(2)	GC OH&P(3)	Contingency(4)	Total
10 MGD	(1) 10 MGD	44'-9" x 14'-0"	0.1 MG	$1,325,000	$204,300	$610,000	$1,604,475	$374,378	$374,378	$2,246,265	$6,738,795
25 MGD	(1) 25 MGD	60'-9" x 22'-0"	0.25 MG	$1,900,000	$341,100	$970,000	$2,408,325	$561,943	$561,943	$3,371,655	$10,114,965
50 MGD	(1) 50 MGD	82'-3" x 32'-0"	0.5 MG	$2,725,000	$532,800	$1,570,000	$3,620,850	$844,865	$844,865	$5,069,190	$15,207,570
75 MGD	(3) 25 MGD	60'-9" x 66'-0"	0.75 MG	$4,725,000	$675,000	$2,100,000	$5,625,000	$1,312,500	$1,312,500	$7,875,000	$23,625,000
100 MGD	(2) 50 MGD	82'-3" x 64'-0"	1.0 MG	$5,250,000	$801,900	$2,300,000	$6,263,925	$1,461,583	$1,461,583	$8,769,495	$26,308,485
450 MGD	(6) 75 MGD	116'-0" x 73'-2"	4.5 MG	$10,000,000	$3,204,900	$6,900,000	$15,078,675	$3,518,358	$3,518,358	$21,110,145	$63,330,435

Notes:
(1) Installation costs are estimated at 115% of the equipment cost per manufacture recommendation.
(2) GC general conditions are estimated at 10% of the total direct cost.
(3) GC OH&P are estimated at 10% of the total direct cost.
(4) 50% of contingency is used for the planning level of cost estimates.

Figure 2-14 - Total Estimated Construction Cost of ACTIFLO® Ballasted Flocculation Unit

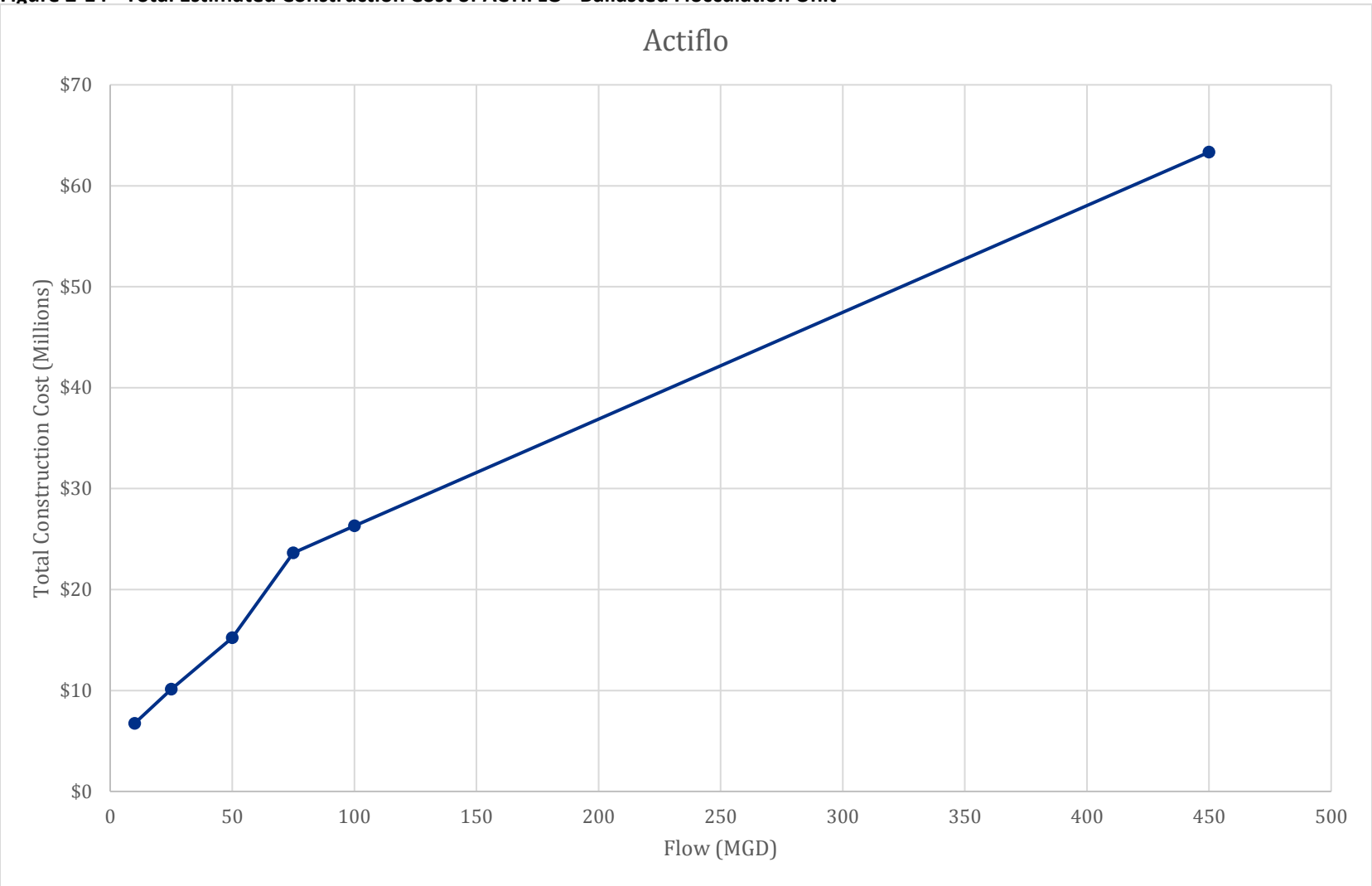


Table 2-19 - Annual Operation Cost of ACTIFLO® Ballasted Flocculation

Flow	Required Horsepower (HP)						Total Power (kW)(1)	Annual Energy Usage (kW-hr)(2)	Annual Power Cost(3)	Alum Usage (lbs)(4)	Polymer Usage (lbs)(5)	Alum Cost(6)	Polymer Cost(7)	Total Annual Cost
	Coag-ulation Mixer	Matur-ation Mixer	Scraper Drive & Mech-anism	Sand Pump	Chemical Pump	Total HP								
10 MGD	10	7.5	2	80	0.5	100	75	37,285	$5,220	173,854	3,477	$10,014	$6,676	$21,910
25 MGD	25	20	7.5	100	0.5	153	114	57,046	$7,986	434,635	8,693	$25,035	$16,690	$49,711
50 MGD	20	30	15	120	1	186	139	69,350	$9,709	869,271	17,385	$50,070	$33,380	$93,159
75 MGD	75	60	22.5	300	1	458.5	342	170,952	$23,933	1,303,906	26,078	$75,105	$50,070	$149,108
100 MGD	80	60	30	240	1.5	411.5	307	153,428	$21,480	1,738,542	34,771	$100,140	$66,760	$188,380
450 MGD	360	270	135	1,080	2	1847	1,377	688,654	$96,412	7,823,438	156,469	$450,630	$300,420	$847,462

Notes:

(1) HP x 0.7457
(2) Assumes 500 hours of annual operation
(3) Assumes energy costs of $0.14/kW-hr
(4) Assume an alum dosage of 100 mg/L
(5) Assumes a polymer dosage of 2 mg/L
(6) Assumes an alum cost of $0.0576/lb
(7) Assumes a polymer cost of $1.92/lb

Table 2-20 - Annual Maintenance Labor Cost of ACTIFLO Ballasted Flocculation Unit

Frequency	Parts	Description	Estimated Man-Hours	Annual Cost(1)(2)
Biannually	Coagulation Mixers	Change oil and grease bearings	1	$300
Biannually	Maturation Tank Mixer	Change oil and grease bearings	1	$300
Biannually	Scraper	Change oil and grease bearings	1	$300
Annually	Chemical pumps	Grease bearings	0.5	$75
Biannually	Sand Pumps	Grease bearings	0.5	$150
Annually	Sand Pumps	Change belts	1	$150
Annually	Hydrocyclone	Inspect / change apex tips	0.25	$38
Monthly	Lamella	Cleaning	1 / basin	$3,600
Weekly	System	Inspection and preventive maintenance	0.5	$3,900
After each overflow event	System	System shut down and drain	2	$30,000
Total Annual O&M Cost				$38,813

Notes:
(1) Assumes 100 events per year
(2) Assumes labor rate of $150/hour

2.3.2.2 DensaDeg® Ballasted Flocculation Process

Description of Process

The DensaDeg® is a is a high-rate settling clarifier process combining solids contact, ballast addition and solids recirculation to provide enhanced, high-rate settling of solids. Different from ACTIFLO®, recycled sludge, instead of microsand, is added to increase floc density and precipitation. The process consists of:

1. Rapid mix / coagulation stage: Raw water flows into the rapid (flash) mix zone where a coagulant is added. Coagulation is the destabilization of colloidal particles, which facilitates their aggregation and is achieved by the injection of a coagulant such as alum or ferric chloride.

2. Flocculation zone: Coagulated water then flows to the flocculation zone where, with a lower energy vertical turbine mixer, a continuous ballast media recirculation feed and a low dose of a flocculating agent (polymer) are added to begin the process of agglomerating the coagulated water into floc particles.

3. Maturation zone: Flocculated particles are then developed and grown into large, very dense mature particles. This is achieved with optimized mixing energy and detention time. The result is a floc which settles at extremely high rates.

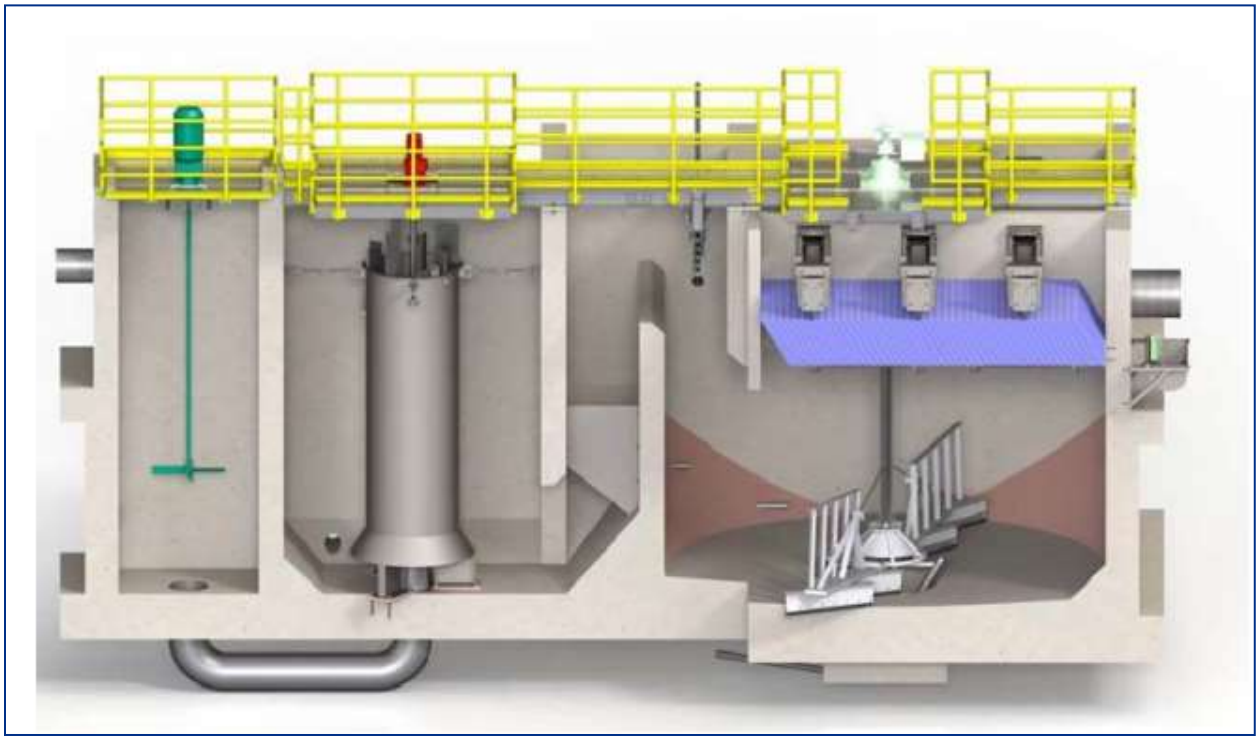
4. Settling & clarification zone: Flocculated solids enter the settling zone, over a submerged weir wall, where dense, suspended matter settles to the bottom of the clarifier. Clarified water is displaced upward from the downward moving slurry, through inclined plate settlers. The plate modules act as a polishing step for lighter, low density solids.

5. Hydrocyclone and ballast recovery: Settled sludge is continuously recycled via a recirculation pump to the hydrocyclone where the ballast media is separated from the waste stream. Ballast is returned to the flocculation zone and the waste stream is sent to sludge handling.

6. Effluent Collection: Uniform collection of clarified water is accomplished in effluent launders above the settling plate assembly.

Cross section of a DensaDeg® unit is shown in Figure 2-15.

Figure 2-15 - Cross Section of a DensaDeg Unit



(Source: Suez North America)

Applicability to the Project

The DensaDeg® ballasted flocculation process is a treatment process that combines solids contact, ballast addition and solids recirculation in a packaged system. It started with the original solids-contact clarifier, the Accelator, which was the first to incorporate internal sludge recycling. In the late 1980's the original DensaDeg clarifier was introduced to the market for high-rate sludge ballasted and solids recirculation systems. The earliest DensaDeg® CSO installation was in 1995.

The DensaDeg® process can be fully automated and the process train(s) can sit idle for extended periods of time and still be fully operational within 30 minutes of start-up. It can be installed at the treatment plant or at a satellite facility within the collection system. Installations at the WWTP also enable the sludge produced by the unit to be processed. When installing the DensaDeg unit in a remote CSO location, the flows will vary widely, and the sludge must be stored so it can be put back into the interceptor at periods of low flow.

Appendix G presents a list of select installations for the original DensaDeg® in CSO/SSO applications.

Performance

The DensaDeg® ballasted flocculation process is sized for the peak hour or day flow to prevent flow from exceeding the capacity of the unit. The units are designed for a surface-loading rate of 40-60 gallons per minute per square foot. When starting up the unit it takes 30 minutes for the process to reach steady state conditions and no sludge inventory is required for startup. The DensaDeg® ballasted flocculation process is very effective in removing vast quantities of pollutants. Its

performance is comparable to ACTIFLO® in terms of contaminants removal with TSS removal of 80-90%, typically providing effluent <30mg/L TSS (inlet dependent) and BOD %-removal similar in magnitude to TSS %-removal, when treating typical municipal WW which is 30-40% of total BOD. Removal could be higher depending on soluble ratio.

Hydraulics

The head loss through the units at peak flow rates are reportedly less than two feet.

Generation of Waste Streams

As previously indicated in the description of the process, a portion of the sludge is wasted. The solids percentage of the waste sludge will vary depending on the concentration of the influent TSS and the coagulant dosage. In most cases the solids concentrations will 4%. The quantity of sludge is approximately equal to 0.5% of the capacity of the DensaDeg® unit. When the DensaDeg® process is located at the WWTP, the sludge is sent back to the head of the plant or primary clarifiers, in some cases it is sent to intermediate gravity thickeners and then on to centrifuges or belt thickeners for final processing.

Complexity

Similar to ACTIFLO®, the DensaDeg® ballasted flocculation process consists of chemical addition, which must be controlled by the flow rate, mixers and flocculators, and sludge pumps.

Limitations

DensaDeg® has similar limitations as previously stated for ACTIFLO® plus it requires a longer start time.

Construction Costs

The preliminary planning level construction cost estimates are provided in Table 2-21 for DensaDeg® ballasted flocculation equipment of design flow ranging from 10 MGD to 450 MGD. It includes equipment cost, installation costs, GC field general conditions, GC OH&P, and contingency. Budgetary equipment pricing information for DensaDeg® ballasted flocculation units was gathered from equipment manufacturer Suez. The equipment price includes engineering and project management time. Cost for concrete structure and auxiliary tank for waste sludge storage were also estimated based on equipment sizing and design flowrate. Installation cost was assumed at 115%. The installation cost includes assembly of the DensaDeg® ballasted flocculation unit, excavation and backfilling, and the cost of the Chemical Building and the chemical feed equipment. The estimated total construction costs for the DensaDeg® ballasted Flocculation Unit are plotted against flowrate from 10 MGD to 450 MGD in **Figure 2-**16.

Operation and Maintenance

Similar to ACTIFLO® ballasted flocculation system, operating costs for the DensaDeg® Ballasted Flocculation unit consist of the power and chemical costs. Power costs are based upon the horsepower of the mixers, flocculators, chemical feed equipment and pumps. Chemical costs are

based on usage of coagulant and polymer. Routine maintenance and preventive care measures are similar to those for ACTIFLO® unit.

Estimated annual operation costs for the DensaDeg® Ballasted Flocculation unit are presented on

containing factors for calculation of operating costs; while estimated DensaDeg® Ballasted Flocculation unit annual maintenance labor cost including cost factors are included on Table 2-23.

Space Requirements

The space requirements of the DensaDeg® unit shall consist of the size of the tanks and a buffer of 5 feet around the unit for access and maintenance.

Case Study

Veolia Water Technologies provided a white paper[1] detailing the City of Akron, OH, BIOACTIFLO™ demonstration project. Beginning in March of 2012, a pilot plant at the City of Akron Water Reclamation Facility (WRF) was constructed to demonstrate effectiveness of the BIOACTIFLO™ technology. Incorporating high-rate activated sludge in the ACTIFLO™ high-rate ballasted flocculation process, BIOACTIFLO™ is designed to remove soluble BOD that would not otherwise be removed. Influent flow to the pilot plant was pumped from a location that had already undergone preliminary treatment, consistent with plans for the full-scale configuration. Return activated sludge (RAS) was supplied to the pilot plant from the gravity belt thickener building of the WWTP, consistent with plans for the full-scale configuration. Optimal doses for coagulant (alum) and polymer were determined. Both BIOACTIFLO™ and main plant secondary effluent were disinfected in a 0.53 MLD (0.14 mgd) pilot UV disinfection system and comparable results were obtained. Following all testing, effluent from the BIOACTIFLO™ pilot was sent back to the main plant for complete secondary treatment.

The pilot unit was operated during a total of twenty (20) wet weather events between April and December 2012, however the last two events (19 and 20) were performed using slightly different Operational Criteria. Pilot plant operation and sampling was conducted over a range of event durations and volumes, ranging from just under an hour to nearly a day in duration. Results showed an average 85% reduction in CBOD (90% reduction for events 19 and 20). Soluble CBOD concentration dropped from 9.2 mg/L in the influent of the BIOACTIFLO™ to 4.1 mg/L in the effluent from the BIOACTIFLO™. Meanwhile, TSS was reduced by 97%, from influent 144.8 mg/L to 4.0 mg/L effluent. Overall results document the effectiveness of BIOACTIFLO™ as a potential parallel wet weather treatment process at facilities facing wet weather treatment challenges.

[1] Heath, Gregory; Gsellman, Patrick; Hanna, Genny; Starkey, Daniel. Pilot Testing of BIOACTIFLO for Wet Weather Treatment at the Akron, Ohio Water Reclamation Facility

Table 2-21 - Preliminary Construction Cost Estimates for DensaDeg Ballasted Flocculation Unit

Flow	System	Length X Width	Budgetary Equipment Price	Concrete Cost	Auxiliary Tank Cost	Install Cost(1)	GC General Conditions (2)	GC OH&P(3)	Contingency(4)	Total
10 MGD	(1) XRC-2 Concrete	39' x 16'	$988,000	$204,300	$210,000	$1,612,645	$301,495	$301,495	$1,808,967	$5,426,901
25 MGD	(1) XRC-5 Concrete	54' x 22'	$1,111,400	$341,100	$320,000	$2,038,375	$381,088	$381,088	$2,286,525	$6,859,575
50 MGD	(1) XRC-8 Concrete	78' x 32'	$1,405,800	$532,800	$420,000	$2,712,390	$507,099	$507,099	$3,042,594	$9,127,782
75 MGD	(3) XRC-5 Concrete	54' x 66'	$2,458,320	$675,000	$550,000	$4,235,818	$791,914	$791,914	$4,751,483	$14,254,448
100 MGD	(2) XRC-8 Concrete	78' x 64'	$2,811,600	$801,900	$610,000	$4,857,025	$908,053	$908,053	$5,448,315	$16,344,945
450 MGD(5)	(8) XRC-9 Concrete	84' x 136'	$5,727,000	$3,204,900	$1,570,000	$12,077,185	$2,257,909	$2,257,909	$13,547,451	$40,642,353

Notes:
(1) Installation costs are estimated at 115% of the equipment cost per manufacture recommendation.
(2) GC general conditions are estimated at 10% of the total direct cost.
(3) GC OH&P are estimated at 10% of the total direct cost.
(4) 50% of contingency is used for the planning level of cost estimates.
(5) The cost was conservatively higher based on nine units of 50 MGD system.

Figure 2-16 - Total Estimated Construction Cost of DensaDeg Ballasted Flocculation Unit

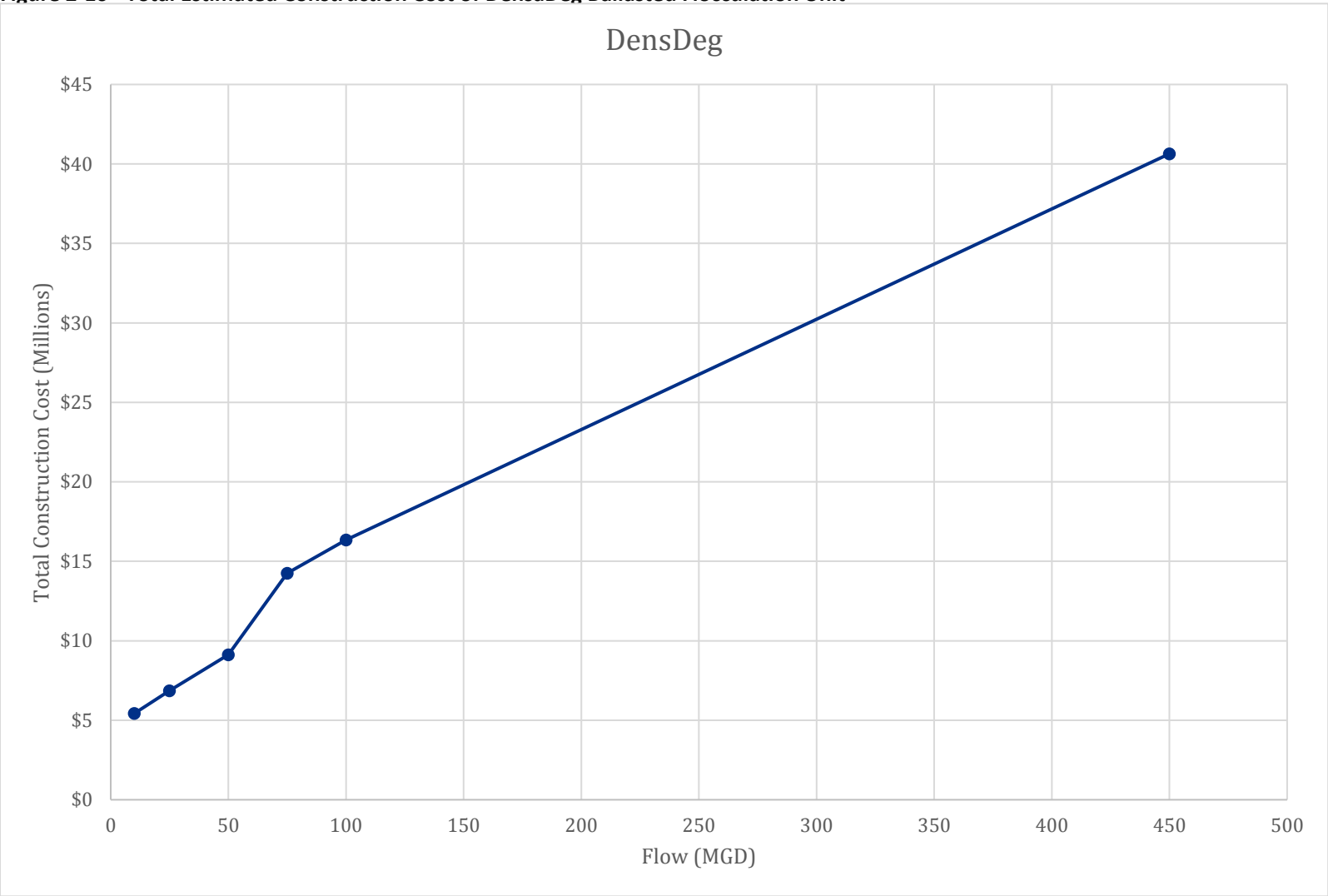


Table 2-22 - Annual Operation Cost of DensaDeg Ballasted Flocculation Unit

Flow	Required Horsepower (HP)						Total Power (kW)(1)	Annual Energy Usage (kW-hr)(2)	Annual Power Cost(3)	Alum Usage (lbs)(4)	Polymer Usage (lbs)(5)	Alum Cost(6)	Polymer Cost(7)	Total Annual Cost
	Rapid Mixer	Reactor Drive	Scraper Drive	Recycle Pump	Chemical Pump	Total HP								
10 MGD	3	5	0.5	30	0.5	39	29	14,541	$2,036	173,854	3,477	$10,014	$6,676	$18,726
25 MGD	5	15	0.5	50	0.5	71	53	26,472	$3,706	434,635	8,693	$25,035	$16,690	$45,431
50 MGD	7.5	15	0.75	50	1	74.25	55	27,684	$3,876	869,271	17,385	$50,070	$33,380	$87,326
75 MGD	12	25	1.25	75	1	114.25	85	42,598	$5,964	1,303,906	26,078	$75,105	$50,070	$131,139
100 MGD	15	30	1.5	100	1.5	148	110	55,182	$7,725	1,738,542	34,771	$100,140	$66,760	$174,625
450 MGD	45	240	6	350	2	643	479	239,743	$33,564	7,823,438	156,469	$450,630	$300,420	$784,614

Notes:
(1) HP x 0.7457
(2) Assumes 500 hours of annual operation
(3) Assumes energy costs of $0.14/kW-hr
(4) Assume an alum dosage of 100 mg/L
(5) Assumes a polymer dosage of 2 mg/L
(6) Assumes an alum cost of $0.0576/lb
(7) Assumes a polymer cost of $1.92/lb

Table 2-23 - Annual Maintenance Labor Cost of DensaDeg Ballasted Flocculation Unit

Frequency	Parts	Description	Estimated Man-Hours	Annual Cost(1)(2)	Frequency
Biannually	Coagulation Mixers	Change oil and grease bearings	1	150	$300
Biannually	Maturation Tank Mixer	Change oil and grease bearings	1	150	$300
Biannually	Scraper	Change oil and grease bearings	1	150	$300
Biannually	Sludge Pumps	Inspect, lubricate pumps and valves, and clean them	2	150	$600
Annually	Chemical pumps	Grease bearings	0.5	150	$75
Annually	Hydrocyclone	Inspect / change apex tips	0.25	150	$38
Monthly	Lamella	Cleaning	1 / basin	150	$3,600
Weekly	System	Inspection and preventive maintenance	0.5	150	$3,900
After each overflow event	System	System shut down and drain	2	150	$30,000
Total Annual O&M Cost					$39,113

Notes:
(1) Assumes 100 events per year
(2) Assumes labor rate of $150/hour

2.3.3 Compressible Media Filtration Process

Description of Process

The compressible media filtration is a process that uses a synthetic, porous filter media. The filter is unusual in a number of ways: (1) the synthetic media is highly porous (89%), (2) filter media and bed properties can be modified because the media is compressible, (3) the fluid to be filtered flows both around and through the media instead of only flowing around the filtering media (as in granular media filters), (4) the fluid that is filtered is used to backwash the filter, (5) to backwash the filter, filter bed volume is increased mechanically, and (6) the filter operates at high filtration rates (up to 40 gal/min/sq. ft.) Performance of the filter, with respect to removal of turbidity and total suspended solids, is similar to the performance of other more conventional filters with the exception that filtration rate is more than 3 to 6 times the rate of other filters. Also, percent backwash water required is significantly less than that used in conventional filtration technologies (typically 1 to 2% versus 6 to 15%).

Compressible media filtration is commercially available as either the "Fuzzy Filter" by Schreiber Industries or the "FlexFilter" by WesTech (both are proprietary technologies covered by patents or pending patents). Both technologies use synthetic fiber spheres as filter media; however, they have different flow configuration, method of bed compression, composition of the synthetic fibers, and media washing details.

The Fuzzy Filter receives the influent at the inlet pipe located at the bottom of the unit. The influent is pressurized upward through the compressed filter media and the effluent is piped out towards the top of the unit, as shown in the process diagram found in Figure 2-17. Porous plates are used to both compress the filter media as well as open up the filter bed to allow movement during backwashing. Figure 17 provides a cross-sectional view of the Fuzzy Filter process, and Figure 2-18 provides an overall picture of the Fuzzy Filter Unit.

Figure 2-17 - Fuzzy Filter Process Diagram

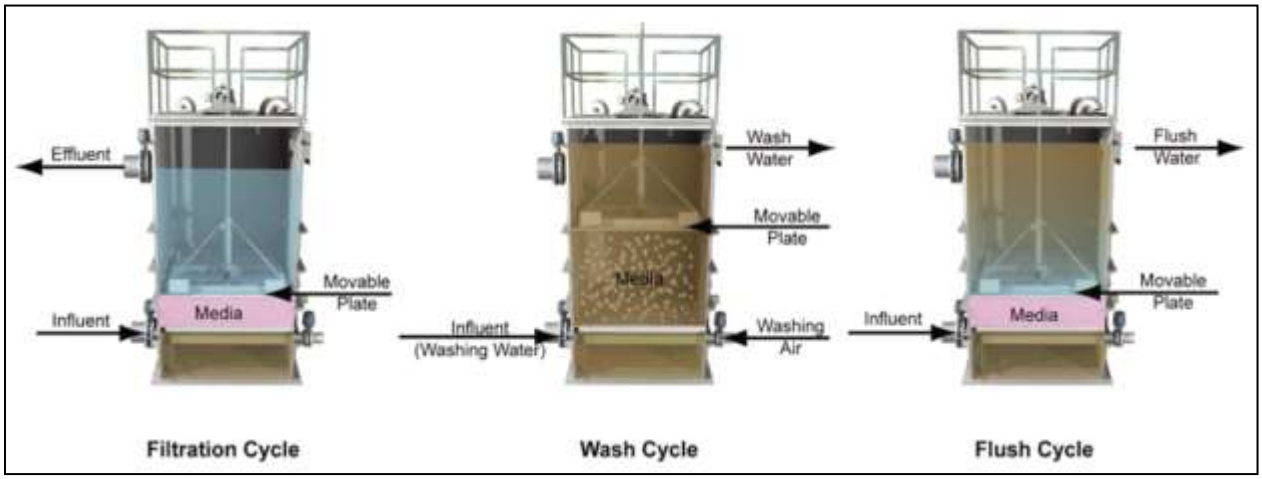
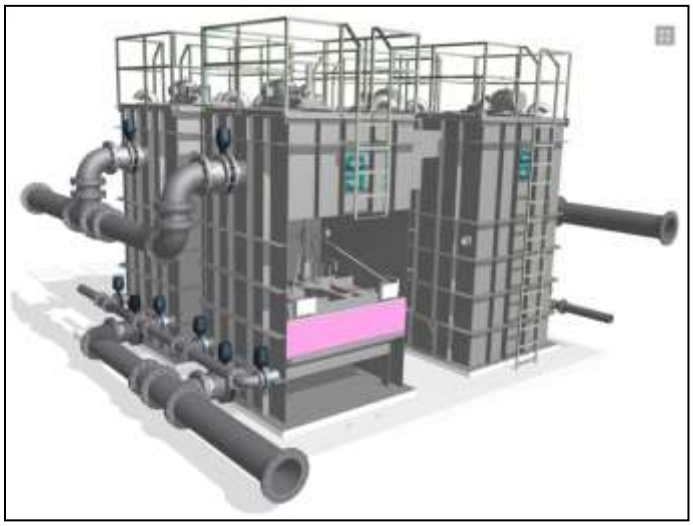


Figure 2-18 - Fuzzy Filter Unit

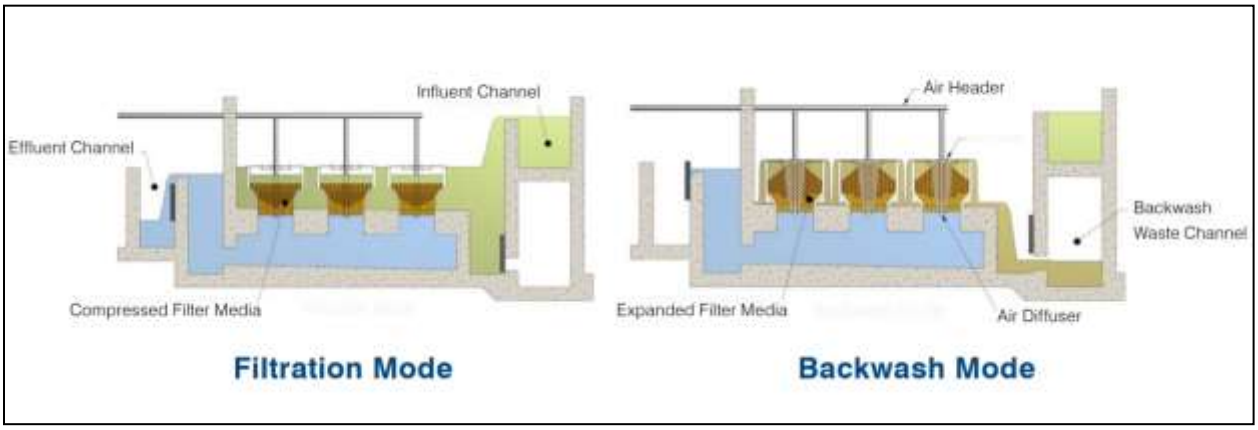


(Source: Schreiber, LLC.)

The FlexFilter receives the inflow from the influent channel. The influent channel is connected to the influent basin where the filter vessels are located. As the influent water accumulates in the influent basin, compression is added to the reinforced rubber sidewalls on the bottom of the filter vessel and compresses the filter bed laterally as the water elevation rises. As the water level in the influent basin reaches the inlet weir elevation, the influent water pours over the influent weir and passes downward through the compressed media bed. Since the bottom of the filter bed compresses more than the top of the filter bed, a porosity gradient is established through the filter bed to capture the largest particles in the upper portion of the filter bed while reserving the deeper portions of the bed to trap finer particles. As particles collect within the media bed, the influent level above the bed rises to a point that signals the need for the media to be cleaned.

The filters use air scouring in the wash cycle to clean the media. During the wash cycle, the feed to the filter is stopped, allowing the media to uncompress. The air scour is initiated along with a small amount of backwash water. The length of the backwash cycle is adjustable. Once cleaned, the filter is put back into service. Figure 2-19 provides a cross-sectional view of the FlexFilter process, and Figure 2-20 provides an overall picture of the FlexFilter Unit.

Figure 2-19 - FlexFilter Process Diagram (Source: WesTech)



(Source: WesTech Engineering, Inc.)

Figure 2-20 - FlexFilter Unit (Source: WesTech)

(Source: WesTech Engineering, Inc.)

Applicability to the Project

The Fuzzy Filter is only used as a polishing step for CSO treatment to meet the most stringent treatment objectives. It does not have a history of treating flows larger than 50 MGD while the FlexFilter has been applied at the 100 MGD Springfield Ohio WWTP treating combined sewer overflow. In addition, the FlexFilter is a simple gravity system requiring no moving parts. The compression of the media is accomplished through a lateral hydraulic force applied from the incoming liquid, eliminating mechanically actuated internal components. For the purpose of the Technical Guidance Manual, FlexFilter was selected for further evaluation.

Performance

For CSO applications FlexFilter is typically operated at 4 gpm/sq. ft. HLR during the first flush portion of a CSO event and gradually increases the operating HLR as the CSO flow rate increases and solids concentration decrease. The maximum HLR of CSO treatment is typically limited to 10 gpm/sq. ft. at design peak flow. The performance information provided by the manufacturer indicates that the contaminants removal efficiency of WWETCO FlexFilter in CSO application ranges from 73% to 94% for TSS removal and 16% to 69% for CBOD removal.

In the Bayonne MUA pilot study, FlexFilter was evaluated in terms of TSS removal. The influent to the FlexFilter was pumped from the Storm King effluent. No raw CSO feed to the FlexFilter was evaluated due to limited wet weather events during the time of the pilot test. The FlexFilter units experienced operating issues primarily related to the pumps and the time needed to backwash. Shorter filter run times and frequent backwashing were experienced when testing was conducted at the higher end of the filter loading rate recommended for CSO treatment.

The pilot study showed that the compressed media filter was consistent and effective in removing finer and organic suspended solids. Overall the FlexFilter was capable of removing 90% of the TSS even at a HLR of 12 to 18 gpm/sq. ft. The unit as tested spent up to 1/2 of the typical four hour run time in backwash cycle, however it was operated at 3 to 4 the recommended hydraulic loading rate in order to supply downstream disinfection with higher flows. TSS removal rates for the FlexFilter improved the ultraviolet transmittance (UVT) of the effluent flow; however, UVT values were still modest. The effluent from the FlexFilter averaged approximately 25 mg/L for TSS and 40% on UVT.

Hydraulics

The headloss through the FlexFilter structure, under the conditions stated above, is about 8 feet.

Generation of Waste Streams

The only waste stream produced by the FlexFilter is the backwashing of the filters. The FlexFilter utilizes low head air to accomplish the media scrubbing while lifting the backwash water to waste, thus minimizing backwash waste volumes. Portions of the backwash water would be diluted with filter drains and recycled back to filter influent. The concentrated backwash water would be stored and put back into the interceptor system when there was available capacity, for removal at the WWTP.

Complexity

As a result of how this unit operates; the automated valves, hydraulically operated porous plate, the air injection into the beds during backwashing, and the monitoring needed for the flow and headloss conditions, this process is the most complex of the pretreatment processes being considered as part of this Technical Guidance Manual.

Limitations

The influent TSS concentration to the FlexFilter is limited to less than 100 mg/L. Higher TSS concentrations will increase the backwash time resulting in overall reduced performance of the units. The 7 feet of headloss through the units is also a limitation since there is usually minimal

head available from the regulator to the discharge at the water body. The valves in the FlexFilter unit are an issue during outdoor operation in freezing weather conditions.

Construction Costs

The preliminary planning level construction cost estimates are provided in Table 2-24 for FlexFilter design flows ranging from 10 MGD to 450 MGD. It includes equipment cost, installation costs, GC field general conditions, GC OH&P, and contingency. Budgetary equipment pricing information for FlexFilter was gathered from equipment manufacturer WesTech Engineering, Inc. The equipment price includes engineering and project management time. Installation cost was assumed at 150% of equipment cost based on equipment manufacturer's recommendations. The installation cost includes assembly of the FlexFilter system, excavation and backfilling, conduits, filter matrix, and backwash and effluent pumping. The estimated total construction costs for the FlexFilter are plotted against flowrate from 10 MGD to 450 MGD in Figure 2-21.

Operation and Maintenance

Estimated annual operation and maintenance costs for FlexFilter unit are presented Table 2-25 based on vendor provided information. It consists of the power costs for the blowers, recycle pumps, and backwash pumps as well as media change-out cost, labor for preventative and routine maintenance, and labor for post event clean-out.

Case Study

According to literature obtained from WWETCO (a subsidiary of WesTech), the FlexFilter™ was installed at the Weracoba Creek Stormwater Treatment system in Columbus, GA. This 10 MGD filter capacity with 2 MGD UV disinfection capacity, was funded by a $0.9 million EPA 319(h) grant to evaluate treatment of urban stormwater runoff. The treatment system has been in operation since 2007. Influent solids ranged from 300 mg/L to 100 mg/L TSS. Effluent TSS was between 5 mg/L and 15 mg/L. Additionally, total maximum daily load (TMDL) requirements for fecal coliform and macro-invertebrates were met. This facility also installed the WWETCO FlexFlow™ Control Valve which allows aquatic biology passage during dry weather flow and causes the head differential needed to operate the filter during wet-weather flow.

Table 2-24 - Preliminary Construction Cost of the FlexFilter

Flow	# Cells	Cell Filter Area (ft^2)	Budgetary Equipment Price	Install Cost(1)	GC General Conditions (2)	GC OH&P(3)	Contingency(4)	Total
10 MGD	5	720	$739,000	$1,108,500	$184,750	$184,750	$1,108,500	$3,325,500
25 MGD	5	1,800	$1,403,000	$2,104,500	$350,750	$350,750	$2,104,500	$6,313,500
30 MGD	5	2,340	$2,797,000	$4,195,500	$699,250	$699,250	$4,195,500	$12,586,500
100 MGD	10	7,200	$3,831,000	$5,746,500	$957,750	$957,750	$5,746,500	$17,239,500
200 MGD	18	12,960	$5,733,000	$8,599,500	$1,433,250	$1,433,250	$8,599,500	$25,798,500
450 MGD	32	23,040	$23,463,000	$35,194,500	$5,865,750	$5,865,750	$35,194,500	$105,583,500

Notes:
(1) Installation costs are estimated at 115% of the equipment cost per manufacture recommendation.
(2) GC general conditions are estimated at 10% of the total direct cost.
(3) GC OH&P are estimated at 10% of the total direct cost.
(4) 50% of contingency is used for the planning level of cost estimates.

Figure 2-21 - Total Estimated Construction Cost of FlexFilter

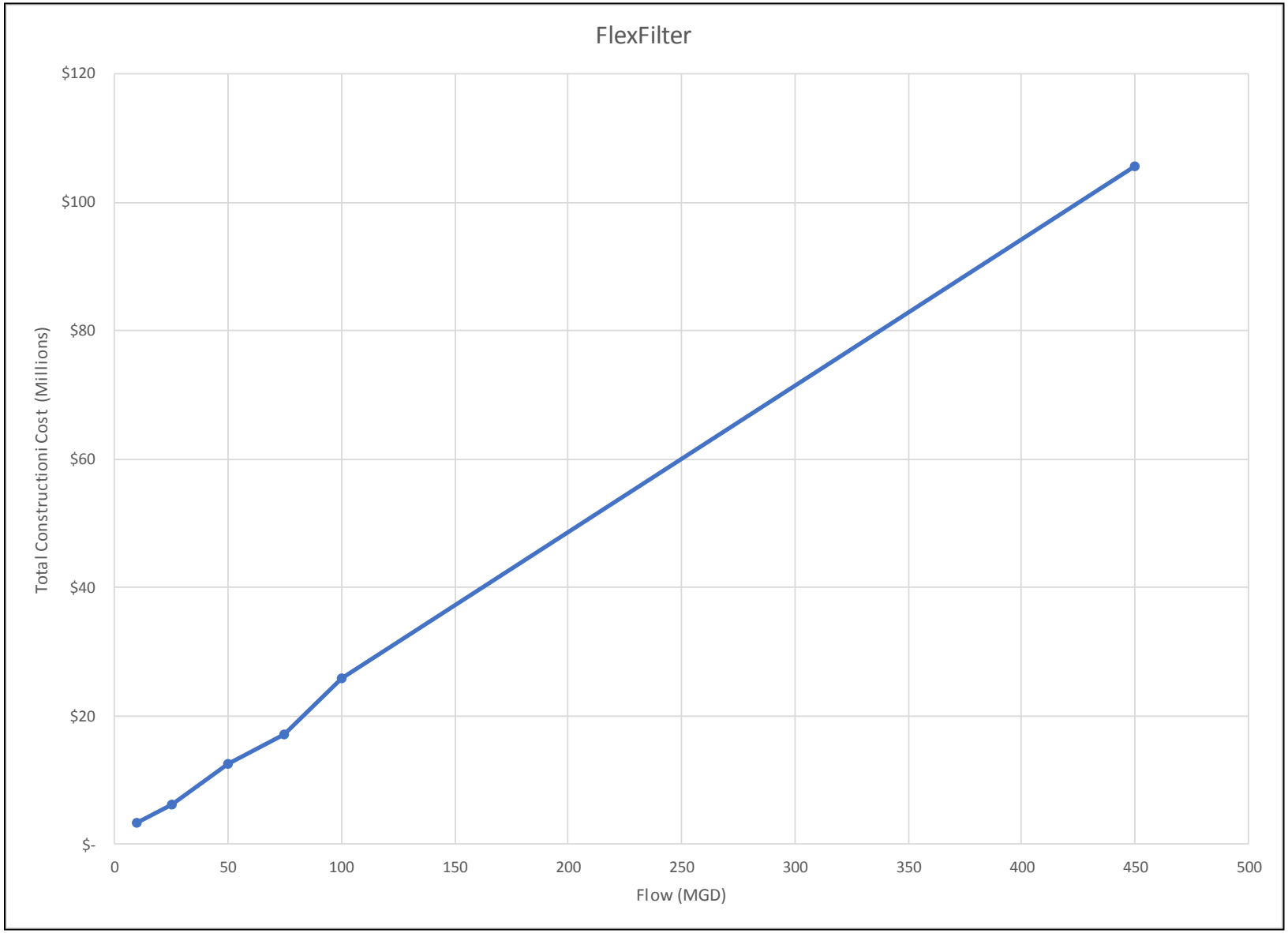


Table 2-25 - Annual Operation and Maintenance Cost of FlexFilter

Flow	Blower Power (kw-hr/MG Treated)	Blower Energy Costs(1)(2)	Media Addition after 10 yrs(3)	Event Labor	Preventative O&M	Backwash & Recycle Pumping	Effluent Pumping	Total Annual O&M
10 MGD	47	$700	$2,254	$20,000	$800	$703	$879	$25,336
25 MGD	48	$1,750	$5,636	$20,000	$2,000	$1,758	$2,198	$33,342
50 MGD	50	$3,500	$7,326	$20,000	$2,400	$2,110	$2,637	$37,973
100 MGD	48	$5,250	$22,542	$20,000	$8,000	$7,033	$8,791	$71,616
200 MGD	53	$7,000	$40,576	$20,000	$16,000	$14,066	$17,582	$115,224
450 MGD	50	$31,500	$72,135	$20,000	$36,000	$31,648	$39,561	$230,844

Notes:
(1) Assumes 500 hours of annual operation
(2) Assumes energy costs of $0.14/kW-hr
(3) Media cost is distributed annually based on given future cost

2.3.4 Evaluation of Pretreatment Technologies

The above process descriptions provide general information on pretreatment processes that may be required for disinfection of CSO discharges. These processes have been evaluated for pretreatment of CSO flow relative to criteria on cost, performance, limitations, and ancillary facilities. Each process was rated from 1 to 5, with 5 being the highest, for approximately twenty different items and totaled. While somewhat subjective, this method does provide a mechanism for comparing each pretreatment process in relationship to each category and subcategory. The results of the evaluation are illustrated in Table 2-26.

Based upon the evaluation results in Table 2-26, the SANSEP process has the highest rating, followed by the ACTIFLO® ballasted flocculation, the DensaDeg® ballasted flocculation, FluidSep vortex units and Storm King®. The Compressible Media Filter received the lowest rating, however this process is used only for polishing the effluent from the other processes in the most stringent treatment objective.

For the vortex/swirl process, the performance of the Storm King® and FluidSep vortex units are essentially the same, but the construction cost of the FluidSep is significantly less, due to the limited use of fabricated metal components, as compared to the Storm King® Unit.

For the ballasted flocculation processes, a similar simplification is possible. The ACTIFLO® process produces less sludge than the DensaDeg® process requiring less ancillary tankage, no cyclone separator and no sand replacement.

Table 2-26 - Evaluation of Pretreatment Technology

Criteria	Vortex Separator		Modified Vortex	Ballasted Flocculation		Polishing Filter
	Fluidsep Vortex	StormKing Vortex	SANSEP	ACTIFLO® Ballasted Flocculation	DensaDeg® XRC Ballasted Flocculation	FlexFilter
Applicability	5	5	4	4	4	2
Performance						
TSS	3	3	5	5	5	5
Hydraulics	3	3	4	3	3	1
Wastestreams	1	1	4	3	3	2
Complexity	5	5	4	3	3	1
Limitations	2	2	4	4	3	3
Construction Cost	4	2	5	3	3	1
Operations	4	4	4	2	2	1
Maintenance	4	4	4	2	2	1
Space Requirements	3	3	3	4	4	2
Requiring:						
Ancillary Tanks	1	1	4	3	3	5
Total	35	33	45	36	35	24

2.4 Disinfection

This section evaluates the implementation of the following chemical and physical disinfection technologies:

- Chlorination (consisting of Chlorine Dioxide, Sodium Hypochlorite, and Calcium Hypochlorite)
- Peracetic Acid
- Ultraviolet (UV) Disinfection
- Ozonation

The evaluation will consist of a description of the particular disinfection technology, the concentrations or intensities normally needed and the equipment or process used to apply the disinfectant. The evaluation will also discuss any limitations of the process or equipment. Also considered in the evaluation will be any inhibiters that will interfere with the disinfection process, and the need for any for dechlorination. The analysis will also consider the safety of the process and the availability of the chemicals or the equipment to produce them.

Disinfection is more difficult to design and operate in CSO applications than in wastewater treatment plants due to the complex characteristics of CSOs. The flowrates of CSOs are highly variable which makes it difficult to regulate the addition of disinfectant. The concentration of suspended solids is high and the temperature and bacterial composition varies widely. Pilot studies are commonly conducted to characterize the range of conditions that exist for a particular area and the design criteria to be considered.

In the cases of chemical addition; chlorine dioxide, sodium hypochlorite, calcium hypochlorite, and peracetic acid, the disinfectant must be mixed with the liquid to be disinfected. Experience has shown that the long contact time required for conventional wastewater treatment is not appropriate for the treatment of CSOs; however, chemical disinfection of CSOs can be accomplished using high-rate disinfection. High-rate disinfection is defined as employing high-intensity mixing to accomplish disinfection within a short contact time, generally five minutes. For this TGM, a chemical induction flash mixer, such as manufactured by The Mastrr Company, will be used to mix either the gas or liquid with the flow to be disinfected. The mixer develops a "G" value of 1,000/sec. The detention time in the mixing zone of the mixer is 3 seconds. Following the mixer, a tank area with a detention time of 5 minutes at the design rate, will be used to provide adequate mixing. In the case of sodium hypochlorite and calcium hypochlorite, a second induction mixer will be used to mix the dechlorination chemicals, sodium bisulfite, with the flow before discharging to the receiving water. No tankage would be provided following the addition of dechlorination chemicals.

The efficiencies of virtually all the disinfection processes being considered in this TGM are dependent upon the TSS concentration of the liquid being disinfected. The required TSS concentration for each of the disinfection processes for different treatment objectives is shown in

Table 2-27.

Table 2-27 - Maximum TSS Concentration for Each Disinfection Process

Fecal Coliform Objectives (MPN/100ml)	Maximum TSS Concentration (mg/L)			
	Chlorine Dioxide	Sodium Hypochlorite	Peracetic Acid	Ultraviolet Disinfection
200	70	45	70	25
770	70	45	70	25
1,500	70	45	70	25

2.4.1 Chlorine Dioxide

Process Description

Chlorine dioxide (ClO_2) is most commonly used for drinking water treatment to oxidize reduced iron, manganese, sulfur compounds, and certain odor-causing organic substances in raw water. Chlorine dioxide is often used as a pre-oxidant because, unlike chlorine, it will not chlorinate organic compounds and therefore will not react with organic matter in the water to form trihalomethanes (THMs) or other byproducts. In industrial markets, chlorine dioxide has been most readily used in the paper and pulping industry. In this application, chlorine dioxide is used as bleach for paper pulp since it does not react with the organic lignin in the wastewater to form by-products such as the THMs.

The data for chlorine dioxide shows that it is a more effective disinfectant than sodium hypochlorite. However, chlorine dioxide needs to be generated on site because it is too unstable even for short periods of time. There is one type of chlorine dioxide generator that utilizes hydrochloric acid and sodium chlorite in either commercially available or diluted concentrations to generate chlorine dioxide. They produce chlorine dioxide and consistently maintain a product yield greater than 95%, making it ideal for drinking water treatment. The use of chlorine gas is not required when using these systems. These systems produce relatively small amounts of chlorine dioxide for disinfection in water systems where low concentrations of ClO_2 are needed.

There is a second process, which produces "large quantities" of gas for disinfection of drinking water and wastewater. This is the Ben Franklin™ process, manufactured by CDG Environmental, LLC. The Ben Franklin™ process uses the chemical reaction of hydrochloric acid with sodium chlorate to generate chlorine dioxide to produce a mixture of chlorine and chlorine dioxide, both in the gas phase. These gases, as produced by the Ben Franklin™ generator, may be applied directly to water as a combination, or they may be separated and applied at different points in the water treatment process. In its most direct application, the mixed chlorine/chlorine dioxide product can be injected into the water to be treated. The result is a mixed disinfectant containing chlorine dioxide and chlorine. The chlorine dioxide acts as a very rapid disinfectant/oxidant while the

chlorine persists longer. This can be an advantage in the water systems where a residual is desired but a disadvantage in the receiving water where disinfection byproduct is a concern.

The use of chlorine dioxide in wastewater disinfection has been very limited in US. Technologies are currently unavailable to provide an easier and safer way to produce chlorine dioxide at a concentration for CSO treatment at remote satellite locations. Chlorine dioxide is extremely unstable and explosive and any means of transport is potentially hazardous. Chlorine dioxide can produce potentially toxic byproducts such as chlorite and chlorate. Chlorine dioxide will not be considered further.

2.4.2 Sodium Hypochlorite

Description of Process

Hypochlorite is a commonly used disinfectant in water and wastewater treatment and has been applied as a CSO disinfectant. It can be produced on site or can be delivered in tanker trunks with concentrations between 3 to 15% of available chlorine. Hypochlorite decays over time. The decay rate can increase as a result of exposure to light, time, temperature increase or increased concentration of the compound. The solution can be stored for 60 to 90 days before the disinfecting ability degrades below recommended values (5% concentration). Degradation of the solution over time is a major disadvantage of sodium hypochlorite for CSO applications, due the variability of the size and frequency of rain events. There are two types of hypochlorite: Sodium hypochlorite (NaOCl) and Calcium hypochlorite ($Ca(ClO)_2$). Sodium hypochlorite is often referred to as liquid bleach or soda bleach liquor, while Calcium hypochlorite is manufactured either as a grain or powder under various names, and all have either approximately 35% or 65% available chlorine content. Sodium hypochlorite is the most widely used of the hypochlorites for potable water and waste treatment purposes. Although it requires much more storage space than high-test calcium hypochlorite and is costlier to transport over long distances, it is more easily handled and gives the least maintenance problems with pumping and metering equipment. It will be used as the basis for evaluating disinfection alternatives.

Based on molecular weight, the amount available as chlorine is 0.83 lbs/gal for a 10% solution of sodium hypochlorite and 1.25 lbs/gal for a 15% solution.

Required Concentrations

The application of sodium hypochlorite as a disinfectant was studied by the USEPA in Syracuse, New York. An equation was developed to estimate the chlorine concentration needed to achieve a particular log-kill of fecal coliform. The parameters included in the equation include the pH of the liquid, the influent fecal coliform count to the disinfection process, the TSS concentration, and the mixing factor of GT. The equation is as follows:

Log-kill = (0.08C^0.36) * (GT^0.42) * (SS^-0.07) * (FC^0.02) * (10^(-0.03pH))

Where:
C = concentration of disinfectant (mg/L as Cl_2)
SS = concentration of SS (mg/L)
FC = Influent level of fecal Coliform, (counts/100 ml)
pH = pH
GT = mixing intensity x detention time.

This is based upon the G of 1000 discussed above, and a three second detention time in the mixing zone of the mixer.

Computations done using this equation, for the range of parameters expected in CSO waters, indicate that a chlorine concentration of between 18-24 mg/L will disinfect the fecal coliform concentrations to the levels expected in the LTCP treatment objectives.

Equipment Needed

Sodium hypochlorite is delivered to the site in liquid form as either a 10% or 15% solution. The sodium hypochlorite is stored in a tank and is fed into a rapid induction type mixer at a rate established by the flow, through a chemical feed pump. A 12.5% solution may degrade to 10% in 6 to 8 weeks, in which case the degradation rate slows. Typically it is stored as a 5% solution of available chlorine. It should be stored at temperatures below 85 degrees Fahrenheit in a corrosion resistant tank and protected from light exposure. For the purpose of this TGM, the chemical storage is estimated to store enough chemical for 24-hours of continuous treatment at the design overflow rate plus a safety factor of 1.5.

The chemical storage tank and the feed pump would be stored in a building with the induction mixer installed in a channel, followed by a detention tank with a 5-minute detention time, as described at the beginning of this section.

Limitations

One of the problems with sodium hypochlorite is that the solutions are vulnerable to a significant loss of available chlorine in a few days. This is described as the shelf life of the chemical. The stability of hypochlorite solutions is greatly affected by heat, light, pH, and the presence of heavy metal cations. The higher the concentration, and the temperature the higher the deterioration. A 15% solution will deteriorate to half strength in approximately 120 days. A 10% solution will take approximately 220 days.

The limited shelf life of sodium hypochlorite makes it difficult in an intermittent application like a CSO to ensure that the correct amount of disinfectant is being introduced into the waste stream. This can lead to under or over disinfecting, which can make it difficult to achieve the required treatment objective.

Inhibitors

High TSS concentrations would be an inhibitor to disinfection using sodium hypochlorite, primarily by shielding the fecal Coliform from the disinfectant.

Need for Dechlorination

The use of chlorine disinfection of wastewater can result in several adverse environmental impacts especially due to toxic levels of total residual chlorine in the receiving water and formation of potentially toxic halogenated organic compounds. Chlorine residuals have been found to be acutely toxic to some species of fish at very low levels. Other toxic or carcinogenic chlorinated compounds can bioaccumulate in aquatic life and contaminate public drinking water supplies. For this reason, excess chlorine must be dechlorinated. Gaseous sulfur dioxide, liquid sodium bisulfite, sodium thiosulfate, sodium sulfite, and sodium metabisulfite can be used for this purpose. Sodium bisulfite

is the most commonly used chemical for dechlorination due to the ease of handling, fewer safety concerns, economic reasons, and availability. For this TGM the use of sodium bisulfite is assumed. Typical characteristics are shown in the Table 2-28 below. Sodium bisulfite can decay about 40 % over a period of six-months. The storage should consider the release of sulfur dioxide when the sodium bisulfite is stored in a warm environment; a water scrubber is typically used to diffuse and dissolve off-gas. Another operational problem is the crystallization of sodium bisulfite when the temperature drops below the saturation point: -6.7^{0}C for 25% solutions and 4.4^{0}C for 38% solutions.

Table 2-28 - Sodium Bisulfite Key Properties

Property	Value
Concentration	38% (25% solutions)
Molecular Weight	104.06
Boiling Point	> 100˚C
Freezing Point	-12˚C
Saturation Temperature	4.4˚C @ 38%
Vapor Pressure	78 mm Hg @ 37.7˚C
Specific Gravity	1.36 @25˚C
pH	3 to 4
Solubility in water	Completely

Sodium bisulfite could be stored indoors in a conditioned building to minimize the degradation due to high temperature and sunlight exposure. To minimize the potential of chemical interaction the storage tanks of sodium hypochlorite and sodium bisulfite have to be isolated from each other.

A rapid induction mixer located in a channel downstream of the contact chamber, as described earlier in this section will accomplish the mixing of sodium bisulfite. Since the Dechlorination process is essentially instantaneous, no contact chamber is required downstream of the injection.

Costs

The costs for the sodium hypochlorite disinfection system include several components including chlorine contact tank, the chemical storage facility for sodium hypochlorite and sodium bisulfite, pumping system for disinfection and dechlorination, mixers, piping and storage tanks.

The preliminary report level construction cost estimates provided in Table 2-29 include the equipment, installation, building, and contingency for a sodium hypochlorite disinfection system of design flow ranging from 10 MGD to 450 MGD. Budgetary equipment pricing information was gathered from equipment manufacturers.

Operation and Maintenance

Operating costs for hypochlorite disinfection systems consist of the power and chemical costs. Power costs are based upon the horsepower of the metering pumps and rapid mixers. Chemical costs are based on usage of sodium hypochlorite and sodium bisulfite.

The equipment would be housed in a building; therefore, maintenance costs consist of labor costs for housekeeping of the building, preventative and corrective maintenance of the mechanical equipment including the chemical metering pumps, mixers, and other appurtenances, and restocking of the chemicals. The chlorine contact tanks will also need periodic maintenance to clean debris.

Estimated annual operation costs for the hypochlorite disinfection system are presented on Table 2-30 containing factors for calculation of operating costs; while estimated annual maintenance labor cost including cost factors are included on

Table 2-31.

Space Requirements

The space requirements of the facilities required for disinfection using sodium hypochlorite are based upon the size of the mixing chamber/tank size for chlorination, the chemical building size for chlorination and de-chlorination, the size of the mixing chamber for de-chlorination, and a buffer of 5 feet around each.

Table 2-29 - Preliminary Construction Cost for Chlorination Systems

Flow	Chlorine Contact Tank Cost	Building Cost	Hypochlorite Pump System and Apprt. Cost	Bisulfite Pump System and Apprt. Cost	Hypochlorite Storage Tank Cost	Bisulfite Tank Cost	Mixer and control valves Cost
10 MGD	$125,000	$156,475	$28,000	$16,450	$21,495	$7,900	$150,000
25 MGD	$310,000	$336,159	$35,700	$16,450	$44,990	$8,495	$200,000
50 MGD	$620,000	$507,778	$49,000	$19,250	$97,485	$10,685	$380,000
75 MGD	$930,000	$681,742	$50,750	$19,250	$129,980	$13,183	$450,000
100 MGD	$1,240,000	$820,039	$61,250	$27,300	$162,475	$13,483	$550,000
450 MGD	$5,580,000	$3,883,107	$231,000	$105,000	$779,880	$50,872	$2,000,000

Flow	Installation Cost(1)	GC General Conditions (2)	GC OH&P(3)	Contingency(4)	Total
10 MGD	$757,980	$126,330	$126,330	$757,980	$2,273,939
25 MGD	$1,427,690	$237,948	$237,948	$1,427,690	$4,283,071
50 MGD	$2,526,297	$421,050	$421,050	$2,526,297	$7,578,891
75 MGD	$3,412,357	$568,726	$568,726	$3,412,357	$10,237,072
100 MGD	$4,311,820	$718,637	$718,637	$4,311,820	$12,935,461
450 MGD	$18,944,788	$3,157,465	$3,157,465	$18,944,788	$56,834,364

Notes:
(1) Installation costs are estimated at 150% of the equipment cost.
(2) GC general conditions are estimated at 10% of the total direct cost.
(3) GC OH&P are estimated at 10% of the total direct cost.
(4) 50% of contingency is used for the planning level of cost estimates.

Table 2-30 - Annual Operation Cost for Hypochlorite Disinfection

Flow	Sodium Hypochlorite Metering Pump(8)	Sodium Bisulfite Metering Pump(8)	Total HP	Total Power (kW)(1)	Annual Energy Usage (kW-hr)(2)	Annual Power Cost(3)	Sodium Hypochlorite Usage (lbs)(4)	Sodium Bisulfite Usage (lbs) (5)	Sodium Hypochlorite Cost(6)	Sodium Bisulfite Cost(7)	Total Annual Cost
10 MGD	1.5	0.5	2	1	746	$104	39,986	8,693	$19,993	$17,385	$37,483
25 MGD	2	0.5	2.5	2	932	$130	99,966	21,732	$49,983	$43,464	$93,577
50 MGD	5	1	6	4	2237	$313	199,932	43,464	$99,966	$86,927	$187,206
75 MGD	7.5	1	8.5	6	3169	$444	299,898	65,195	$149,949	$130,391	$280,784
100 MGD	5	1.5	6.5	5	2424	$339	399,865	86,927	$199,932	$173,854	$374,126
450 MGD	25	4	29	22	10813	$1,514	1,799,391	391,172	$899,695	$782,344	$1,683,553

Notes:
(1) HP x 0.7457
(2) Assumes 500 hours of annual operation
(3) Assumes energy costs of $0.14/kW-hr
(4) Assumes a sodium hypochlorite dosage of 23 mg/L
(5) Assumes a sodium bisulfite dosage of 5 mg/L
(6) Assumes a sodium hypochlorite cost of $0.50/lb
(7) Assumes a sodium bisulfite cost of $2/lb
(8) Metering pump HP based on quotations by Pyrz Water Supply Co., Inc.

Table 2-31 - Annual Maintenance Labor Cost of Hypochlorite Disinfection

Frequency	Estimated Man-Hours	Annual Cost
Daily Check	1	$54,750
Weekly Check	4	$31,200
Monthly Check	8	$14,400
Quarterly Clean and Check	12	$7,200
Total Annual Maintenance Cost		$107,550

Notes:
(1) Assumes labor rate of $150/hour

2.4.3 Peracetic Acid Disinfection

Description of Process

Peracetic acid (CH_3CO_3H), also known as PAA, is an organic peroxy compound, which has strong oxidizing properties. In the presence of water (H_2O), it breaks down into a mixture of hydrogen peroxide (H_2O_2) and acetic acid (CH_3CO_2H). The mixture is clear and colorless with no foaming capabilities and has a strong pungent acetic acid (vinegar) odor. PAA is a very strong oxidizing agent and has a stronger oxidation potential than chlorine or chlorine dioxide. It has been used as a bactericide and fungicide in various industries including the food and beverage industries, the textile and pulp and paper industries, as well as smaller, more confined applications, including hospital settings.

The U.S. EPA approved peracetic acid as a primary disinfectant for wastewater in 2007 while PAA has been used to treat wastewater in Europe for over a decade. Since the EPA approval, only a limited number of wastewater treatment plants in the United States have adopted PAA as a primary disinfectant, including a wastewater treatment plant in St. Augustine, Florida that discharges treated flow to environmentally-sensitive wetlands. Case studies have also been conducted at a number of treatment plants including a wastewater treatment plant in Frankfort, Kentucky and the Bayonne MUA pilot study for CSO treatment.

PAA decomposes quickly and its ultimate fate in the environment is the basic molecules of carbon dioxide, oxygen, and water. Toxicity studies were conducted on PAA in the 1980's to evaluate impact of PAA disinfected primary effluent on the bay environment. The study concluded that there was no toxicity impact. The Bayonne MUA pilot study and other studies on PAA disinfection of wastewater did not experience toxicity of residual PAA. However, more studies are still required to prove that residual PAA poses no toxicity to aquatic life.

Solutions of PAA for wastewater disinfection are typically of 10% and 15% concentrations, higher concentrations have issues with stability. The shelf life of PAA is normally 12 months. However, PAA must be stored at the site where it is dispensed, as underground piping is not permitted. PAA are fed using a diaphragm pump with Teflon diaphragms and polypropylene, Teflon materials and degassing heads are recommended for feeding. The product should be fed into the waste stream at an area of good mixing to promote rapid dispersion. It may be introduced continuously or intermittently depending upon the needs of the user.

Required Concentrations

This is an area where more research and investigation needs to be done, particularly as it related to disinfection of CSOs. The application of PAA as a disinfectant was studied in the Bayonne MUA pilot study. PAA disinfection tests were performed with PAA dose of typically 2 to 3 mg/L, but up to 7 mg/L, targeting PAA residual in 1 to 2 mg/L range. The best-defined relationship derived from the study results was that between the applied dose of PAA as normalized by COD present in the wastewater and the log reduction of pathogen indicators. PAA dose of 0.01 mg/L of PAA per mg/L of COD present in wastewater resulted in 3-log reduction of fecal coliforms (on average), with slightly higher effectiveness for E. coli and slightly lower for Enterococci. Increasing the relative dose to above 0.015 mg/L of PAA per mg/L of COD increased log reduction to 4. Further increase of

the PAA dose appeared to have limited effect on further increasing reduction of the bacterial densities, although data in that range are too limited to allow for a firm conclusion.

Equipment Needed

PAA is typically delivered to the site in liquid form as a 12% solution. The PAA is stored in a tank and is fed into a rapid induction type mixer at a rate established by the flow, through a chemical feed pump. The chemical storage tank and the feed pump would be stored in a building with the induction mixer installed in a channel, followed by a detention tank. Pilot testing has determined that the majority of kill happens in the first 10 minutes regardless of the concentration of PAA. Therefore, the contact time required by PAA has been determined to be between 2 and 10 minutes.

Limitations

The use of peracetic acid in wastewater disinfection has been very limited in the US. There is no known application of peracetic acid in CSO disinfection in the US. In addition, the cost of PAA may be of concern largely due to small consumer market worldwide and the limited production capacity. One manufacturer has listed the price per pound between $0.50 and $0.70 in 2008 dollars, which corresponds to between $3 per gallon and $5.50 per gallon depending on concentrations. Use of peracetic acid in CSO locations could also be complicated by a need for on-site storage of the chemical, which requires secondary containment and appropriate safety measures.

Inhibitors

Studies have shown that variations in water quality parameters related to NH3, TSS, COD, dissolved oxygen and pH, did not have significant effect on the performance of PAA and PAA produces negligible disinfection by-products.

Need for Dechlorination

At the time of this TGM, there is no indication that de-chlorination will be required. The short half-life means that PAA is not persistent and rarely needs to be neutralized prior to discharge.

Costs

The Bayonne MUA pilot study presented equipment cost of PeraGreen, INJEXX TM unit for flowrate ranging from 5 MGD to 250 MGD (Figure 2-22). The costs provided include the cost of equipment delivered to the site and are 2017 dollars as well the cost of a contact tank providing three minutes of hydraulic retention time.

Operation and Maintenance

O&M costs were also provided by the Bayonne MUA pilot study to maintain a PAA residual of 0.8-1.0 mg/l in flowrate ranging from 5 MGD to 250 MGD (Figure 2-23).

Figure 2-22 - Equipment Cost for Peracetic Acid System

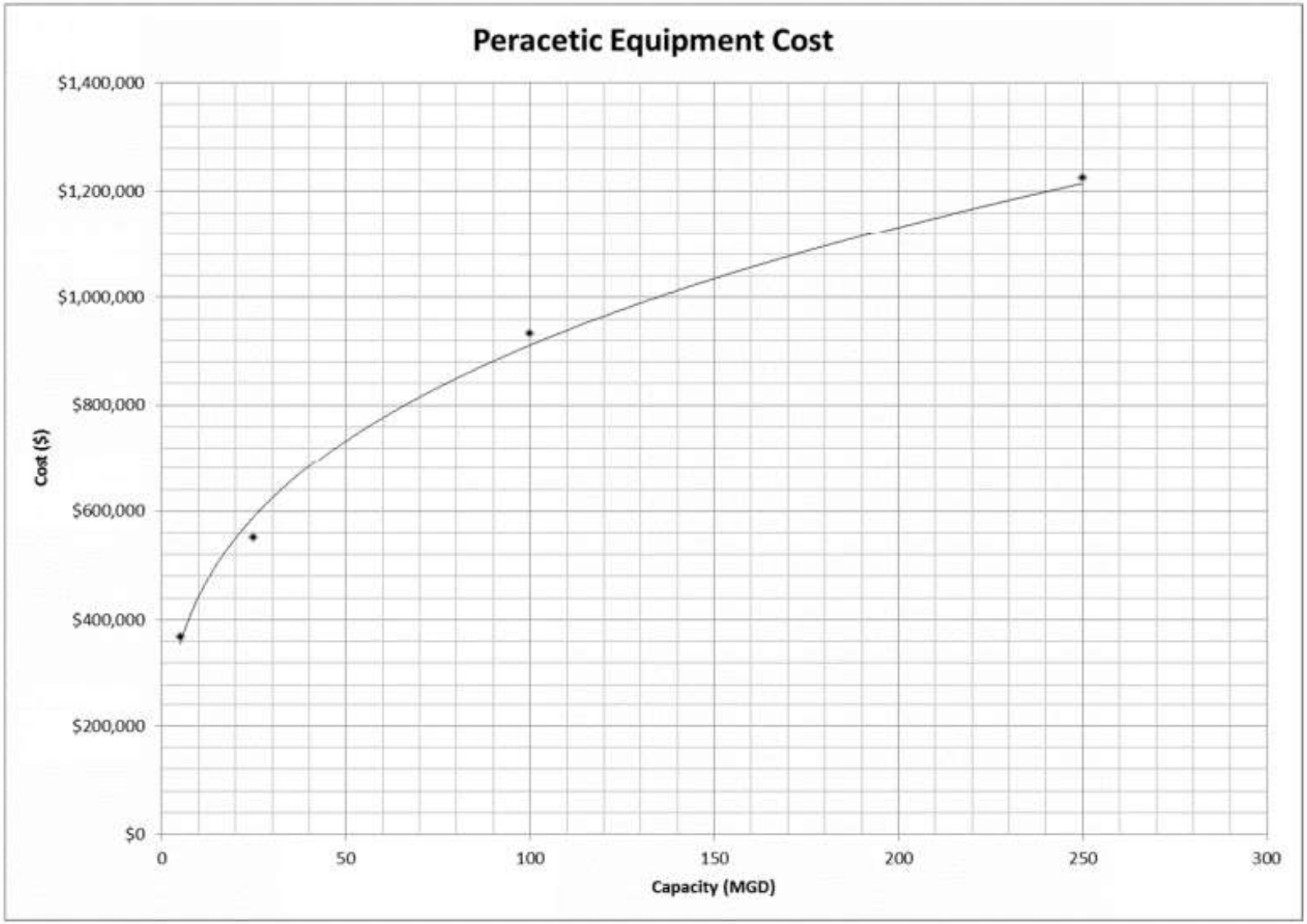
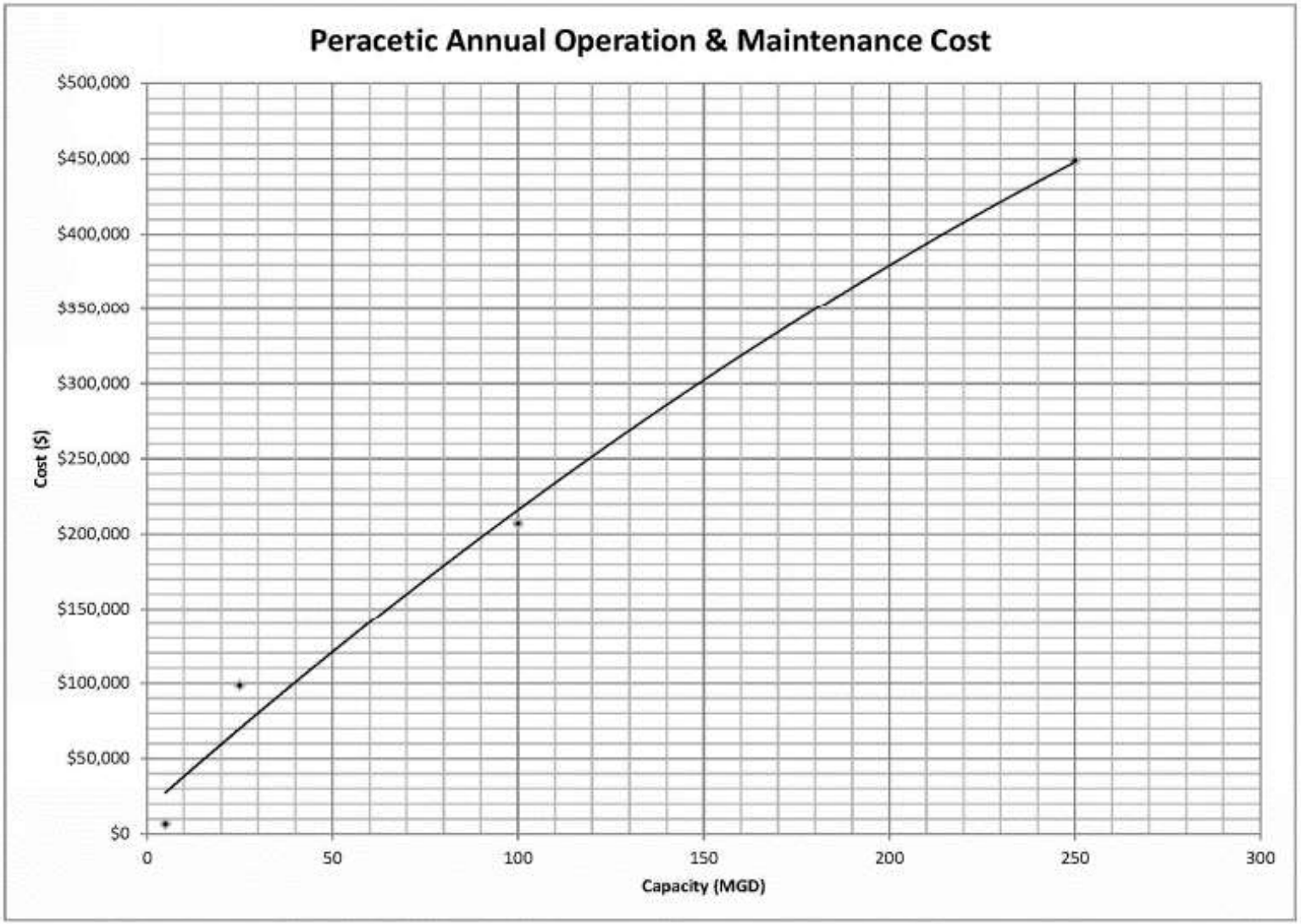


Figure 2-23 - Annual O&M Cost for Peracetic Acid System



2.4.4 Ultraviolet Disinfection

Description of Process

The use of ultraviolet (UV) light is one of the common methods for disinfection of treated wastewaters. In fact, UV disinfection has become the favored technology for new plants and upgrades for existing plants. There are reportedly over 3,500 UV wastewater disinfection systems currently operating in North America, treating flows of up to 300 mgd. UV disinfection eliminates the operational and environmental hazards associated with the use of chlorine compounds, which is a strong oxidant (and sulfite compounds when dechlorination is required), and is cost-competitive with alternative technologies. UV systems are modular and since they require smaller volumes than a chlorination contactor, they can be easily retrofitted into existing chlorination channels.

UV disinfection is a physical process, relying on the transfer of electromagnetic energy released from UV lamps to be absorbed by the nucleic acids (DNA and RNA) in the microorganisms. When the nucleic acids of the organisms are subjected to sufficient quantity of UV radiation (the "dose"), the energy damages the DNA strands by causing specific thymine monomers to combine, which in turn prevents the cell from replicating. This inability to reproduce is, in itself, the lethal effect of UV. Organisms rich in thymine such as *C. parvum* and *G. muris* tend to be more sensitive to UV radiation. The UV radiation in the spectral region between 220 and 320 nm is germicidal, where the wavelengths between 255 nm to 265 nm are considered to be most effective for microbial inactivation. UV disinfection is very effective in inactivation of protozoa, bacteria and viruses, where viruses generally require higher UV radiation dose than protozoa and bacteria.

Electrode type lamps are used to produce light at UV wavelength. Based on the internal operation of these lamps, there are three categories of UV lamps available for use in water/wastewater treatment. These are *low-pressure low-intensity/output (LP-LO)*, *low-pressure high-intensity/output (LP-HO)* and *medium-pressure high intensity/output (MP-HO)* configurations.

In the low-pressure design, lamp output is optimized via mercury vapor pressure and electric current control to generate a broad spectrum of essentially monochromatic radiation in 200nm to 280 nm range (UV-C). Low-pressure lamps produce an intense peak at 254nm which is close to 260nm wavelength considered to be the most effective for microbial inactivation. These low-pressure lamps are highly efficient, converting 30-50% of their input energy to germicidal range of UV light, where 85 – 88 % of this light is at 254 nm. The difference between low-pressure low-intensity and high-intensity lamps are low-intensity lamps use liquid mercury where high intensity lamps use mercury-indium amalgam. Because of this difference, output of LP-LO lamps decreases when the lamp wall is not near optimum temperature of 40°C. LP-HO lamps operate at temperature range of 100 -150°C and can maintain greater stability of lamp output over a wide range of temperatures. In addition, UV output of LP-HO lamps can be modulated between 30 – 100% to adjust the UV dose.

The absolute output of LI-LO lamps is relatively low, with typical UV ratings of 25 to 27 Watts per lamp at 254 nm, for 40 to 100 W input lamps. In LP-HO higher input power (200 to 500 W) have resulted in higher lamp output at 254 nm (60 to 400 W), while retaining their highly efficient energy conversion characteristic.

A number of medium-pressure high-intensity/output UV lamps have been developed over the last decade. MP-HO lamps operate at vapor pressure of 10^2 to 10^4 mm Hg while the low-pressure lamps operating at less than 0.8 mm Hg. Also, the operation temperature of MP-HO lamps are significantly higher (600 – 800°C)_than the LP lamps. With the higher mercury pressures, the lamps are driven at substantially higher input power levels (in the range of 1,000 w to13,000 W). Medium-pressure lamps are polychromatic, effectively radiating 20 to 50 times more the total UV-C output (200 to 280 nm) compared to LP-HO lamps. However, MP-HO lamps have lower efficiency than LP-LO and LP-HO lamps. MP lamps can convert about 7 to 9% of their input power to 254 nm output, and 10 to 15% of the total output is in the germicidal region. Overall, the efficiency of the MP-HO lamps is 4 to 5-fold less than the efficiency of the low-pressure lamps. In addition, the lamp, sleeve and ballast life of MP-HO lamps are significantly lower than LP lamps. However, because of their much higher absolute output levels, fewer lamps are needed, often resulting in a smaller footprint for the UV system.

The actual application of UV to wastewater disinfection is fairly simple. The lamps are enclosed in quartz sleeves (highly transmissible in the UV region), and submerged in the flowing wastewater. The lamp/quartz assemblies are typically arranged in modules, with several modules comprising a bank of lamps. In wastewater applications, these banks of lamps are typically placed in open channels, either horizontally or vertically oriented, with level control devices that maintain water levels above the submergence level of the lamps. Pressure units, using closed-vessel reactors, are also used for wastewaters, although pressure units are more frequently applied in drinking water applications. Generally, automatic cleaning systems/wipers are integrated with each bank of lamps to periodically clean the surface of the quartz sleeve and prevent fouling of the sleeve surface and maintain high transmissivity of the sleeves.

There are many benefits associated with UV disinfection:

1. Since no harmful chemicals are added to the wastewater and no known disinfection byproducts are produced as a result of UV radiation.

2. UV system has a compact footprint and the inactivation of microorganisms occur almost instantaneously as the water passes through the UV lamps. Therefore, UV disinfections systems are set up as a modular system and can be easily configured in one or more channels.

3. Chemical storage, transportation and handling is eliminated for the purpose of disinfection.

UV disinfection does, however, require more power than chemical disinfection, which could be a significant consideration for the larger overflow applications.

Required Concentration

There are several factors that affect the design of a UV system for wastewater disinfection. These center about the design goal to efficiently deliver the necessary UV dose to the targeted microorganisms. Dose is defined as the product of the intensity of UV energy (the rate at which it is being delivered, mJ/cm^2 and the exposure time of the organism to this intensity. Ideally, these factors can be applied such that every element in the water receives the same dose as it passes through the UV unit. However, in practice, the UV dose will not be identical for all particles in the water. There is a variation in the intensity field within the unit and variation in the exposure times,

resulting in a dose distribution. Effective design optimizes this dose distribution and avoids any appearance of hydraulic short circuiting through the UV unit. Exposure time is dependent on the hydraulic characteristics of the unit, reflecting the spacing of the quartz/lamp assemblies, inlet and outlet conditions, and hydraulic loading rates. The output energy of the lamps, the transmissibility of the quartz sleeves, and the transmittance of the wastewater itself affect intensity. The loss of energy due to the aging of the lamps and degradation of the quartz sleeve transparency must be incorporated in the design of the UV units. Generally, the lamp output will decrease to between 50% and 80% of their nominal output by the end of lamp life (typically LP-HO lamps have 9,000 to 15,000 hours and MP-HO lamps have 3,000 to 8,000 hours lamp life). Sleeve fouling will typically account for a 20% to 30% decrease in transparency through the life of the quartz sleeve, even if they get cleaned regularly. The transmittance of treated wastewater effluents will range between 50% and 75%, depending on the influent water quality and the degree of treatment provided before disinfection. Combined sewer overflows and storm water have significantly low UV transmittances and it is generally in the range of 20% to 50% per cm at 254 nm. Since this directly affects the portion of the energy from UV lamps reaching the microorganism, design should call for closely spacing the lamps and using higher-powered lamps. The medium-pressure lamp units can meet these criteria, as can the LP-HO lamp technologies, although to a lesser degree. Head losses are generally manageable for these systems, typically in the order of 6 to 24 inches for the medium-pressure units. Typically, a dose of 30 to 40 mJ/cm^2 is specified for treated wastewater disinfection, where three to four log inactivation rates are generally required to meet disinfection targets. Demonstration that the proposed unit will deliver this dose under design conditions (flow, UV transmittance, end-of-lamp life output, degraded quartz surfaces, etc.) is often required either as a prequalification for bidding, or at the time of commissioning. This is done through direct bio-dosimetric testing on full-scale or scaled systems, whereby a challenge organism of known dose-response is injected into the UV unit under design flow and UV transmittance conditions. By measuring the kill of the organism, the dose that was delivered by the unit can be estimated. This method has become an industry standard for validating the performance of UV systems. These protocols are articulated by the USEPA UV Design Guidance Manual (November 2006), the NWRI/AWWA RP UV Guidance (May 2003), and the USEPA Environmental Verification Program protocols for reuse, secondary effluents, and wet weather flows (2002). This method accounts for the variations in hydraulics through the UV lamps and UV radiation intensity in a system, and allows for a more consistent comparison of performance expectations and design sizing between different UV technology configurations.

The Bayonne MUA pilot study evaluated performance of Trojan UV3000Plus unit using low-pressure lamps. Correlation of all the individual data from the study indicated required approximately 25 mJ/cm^2 effective irradiation dose input to achieve 3log inactivation of pathogen indicators.

Equipment Needed

For purposes of this preliminary assessment of cost associated with the disinfection of combined sewer overflows, the low-pressure high intensity lamp technology is considered. As discussed earlier, the LPHO lamps are very efficient and with advancement in UV lamp technology, there are up to 1,200 W lamps available. The Sigma low-pressure high-intensity lamps offered by Trojan

Technologies has been used for preliminary sizing, layout, design and costs estimation; however, it is not the intent of this exercise to recommend a given manufacturer for such applications.

Limitations

In large applications, significant power is required for operation of UV system. In some locations power availability can be a limitation.

Inhibitors

Certain water quality parameters can have a big impact on the disinfection efficiency of the UV system. UV transmittance or UV absorbance is one the key parameter which impact the UV dose that the microorganisms get subjected to. Iron, ozone, manganese, natural organic matter (NOM), TSS are strong absorbers of UV light, which would reduce the UV transmittance. The threshold values for Ferric iron, Ferrous iron and ozone are set as 0.057 mg/L, 9.6 mg/L and 0.071 mg/L, respectively. If iron salts are used within the treatment process, alternative should be evaluated to compare savings of smaller UV system compared to cost associated with change of precipitation aid. Alkalinity, hardness (Ca, Mg and other salts) and TDS can form mineral deposits on quartz tubes and reduce the UV dose reaching microorganisms and would increase the frequency and sleeve cleaning. Alkalinity and pH also effect the solubility of metals carbonate which may absorb UV light. Oil and grease in the wastewater would accumulate on the quartz sleeves and reduce the UV transmittance.

Need for De-chlorination

Since no chemical is used in UV disinfection and there is no residual disinfectant in the wastewater due to UV disinfection, de-chlorination or residual disinfectant removal is not required in UV disinfection systems. If any chemical disinfectant is added in upstream of the UV disinfection, residual disinfectant removal may be required specific to chemical disinfectant used.

Costs

The costs for the ultraviolet disinfection system consist of the equipment cost, including its installation, the cost of the channels for the ultraviolet disinfection equipment.

The preliminary report level construction cost estimates provided in Table 2-32 include the equipment, installation, building, and contingency for UV disinfection system of design flow ranging from 10 MGD to 450 MGD. Budgetary equipment pricing information was gathered from equipment manufacturers.

Operation and Maintenance

UV disinfection systems have been used for continuous operation for many years at various treatment facilities. Routine operating and maintenance programs and guidelines have been established for these continuous operations. However, in the case of CSO discharges, the O&M requirements for the UV disinfection technology would be intermittent during the year and be based on the number of storm events per week, month or year. The CSO locations at remote sites would require field crews to be on site before a storm event to make sure the system is in operating conditions and after the storm event to perform general washdowns and maintenance check.

The O&M requirements would center on lamp cleaning, parts replacement, and general maintenance. Recent applications of UV lamps have cleaning systems that employ chemically-

assisted mechanical wipers, which are effective for low-grade wastewater applications such as CSOs. This has significantly reduced labor time required for lamp cleaning and has also improved lamp effectiveness. However, one of the main challenges with CSO systems is that the lamps are not always submerged in the water and when there is long period between storm events, dust will accumulate on the sleeves. These dust particles would scratch the surface of the sleeve and reduce the penetration/transmittance of the UV light. Therefore, additional precaution and manual cleaning would be required from time to time. It is recommended that UV banks would be raised and inspected for debris after each event to ensure that there is not large debris caught up in the system. The wipers have a debris scraper that will handle smaller debris and push it out of the way, but it will be a good practice to inspect the equipment after each event.

Parts replacement is another major maintenance requirement and would include the replacement of lamps, ballasts, wipers and quartz sleeves. Since the UV system is not going to be operating continuously, lamp replacement is not going to be as often as continuously operating systems in wastewater treatment plants. While some manufacturers offer a lamp warranty only for set operation hours ranging from 12,000 hours to 16,000 hours for LP-HO lamps, which equates to 24 to 32 years of warranty for lamps. This long duration of lamp operation is not believed to be reasonable due to operational conditions of CSO systems. On the other hand, some manufacturers provide a warranty based on a set limit of operation hours or a set duration, which occurs first. The output of UV lamps decreases as lamps age. Generally, after 12,000 to 15,000 hours of operation, the lamps need to be replaced due to low power output. In this report, it is assumed that UV lamps would be replaced every 10 years. In addition to lamp replacement, the ballasts, a type of transformer that is used to limit the current to the lamps, will need to be replaced. For the specific brand and model used for cost estimation in this report, each ballast serves 2 lamps and has an expected life of 5 years.

The third major maintenance requirement would be general O&M requirements at the CSO site. General maintenance at each UV disinfection site would include repairs, cleaning the channels and surrounding areas, maintaining product inventories, system monitoring, and documenting site visits. Assuming that there would be a two-person field crew visiting each site for one hour before and after each storm event, the estimated maintenance hours per event would be 4 to 8 hours depending on the system sizes. UV disinfection systems for CSO discharges can be designed to operate intermittently during the year and also during winter conditions.Instrumentation for intermittent disinfection operations would be incorporated into the UV reactor's operation including monitoring CSO flows, CSO characteristics such as UVT and CSO water levels in the reactor and support channel. These controls would be programmed to turn the reactor on and off, increase or decrease the lamps' intensity based on UVT and open appropriate valves to drain the reactor when not in operation. Operations in the winter, however, would include other specific requirements in the reactor for controlling freezing conditions in the reactor. These requirements would include any or all of the following guidelines:

1. Drain the reactor and apply warm air to the module to maintain temperature above 32°F; and

2. Manually drain the cleaning solution from the wipers and refill the wipers before the next storm event (approximately 5 minutes per lamp). Leave the reactor full of water and

provide a heat source to maintain the water temperature above 32°F during freezing temperatures.

Space Requirements

The space requirements of the facilities required for disinfection using UV are based upon the size of the contact chamber and a buffer of 5 feet on upstream and downstream of the UV lamps.

Table 2-32 - Preliminary Construction Cost Estimates for UV Disinfection

Flow	Length x Width X Depth(1)	Budgetary Equipment Price	Concrete Cost(2)	Install Cost(3)	GC General Conditions (4)	GC OH&P(5)	Contingency(6)	Total
10 MGD	4'-0" x 4'-0" x 9'-0"	$300,000	$885,600	$1,778,400	$296,400	$296,400	$1,778,400	$5,335,200
25 MGD	50'-5" x 5'-1" x 9'-0"	$625,000	$1,138,536	$2,645,304	$440,884	$440,884	$2,645,304	$7,935,912
50 MGD	50'-5"x 5'-1" x 9'-0"	$1,100,000	$1,959,552	$4,589,328	$764,888	$764,888	$4,589,328	$13,767,984
75 MGD	53'-5"x 5'-1" x 9'-0"	$1,400,000	$2,076,192	$5,214,288	$869,048	$869,048	$5,214,288	$15,642,864
100 MGD	52'-3" x 4'-10" x 9'-0"	$1,600,000	$2,931,552	$6,797,328	$1,132,888	$1,132,888	$6,797,328	$20,391,984
450 MGD	68'-8" x 8'-11" x 11'-9"	$8,480,000	$12,060,757	$30,811,136	$5,135,189	$5,135,189	$30,811,136	$92,433,408

Notes:
(1) Channel size based on assumed channel size with length of twice the width before and after UV lamp banks, and 1.5 feet of free board for the side walls
(2) Concrete costs based upon assumed $900 per cubic yard
(3) Installation costs are estimated at 150% of the equipment cost.
(4) GC general conditions are estimated at 10% of the total direct cost.
(5) GC OH&P are estimated at 10% of the total direct cost.
(6) 50% of contingency is used for the planning level of cost estimates.

Table 2-33 - Annual Operation Cost for Ultraviolet Disinfection

Flow	Total Number of UV Lamps	Power Consumption per Lamp (kW)	Total Power (kW)	Annual Energy Usage (kW-hr)(1)	Total Cost(2)
10 MGD	32	1	32	16,000	$2,240
25 MGD	66	1	66	33,000	$4,620
50 MGD	132	1	132	66,000	$9,240
75 MGD	176	1	176	88,000	$12,320
100 MGD	240	1	240	120,000	$16,800
450 MGD	1152	1	1152	576,000	$80,640

Notes:
(1) Assumes 500 hours of annual operation
(2) Assumes energy costs of $0.14/kW-hr

Table 2-34 - Annual Maintenance Cost for Ultraviolet Disinfection

		Annual Number of Units Replaced			
Flow	**Lamps**	**Lamps(1)**	**Ballasts(2)**	**Sleeves(3)**	**Wipers(4)**
10 MGD	32	3	3	6	16
25 MGD	66	7	7	13	33
50 MGD	132	13	13	26	66
75 MGD	176	18	18	35	88
100 MGD	240	24	24	48	120
450 MGD	1152	115	115	230	576

	Annual Maintenance Labor Costs (5)						
	Lamps	**Ballasts**	**Sleeves**	**Wipers**	**Check UV Sensors(6)**	**Routine(7)**	**Total Annual Labor**
Estimated Man Hours per Unit	*0.25*	*0.25*	*1*	*1*	*2*	*4 to 8*	
10 MGD	$150	$150	$1,050	$2,400	$7,800	$60,000	$71,550
25 MGD	$300	$300	$2,100	$4,950	$7,800	$60,000	$75,450
50 MGD	$600	$600	$4,050	$9,900	$7,800	$75,000	$97,950
75 MGD	$750	$750	$5,400	$13,200	$7,800	$90,000	$117,900
100 MGD	$900	$900	$7,200	$18,000	$7,800	$90,000	$124,800
450 MGD	$4,350	$4,350	$34,650	$86,400	$7,800	$120,000	$257,500

	Annual Maintenance Equipment Costs					
	Lamps	**Ballasts**	**Sleeves**	**Wipers**	**Total Annual**	**Total Annual Maintenance**
Unit Costs	*$300*	*$750*	*$175*	*$30*		
10 MGD	$960	$2,400	$1,120	$480	$4,960	$76,510
25 MGD	$1,980	$4,950	$2,310	$990	$10,230	$85,680
50 MGD	$3,960	$9,900	$4,620	$1,980	$20,460	$118,410
75 MGD	$5,280	$13,200	$6,160	$2,640	$27,280	$145,180
100 MGD	$7,200	$18,000	$8,400	$3,600	$37,200	$162,000
450 MGD	$34,560	$86,400	$40,320	$17,280	$178,560	$436,060

Notes:
(1) Assumes lamps replaced every 10 years
(2) Assumes ballasts replaced every 5 years
(3) Assumes sleeves replaced every 5 years
(4) Assumes wipers replaced every 2 years
(5) Assumes labor rate of $150/hour
(6) Assumes UV sensors are inspected bi-weekly
(7) Routine inspection and maintenance should be performed after each event with 4hr for 10MGD and 25 MGD system, 5 hours for 50 MGD System, 6 hours for 75MGD and 100 MGD systems, and 8 hours for 450 MGD system. Assumed 100 events.

2.4.5 Ozone Disinfection

Description of Process

Ozone (O_3) is an unstable gas that is produced when oxygen molecules are dissociated into atomic oxygen and subsequently collide with another oxygen molecule to produce ozone. Due to the instability of ozone, it must be generated on-site from air or oxygen carrier gas. The most efficient method of producing ozone today is by the electric discharge technique, which involves passing the air or oxygen carrier gas across the gap of narrowly spaced electrodes under a high voltage. Due to this expensive method of producing ozone, it is extremely important that the ozone is efficiently transferred from the gas phase to the liquid phase. The two most often used contacting devices are bubble diffusers and turbine contactors. With the bubble diffusers, deep contact tanks are required. Ozone transfer efficiencies of 85% and greater can be obtained in most applications when the contactor is properly designed. The contactors must be covered to control the off-gas discharges. Since any remaining ozone would be extremely irritating and possibly toxic, the off-gases from the contactor must be treated to destroy the remaining ozone. Ozone destruction is normally accomplished by thermal or thermal-catalytic means.

An ozonation system can be considered to be relatively complex to operate and maintain compared to chlorination. The process becomes still more complex if pure oxygen is generated on site for ozone production. Ozonation system process control can be accomplished by setting an applied dose responsive to wastewater flow rate (flow proportional), by residual control, or by off-gas control strategies. Ozone disinfection is relatively expensive with the cost of the ozone generation equipment being the primary capital cost item, especially since the equipment should be sized for the peak hourly flow rate as with all disinfectant technologies. Operating costs can also be very high depending on the power costs, since Ozonation is a power intensive system.

Since ozonation is expensive to operate, and maintain, produces off-gas that can be toxic, is a complex system, and not utilized for disinfection at wastewater treatment plants where flow is more controlled and less variable, we feel it is not an acceptable application for disinfection of CSO flows and will not be evaluated further.

2.4.6 Evaluation of Disinfection Technologies

The above sections evaluated each of the disinfection technologies considered for treatment of CSO flow relative to criteria on cost, performance, limitations, and ancillary facilities. Each process was rated from 1 to 5, with 5 being the most effective, for approximately twenty different items and totaled. While somewhat subjective, this method does provide a mechanism for comparing each screening unit in relationship to each category and subcategory. The results of the evaluation are illustrated on Table 2-35.

Table 2-35 presents the relative effectiveness of the different disinfection technologies with respect to bacteria, viruses, and encrusted parasites. For the purposes of this table the bacteria are identified as pathogens, E. coli, enterococci, and salmonella. Viruses are identified as the polio virus, with encrusted parasites consisting of giardia and cryptosporidium.

Table 2-35 - Evaluation of Disinfection Technologies

Criteria	Sodium Hypochlorite	Peracetic Acid	Ultraviolet Disinfection
Complexity	5	5	2
Safety	4	4	5
Limitations	3	3	3
Inhibitors	3	5	3
De-chlorination Requirement	1	5	5
Commercial Product Availability	5	1	5
CSO Application	5	2	2
Total	26	25	25

Section 3

Storage Technologies

Storage technologies are used to store flow for subsequent treatment at the wastewater treatment facility when downstream conveyance and treatment capacity are available. Two general types of storage need to be considered: in-line storage, which is storage in series with the sewer; and off-line storage, which is storage in parallel with the sewer. More detailed information on each type and sub-type is provided below.

3.1 In-Line Storage

In-line storage is generally developed in two ways. One way would be to use control structures to store the flows from smaller storm events (those below the design storm for the facilities) using the excess pipe capacity within the existing sewer. The other, also used with a control structure, is to replace segments of the existing sewer with larger diameter pipes to act as storage units. In both cases the use of in-line storage typically needs large diameter pipe with flat slopes. In-line storage within the existing combined sewer system is currently provided to some extent by the overflow weir typically used in existing CSO control facilities. Maximizing that storage, selecting the location of other flow control structures, and sizing of these facilities must be determined and verified by using a calibrated and verified hydraulic model.

In-line storage facilities require an extensive control and monitoring network. These includes flow regulators, such as orifices, weirs, flow throttle valves, automated gates and continues monitoring network such as level sensors, rain gages, flow monitors, and overflow detectors. Effective and efficient in-line storage requires the utilization of site-specific information together with modeling data and information on downstream flow elevations and available capacity.

3.1.1 Using Existing Sewers

Existing sewers can sometimes provide additional in-line storage by installing an in-line weir structure or flow regulator within a pipe section or at a manhole. On large diameter sewers, the weir structure would typically consist of an inflatable rubberized fabric dam, which could be pressurized to create an impoundment on the upstream of the regulator and thus create inline storage. Another flow regulator that has been used to develop in-line storage is an automatically controlled sluice gate. Instrumentation is typically provided for automatic control to prevent overloading the system. Sections of pipe utilized for in-line storage should not have any service lateral connections, or should be deep enough to prevent sewage backups within the system.

The storage available in a sewer is directly related to the cross-sectional area of the sewer that is typically unused during typical wet weather events. Typical storage requirements for wet weather flows are in the tens or hundreds of thousands of gallons. A 4-foot (48- inch) diameter circular pipe has a total capacity of less than 100 gallons per foot, a 6- foot (72-inch) pipes has a total capacity of around 210 gallons per foot, while a 6-foot x 12-foot rectangular section has a total capacity of around 540 gallons per foot.

Most combined sewer systems within the region were constructed during the period of 1880 through 1920 when few paved roads and concrete sidewalks and other impervious areas were limited to roofs. Land development, changes within land use, and changes in sewer utilization over the past century have all impacted the flow characteristics of most combined sewer systems. Most of the combined sewer systems within the region have a diameter of 48-inch or less. These sewers are expected to have little or no storage capacity due to increase inflow rates and limited pipe size and slope.

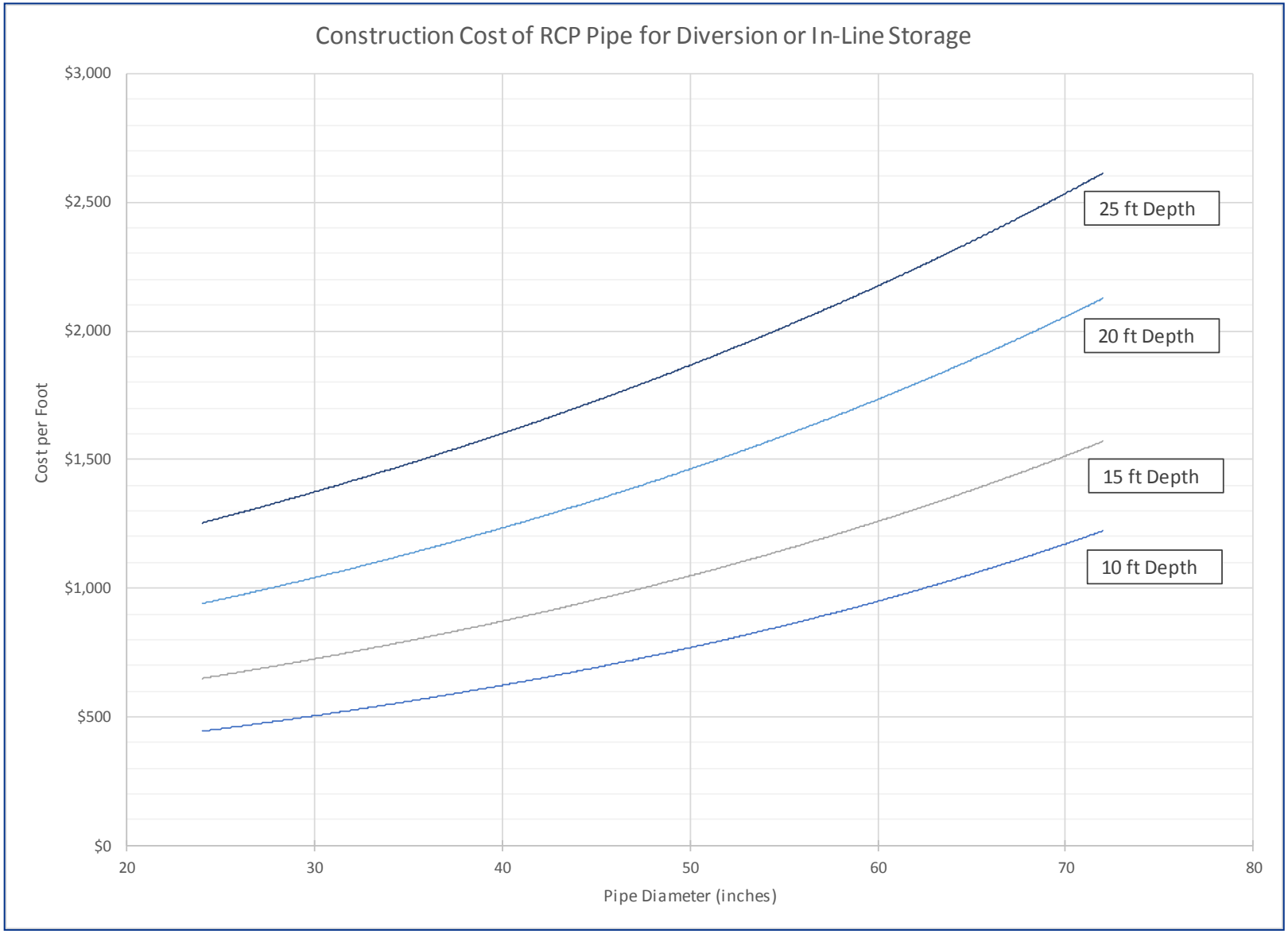
A CSO Facility Plan was completed by Killam Associates (now Mott MacDonald) in 1983 for the Passaic Valley Sewerage Commissioners on the combined sewer systems within the Cities of Newark and Paterson, and Towns of Harrison and Kearny, and the Borough of East Newark. The evaluation of in-line storage was conducted to review the feasibility of inline storage within the region. This study concluded that, with the exception of a few areas within the City of Newark, the volume of inline storage available within the sewer system was insignificant. It is anticipated that in-line storage using existing sewer will not provide a significant volume of storage.

3.1.2 Using New Large Dimension Sewers

In-line storage can also be developed by the construction of new large diameter sewers in place of, or parallel to existing combined sewers. The general principal that governs inline storage in either existing or new sewers are the same. In-line storage developed by replacing segments of the existing combined sewer system with larger diameter pipes still requires extensive controls and monitoring to assure proper operation. Accordingly, the cost of constructing the additional sewer capacity must be determined in addition to the cost of the control and monitoring network.

The original Technical Guidance Manual provided cost information suitable for the preliminary analysis of in-line storage using newly constructed large dimensional sewers in place of existing pipe. Those cost estimates were based on an assumed minimum replacement length of 500 feet for circular conduit sizes varying from 24-inch to 72-inch, and were based on an Engineering News Record (ENR) Construction Cost Index (CCI) of 7630. For this TGM update, that cost information was obtained from those cost curves and escalated to 2017 dollars using the October 2017 ENR CCI of 10817. The resultant cost estimates for the construction of segments of large diameter pipe are provided in Figure 3-1. The cost of the control and monitoring network is site specific, and should also be considered when evaluating the use of in-line storage.

Figure 3-1 - Construction Cost Estimates for RCP Pipe for Diversion or In-Line Storage



3.1.3 System Evaluation

Effective control of in-line storage can be achieved through proper flow regulator equipment and hardware selection, a SCADA system that provides early warning and accurate storm forecast. Seasonal storm patterns and types need to be identified and thoroughly evaluated to assure that the control system can properly handle current and potential rainfall patterns within the drainage area. The cost of implementation is significant for areas with limited existing storage due to the cost and challenges associated with the construction of new sewers especially in urban areas, where the access to sewer can be limited and above ground vehicle and pedestrian traffic is heavier. One advantage of in-line storage is the potential of reducing flooding and other system problems that may be localized within the system.

Operational problems that have been noted include computer programming and hardware problems especially with telemetry or data transmission, which could lead to a loss of accuracy in system control. In addition, deposition of solids in the sewers can occur, since the flow velocity during dry weather can be lower than self-cleansing velocity in large diameter sewers. In areas where smaller diameter sewers are replaced with large diameter sewers to provide in-line storage, consideration should be given to provide a low flow channel within the invert. A thorough analysis should be conducted for the potential of sewage backups in service laterals due to surcharging the system above previous hydraulic grades.

3.2 Off-line Storage

Off-line storage is storing the combined sewage in a storage system that is not on the typical flow path of dry weather flow. Off-line storage systems use tanks, basins, tunnels or other structures located adjacent to the sewer system for storing wet weather flow that is above the capacity of the conveyance system. The wastewater flows from the collection or conveyance system is diverted to off-line storage when conveyance capacity of the collection system has been exceeded. They can be used to attenuate peak flows, capture the first flush, or to reduce the frequency and volume of overflows. Wastewater flows diverted to storage facilities must be stored until sufficient conveyance or treatment capacity becomes available in downstream facilities. Off-line storage is typically accomplished by the construction of storage tanks, lagoons, basins, or deep tunnels.

Off-line storage is the predominant form of CSO prevention method currently in operation throughout the United States. The major advantages of off-line storage include:

- It can accommodate intermittent and variable storms.
- It is not impacted by varying water quality flow characteristics.
- It can accommodate solids deposition and control; and
- Storage tanks are easily accessible.

Off-line storage is not a flow through facility and thus ancillary facilities must be constructed for a complete installation. Ancillary facilities typically include some type of flow diversion or regulator structure, possibly coarse screening to keep large solids from entering the tank, and some type of tank drain facility to divert the sewage back to sewer system. To keep solids from accumulating

within the tank, most storage facilities also provide facilities to flush solids from the bottom of the tanks into the pumping sump or gravity sewer.

Two types of off-line storage are typically used in CSO system depending on the volume of the overflows that need to be captured. The most prevalent form of off-line storage is a concrete storage tank/structure. These tanks/structures can be constructed above or below ground. The second form is the deep tunnel, wherein a large diameter tunnel is constructed to capture and store CSO discharges. While other forms, including uncovered earthen basins, have been used in less populated areas, open forms of CSO storage would not be applicable to highly urbanized areas.

3.2.1 Off-line Storage Tanks

The most prevalent form of off-line storage for CSO discharges is the concrete/steel tank. While large diameter parallel sewers can provide a mechanism for off-line storage, the storage volumes associated with these facilities are limited and thus are typically used within the collection system to prevent or minimized the surcharging associated with local restrictions or conditions. Large volume storage requirements can best be accommodated by the construction of off-line storage facilities at or near the CSO outfall. The design and sizing of these facilities are based upon computer modeling of drainage area and collection system to develop an understanding of the frequency and volumes associated with individual outfalls.

Advantages of off-line storage using concrete tanks are simplicity of operation and maintenance, and capability to handle high flow and water quality variations. In addition, storage tanks have the capacity for storage and collection of solids even when storm events exceed the design capacity of the off-line storage tank. In these cases, the off-line storage tank acts like a sedimentation tank. Storage tanks, in conjunction with fine screening of CSO discharges above the storage volume, are used as a primary means of CSO control throughout Europe.

As with in-line storage, the original Technical Guidance Manual provided cost information for off line storage that was obtained and escalated to 2017 dollars based on the ENC CCI. Those cost estimates were developed for concrete tanks of various storage volumes and are inclusive of all ancillary facilities and include construction costs for coarse screens, diversions, control gates, pumping facilities, flushing facilities and ventilation. The resultant cost curves are presented in Figures 3-2 through 3-4.

Figure 3-2 - Construction Cost Estimates for Off-Line Storage – 15’ SWD Rectangular < 1 MG

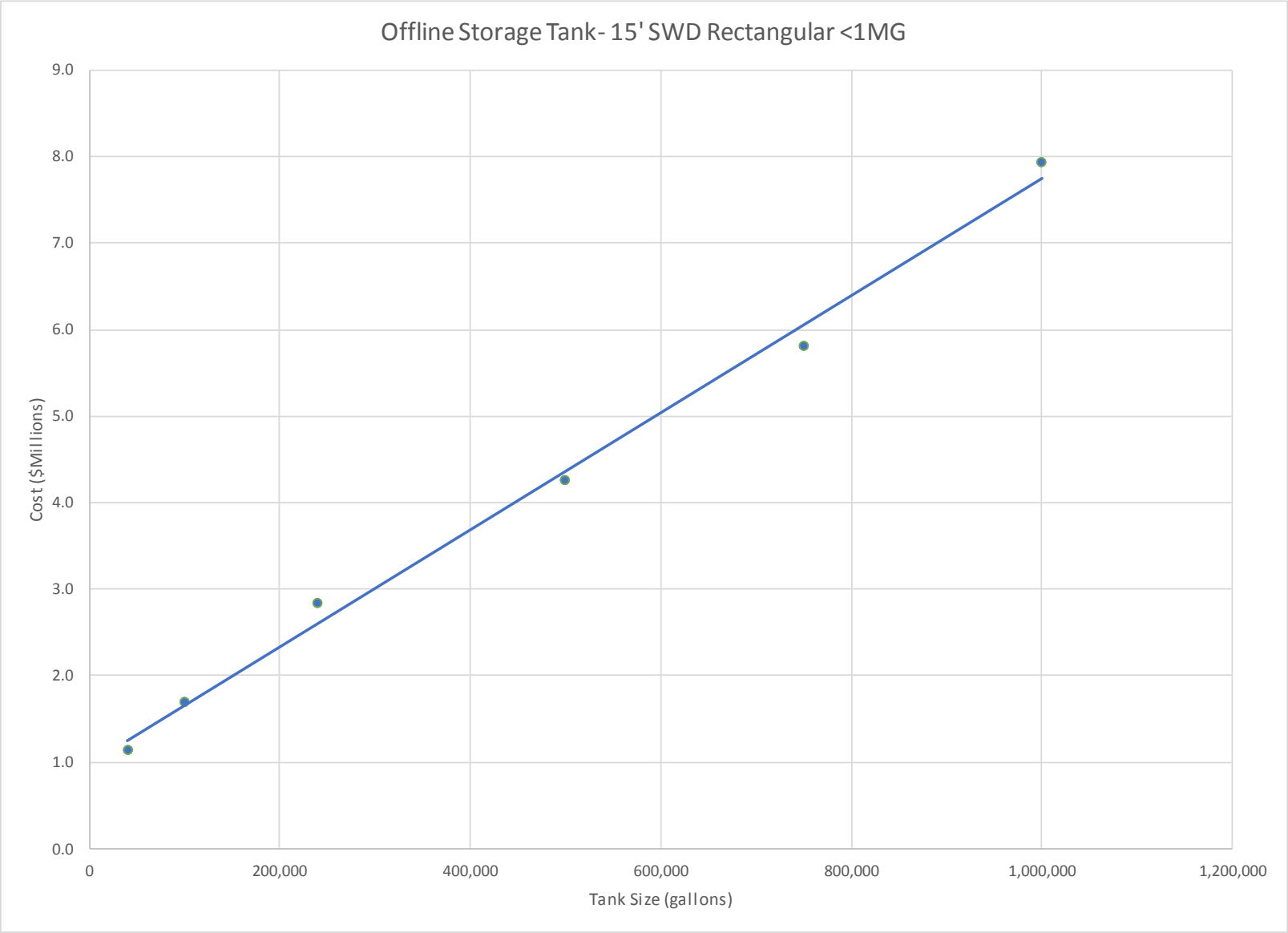


Figure 3-3 - Construction Cost Estimates for Off-Line Storage – 15’ SWD Rectangular > 1 MG

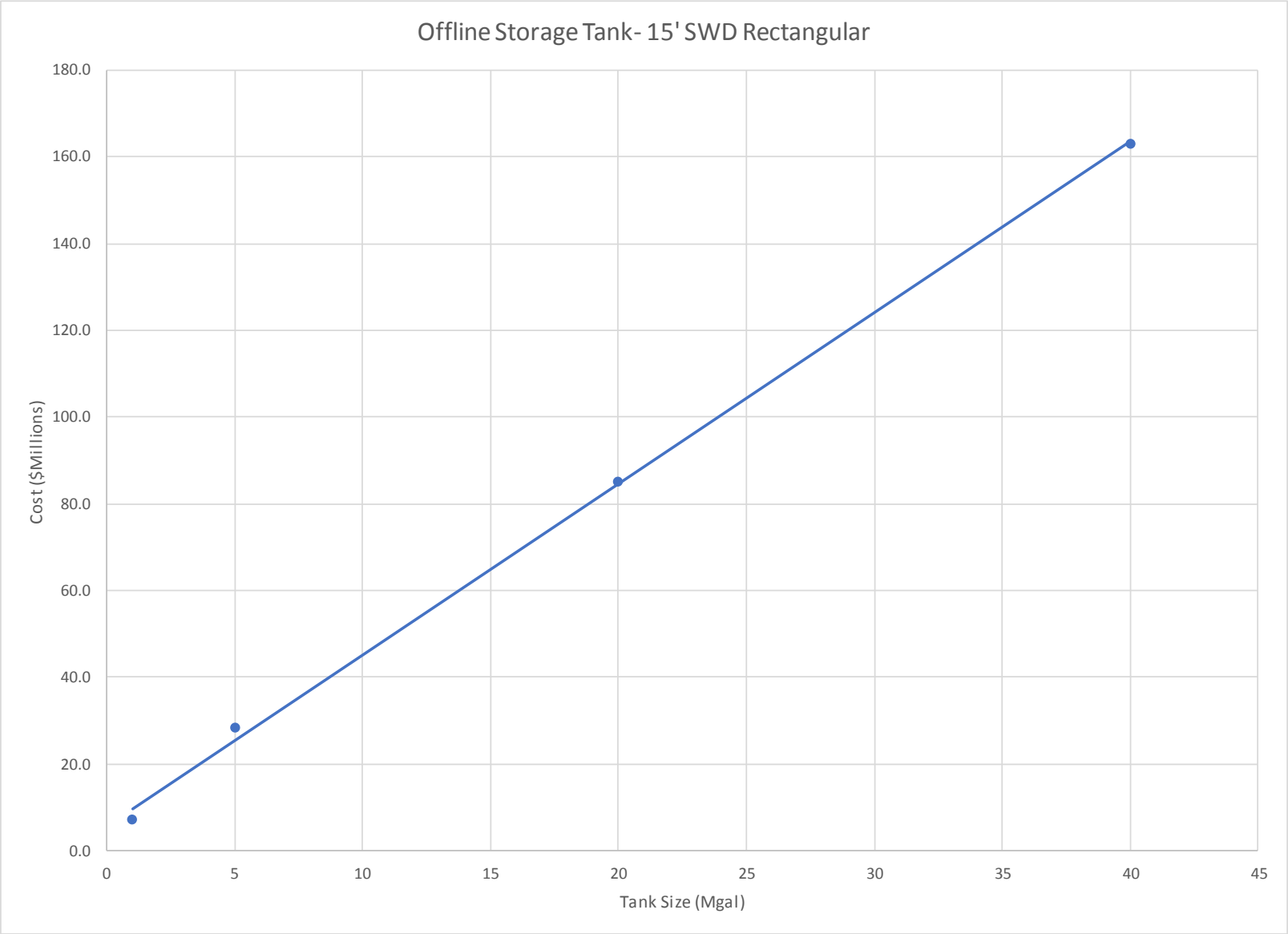
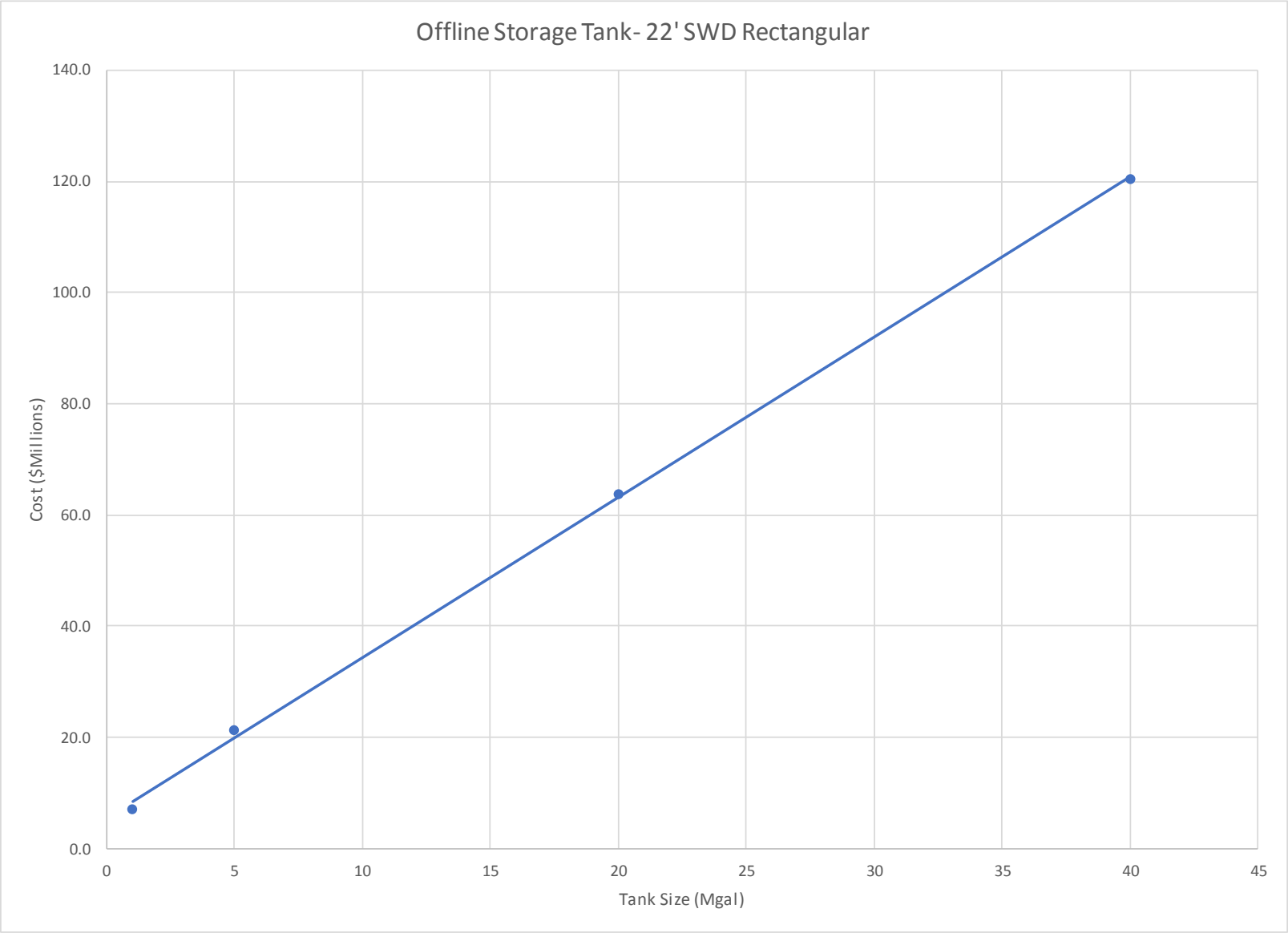


Figure 3-4 - Construction Cost Estimates for Off-Line Storage – 22’ SWD Rectangular



3.2.2 Deep Tunnel Storage

Deep tunnel storage has been gaining popularity as a positive means of reducing the volume of CSO discharges, especially in large urban areas where property values and disruptions to existing utilities and structures prohibit other forms of control. This control alternative involves the capture and storage of CSO discharges in a tunnel during wet weather events, and pumping the stored overflow back into sewer when conveyance and treatment capacity is available. New methods of construction have made deep tunnel storage a competitive option when considering the relatively low land requirements. Limitations of deep tunnels primarily include the need for specialized high-lift pumping stations and the inability to provide any treatment when the overflow exceeds the deep tunnel storage volume.

As with in -line and off-line storage, the original Technical Guidance Manual provided cost information for deep tunnel storage. Preliminary tunnel cost estimating graphs were prepared using compiled cost data from previously completed projects for the following tunneling scenarios:

- Tunnel in soft ground above the water table using an open faced boring machine with ribs and lagging primary liner and cast-in-place concrete final liner.
- Tunnel in soft ground below the water table driven using an earth pressure balanced boring machine with full gasketed concrete segmental liner erected immediately behind.
- Tunnel in rock driven using a rock-boring machine with pattern rock bolting and mesh reinforcement in the tunnel crown for primary support, and cast-in-place concrete final liner.

Since ground conditions may be unknown, an idealized cost estimate using certain assumptions on the amount of difficult conditions was also presented. A determination will need to be made as to the method that would need to be used based on general soil classifications and conditions within the region.

Notwithstanding the above, construction costs on tunneling projects are influenced by a multiplicity of factors. Tunnel cost estimates should only be used as a general initial guideline as they are based on a number of base assumptions and are not at all project specific. The major factors influencing costs on tunneling projects are described below:

- Tunnel length - assuming similar size and type of tunnels, a longer tunnel will generally have a lower unit rate than a smaller tunnel due to economies of scale. The original Technical Guidance Manual cost graphs assumed a 1.5 miles length of tunnel.
- Tunnel depth relative to the surface - deeper tunnels have deeper access shafts, which adds to the overall cost of the project. The original Technical Guidance Manual cost graphs assumed a tunnel no deeper than 30ft.
- Ground type & water table elevation - this can often be the most important cost factor as it influences the advance rates achieved, and choice of equipment and tunnel support. The original Technical Guidance Manual cost graphs assumed reasonable ground conditions and minimal water ingress problems to hinder the tunneling effort.

- Rate of advance achieved in the prevailing ground conditions. Average advance rates were assumed in the preparation of the tunnel cost graphs.
- Local labor conditions including availability of experienced personnel, prevailing wage rates, and union rules governing workers conditions, hours, and the minimum number of personnel which should be utilized for construction of the tunnel. The tunnel cost graphs presented in the original Technical Guidance Manual utilized labor conditions and numbers, which were believed to be appropriate for New Jersey.
- Local availability of appropriate tunneling equipment. The tunnel original Technical Guidance Manual cost graphs assumed that appropriate tunneling equipment is readily available in New Jersey.
- Occurrences of unforeseen ground conditions and obstructions. The original Technical Guidance Manual cost graphs assumed no major unforeseen conditions.
- Presence of sub-surface utilities and structures above requiring advance protection or monitoring during construction. The original Technical Guidance Manual cost curves assumed that no advance protection is required.

The foregoing list represents only a few of the factors which influence tunnel construction costs, and beyond the earliest stages of conceptual design it is recommended that all tunnel cost estimating be undertaken by an experienced tunneling engineer with an intimate awareness of the factors influencing tunnel costs. To cater for the unknown components inherent in preparation of the cost curves a relatively large cost contingency of 65% was applied throughout. In practical cost estimating, the cost contingency is reduced to as low as 5% as the design develops and more is known about the conditions which are likely to be encountered, and the tunneling techniques which will be utilized for the project.

In addition to tunnel costs, there are costs associated with conveying the flow into the tunnels. Typically, the discharges from outfalls are consolidated to decrease the number of drop shafts that will be needed. In addition, drop shafts are needed to transport flow from the regulators to the tunnel. The drop shaft consists of a large diameter shaft in which a vortex drop tube, vent shaft and access way are constructed. The space between the various components in a large diameter shaft is backfilled upon completion.

The original Technical Guidance Manual deep tunnel cost information was obtained and escalated to 2017 dollars based on the ENC CCI. The resultant cost curves are presented in Figures 3-6 through 3-8.

Figure 3-6 - Estimated Cost of Deep Tunnels Less Than 10,000 Linear Feet

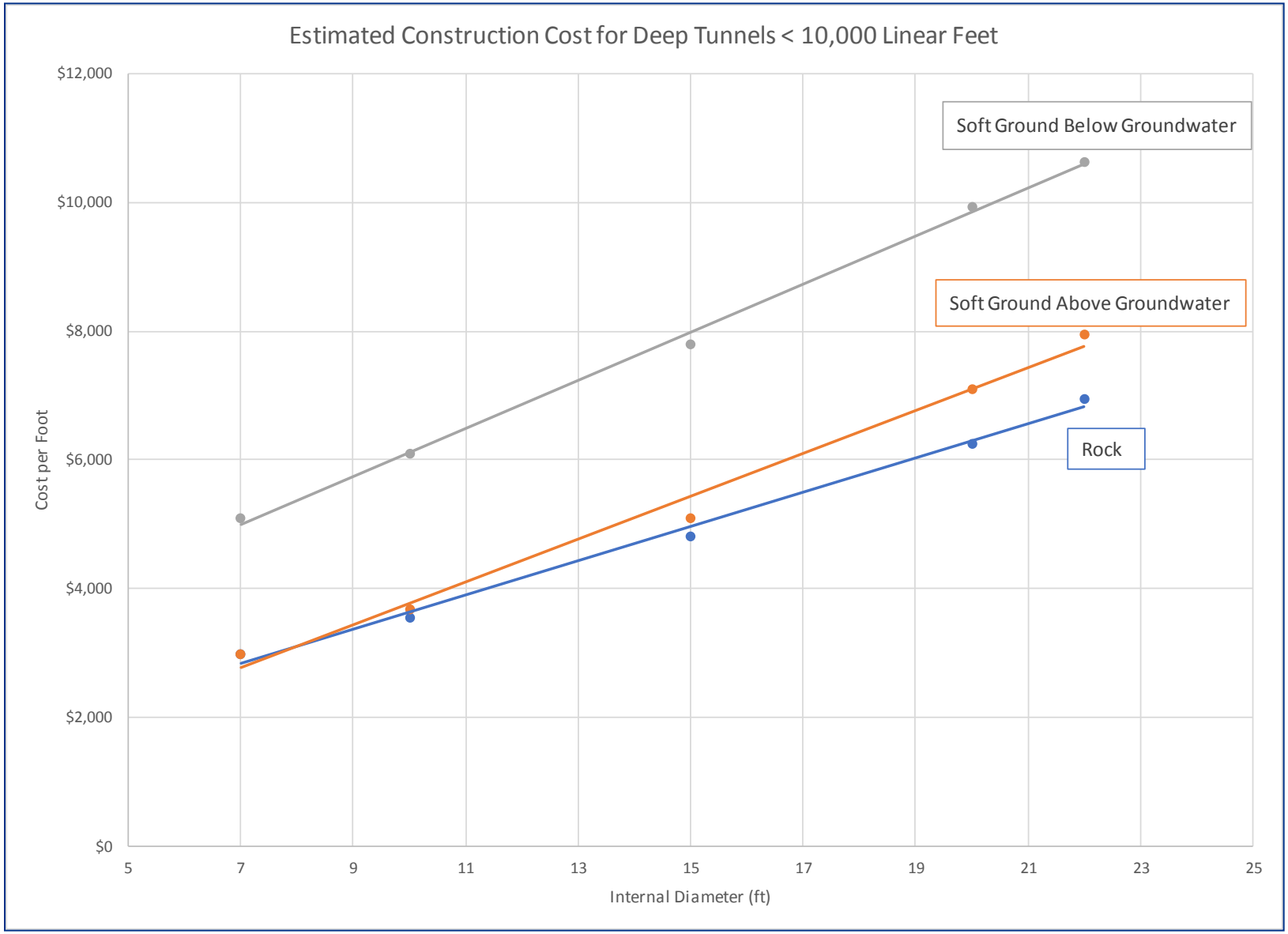


Figure 3-7 - Estimated Cost of Deep Tunnels Greater Than 10,000 Linear Feet

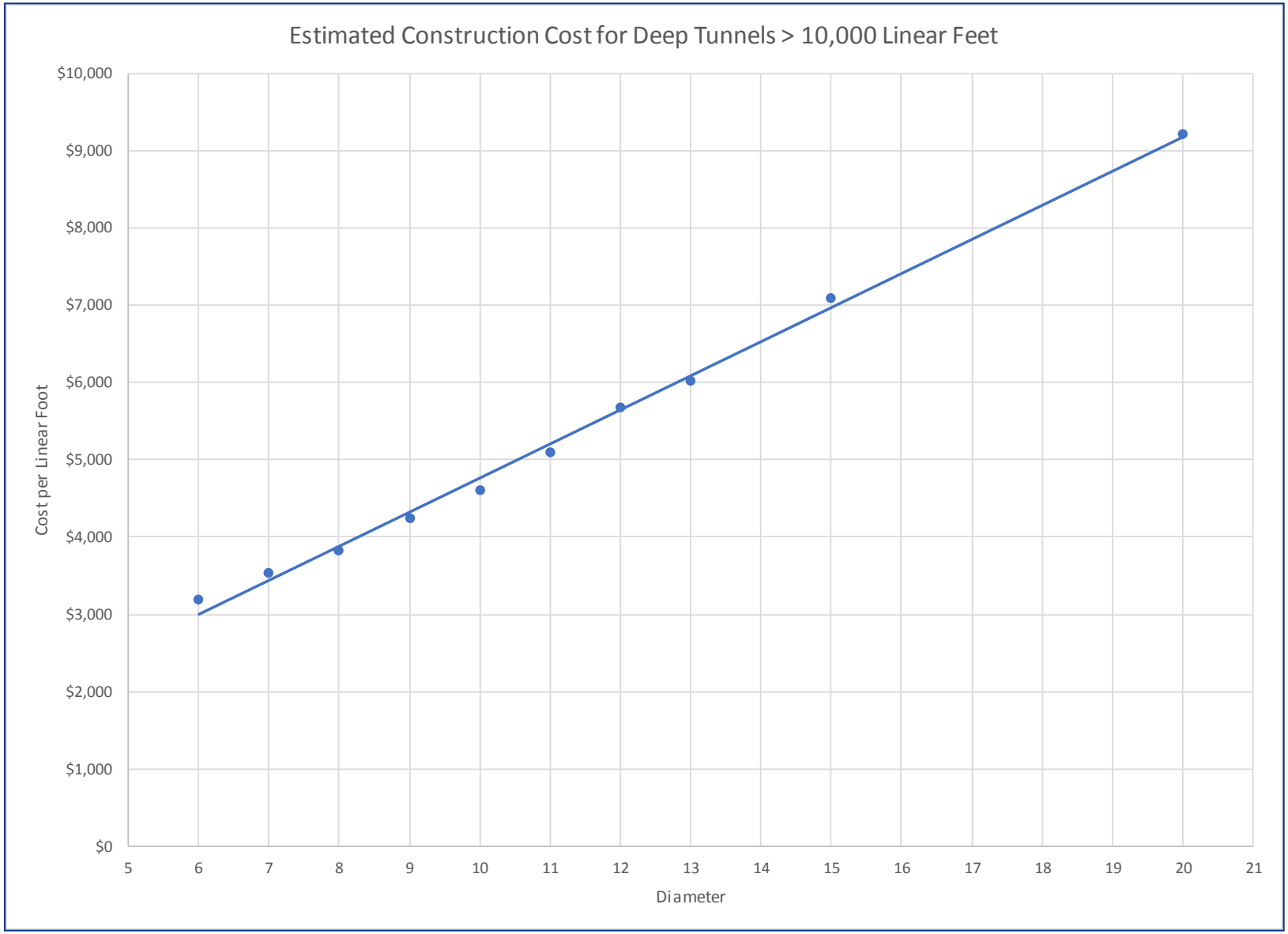
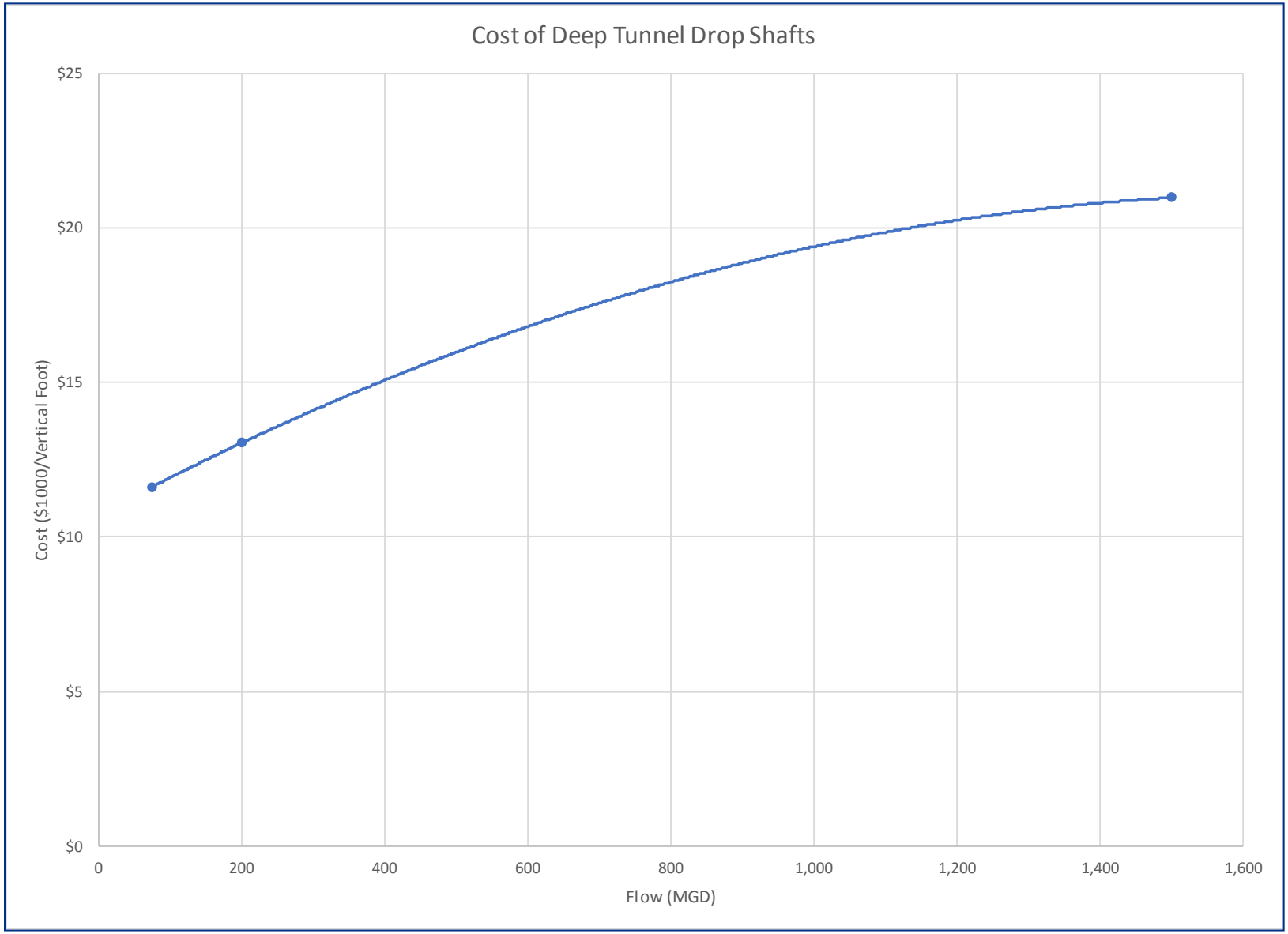


Figure 3-8 - Construction Cost Estimates for Tunnel Drop Shaft



Section 4

Green Infrastructure

The evaluation of Green Infrastructure for CSO control was not required by the prior NJPDES permit, and therefore was not included in the original Technical Guidance Manual. The NJPDES permits issued in 2015 however require permittees to evaluate Green Infrastructure as one of the CSO control alternatives.

The term "Green Infrastructure" is sometimes used to describe an array of source controls measures designed to capture stormwater before it enters the combined sewer collection system, as well as initiatives and regulatory requirements that reduce or limit runoff and pollutant loads. The Green Infrastructure described in this section of the TGM refers to physical structures that retain or detain stormwater runoff near where it originates. These structures are not necessary "green" in terms of being vegetated.

Green Infrastructure practices are designed to reduce the volume and/or peak of stormwater runoff that entering the combined sewer system. In retention systems, such as a rain garden, the runoff is routed to a permeable surface and allowed to infiltrate back into the ground. By preventing this stormwater from ever entering the collection system, the volume of overflow and associated pollutant loads discharging to the receiving waters is reduced. In detention systems, runoff is routed to a storage unit and returned to the combined sewer collection system, ideally after conveyance and treatment capacity have returned. By attenuating these flows, the conveyance system can accept a greater percentage of the overall runoff volume over a longer period of time, resulting in a net reduction of overflow volume and pollutant loads to the receiving waters.

4.1 Vegetated Practices

Many green infrastructure practices are in fact "green", in that they have a vegetative layer. That vegetative layer usually aides in the retention of stormwater runoff through transpiration, and the root system helps to promote soil porosity and aids infiltration. The green infrastructure practices also provide ancillary benefits, such as beautifying neighborhoods, improving air quality, and reducing urban heat. Through this section, several vegetated green infrastructure practices will be discussed:

- Rain Gardens
- Right-of-Way Bioswales
- Tree Pits
- Green Roofs
- Downspout Disconnection

4.1.1 Rain Gardens

Description of Practice

A rain garden consists of a shallow depressed area that is designed to collect stormwater runoff from surrounding surfaces. The collected water infiltrates into the ground, evaporates back into the atmosphere, or is transpired by the vegetation. To increase water absorption and promote infiltration, rain garden designs typically include an upper layer of amended soil with high porosity.

Plant selection and maintenance is critical to the long-term viability of a rain garden. Native plants should be selected that are capable of withstanding periods of ponded water as well as periods of dryness. Using native plants helps to reduce the amount of maintenance that will be required. Figure 4-1 provides a picture of a typical rain garden.

Figure 4-1 - Photo of Rain Garden

(Source: http://nemo.uconn.edu/raingardens/)

Applicability to The Project

Rain gardens can be implemented on public and private properties to capture and retain runoff. When properly designed and maintained they can provide aesthetic improvements to the urban landscape, natural wildlife habitat, and education opportunities for schools. Their shallow and relatively simple design means they can often be constructed without the use of heavy machinery.

Rain gardens are already used in CSO programs across the Country, and within the State of NJ. The Camden County MUA has installed an ~800 square foot rain garden that captures runoff from ~2,000 square feet of surrounding roadway.

Limitations

Proper rain garden design generally allows for a loading ratio of 5:1, with a maximum of about 10:1. The loading ratio is the ratio of contributing drainage area to the available infiltration area. In other words, to control runoff from a 500 square foot rooftop, a 100 square foot rain garden would be required. Infiltration practices that function at higher loading ratios have increased risk for failure due to the higher hydraulic, sediment, and pollutant loads.

The small loading ratio means that rain gardens require relatively large amounts of space. This makes them impractical for wide-spread public right-way application where such space is not available.

Construction Costs

The cost for constructing a rain garden can vary significantly based upon the complexity of the design, the location it is being built, and other local factors. The NJDEP guidance document "Review of GI as a Component of LTCPs" provides a range of $11/sf to $35/sf for construction costs, in 2016 dollars, compiled from projects across the United States. For wide-scale green infrastructure planning, costs are often normalized to units of dollars per impervious acre controlled. Using the 5:1 loading ratio, this range of construction costs is $96,000 to $305,000 per acre controlled which is in-line with local project experience.

4.1.2 Right-of-Way Bioswales

Description of Practice

The right-of-way bioswale is a curb-side green infrastructure design being widely employed as part of New York City's green infrastructure program for CSO control. To date several thousand units have been constructed or are in construction. There are several variations of the design with different widths and depth (right-of-way greenstrips, right-of-way raingardens) but the functionality is essentially the same.

The typical right-of-way bioswale is between 4 and 5 feet wide by 10 to 20 feet long. They are constructed in the existing sidewalk, with curb cuts to allow street runoff traveling along the gutter to enter the bioswale on the upstream side and excess flow to return to the street on the downstream side. It is this conveyance aspect of the practice that makes it a bioswale instead of a deep raingarden.

On the surface, the right-of-way bioswale looks and functions much like a rain garden described above. The unit includes a shallow ponding area, and a vegetative surface that may or may not include a tree. However, whereas a raingarden is generally less than a foot deep, the right-of-way bioswale is approximately 4 ½ feet deep. The first 2 ½ to 3', depending on the design is made up of an engineered soil designed to allow for rapid infiltration. The lower portion of the bioswale is a stone base to provide storage. A rendering of a New York City bioswale is provided in Figure 4-2.

Figure 4-2 - Rendering of Right-of-Way Bioswale

(Source www.nyc.gov/html/dep/html/stormwater/bioswales.shtml)

Applicability to The Project

The right-of-way makes up a significant amount of a city's impervious cover. Sidewalks and streets are generally pitched to capture and convey runoff directly towards the collection system, making them efficient locations to intercept the flow. Furthermore, the municipality already has ownership of these areas.

New York City is constructing thousands of right-of-way bioswales to capture urban runoff before it enters their combined sewer collection systems. The designs could easily be adapted to meet the needs of other combined sewer municipalities.

Limitations

The New York City standard design process sizes the bioswales based upon the calculated volume that can be managed through infiltration through the native surrounding soils, and storage within the unit, during a specified period. This generally results in loading ratios well above standard rule of thumb loading ratios for bio-infiltration practices. To date New York City's post construction monitoring program has shown that overall the units are functioning at or beyond their intended designs, but long-term monitoring results are not yet available. Permittees should consider the potential failure risks of utilizing similarly high loading ratios. Infiltration practices that function at higher loading ratios have increased risk for failure due to the higher hydraulic, sediment, and pollutant loads.

Constructing bio-infiltration practices in the sidewalk requires that the existing sidewalks are wide enough to allow for the feature while still maintaining functionality for pedestrian traffic. The ability to site right-of-way bioswales will have to be determined by each permittee.

Construction Costs

The actual construction costs for right-of-way bioswales is estimated to be approximately $15,000 unit, which equates to approximately $150,000 per acre controlled. These costs are based on large construction contracts generally including 100 – 200 units where an economy of scale can be achieved. For single unit or low quantity construction estimates, the costs can be significantly higher.

Prior to construction, identifying appropriate and effective locations for right-of-way bioswales requires planning, field work, and geotechnical investigations. When attempting to implement a wide-scale right-of-way green infrastructure program, many locations will be screened out due to site constraints or poorly infiltrating soils. Typical per-site survey and geotechnical costs can be approximately $4,000 to $5,000 per location. When sites are screened out after these costs have been incurred, the programmatic cost per constructed unit goes up to as much as $50,000 per unit.

4.1.3 Enhanced Tree Pits

Description of Practice

Enhanced tree pits, or stormwater trees, can appear similar to a standard city tree pit. Unlike a standard tree pit, however, they utilize an underground system designed to infiltrate runoff. The underground system includes engineered soil capable of rapidly infiltrating water, crushed stone, and an underdrain system. Although they can be built individually, they become more effective when they are installed as a connected multi-unit linear system. In such a system, permeable pavement can be used between the tree pits to allow additional water to infiltrate into a subsurface stone layer that connects the tree pits. A photo of an enhanced tree pit is provided in Figure 4-3.

Figure 4-3 - Photo of Enhanced Tree Pits

(Source: NJ Tree Foundation)

Applicability to The Project

Enhanced tree pits are already in use in cities across the United States as stormwater control measures. They can be constructed in sidewalks, in parking lots, courtyards, etc.

Limitations

The design of enhanced tree pits can vary greatly based on capture needs. The limitation for applicability are similar to those described for rain gardens and bioswales, depending on the desired loading ratio and available space.

Construction Costs

Pre-fabricated tree pits are available for approximately $10,000 each, and cost about $5,000 to install.

4.1.4 Green Roofs

Description of Practice

A green roof generally consists of a vegetated layer on top of a lightweight soil medium, below which lies an underdrain system and waterproof membrane. The depth of the soil medium will determine the type of vegetation that can be sustained and also the weight of the vegetated roof.

A portion of the precipitation that falls on the vegetated surface is retained in the soil medium and eventually released back to the atmosphere through evaporation and taken up through transpiration. The underdrain system acts as additional detention system before the excess water is eventually discharged through the buildings downspouts to the ground or directly into the combined sewer system. A photo of the green roof on Chicago's City Hall is shown in Figure 4-4.

Figure 4-4 - Photo of Green Roof on Chicago City Hall

(Source: www.greenroofs.com/)

Applicability to The Project

Green roofs have been constructed in cities around the world and across the country, including as part of CSO programs.

Limitations

Wide spread application of green roofs is generally cost prohibitive. Most existing buildings cannot support the additional weight of a green roof without costly retrofitting.

Green roofs are generally designed with a loading ratio of 1:1, meaning that the managed area is limited to the footprint of the vegetated area itself.

Construction Costs

The cost for constructing a green roof can vary significantly based upon the complexity of the design, the location it is being built, and other local factors. The NJDEP guidance document "Review of GI as a Component of LTCPs" provides a range of $11/sf to $56/sf for construction costs, in 2016 dollars, compiled from projects across the United States. Using the 1:1 loading ratio, this range of construction costs is $480,000 to $2,440,000 per acre controlled which is in-line with local project experience.

4.1.5 Downspout Disconnection

Description of Practice

In many urban areas, downspouts are connected directly into the combined sewer system. Disconnecting these downspouts provides opportunity for rooftop runoff to be infiltrated or intercepted before entering the combined sewer system. For buildings with exterior downspouts, disconnection can be as simple as cutting the existing downspout, installing an elbow, and routing the downspout to a pervious surface or storage unit, such as a rain barrel. For buildings with interior downspouts the process can be more complicated and may not be practical. However, opportunities may still exist where the internal drain can be located and re-routed through an exterior wall. A photo of the disconnected external downspout is shown in Figure 4-5.

Figure 4-5 - Photo of Disconnected Downspout

(Source: https://www.mmsd.com/what-you-can-do/downspout-disconnection)

Applicability to The Project

Many cities across the United States have adopted programs either requiring or encouraging downspout disconnection. A downspout disconnection program often provides the simplest and lowest cost for reduction in wet weather flow to the sewer system. The combined sewer communities within the PVSC service area should evaluate the potential for adopting such a program.

Construction Costs

Exterior downspout disconnections are usually simple, and can be accomplished for approximately $25 to $50.

4.2 Permeable Pavements

The term Permeable Pavements refers to several distinct surfaces, each of which are intended to provide a reduction in stormwater runoff as compared with traditional paving methods. The nomenclature for these different surfaces is often used interchangeably and can be confusing. The major types of permeable pavements will be discussed in this section, including:

- Porous Asphalt
- Pervious Concrete
- Permeable Pavers

4.2.1 Porous Asphalt

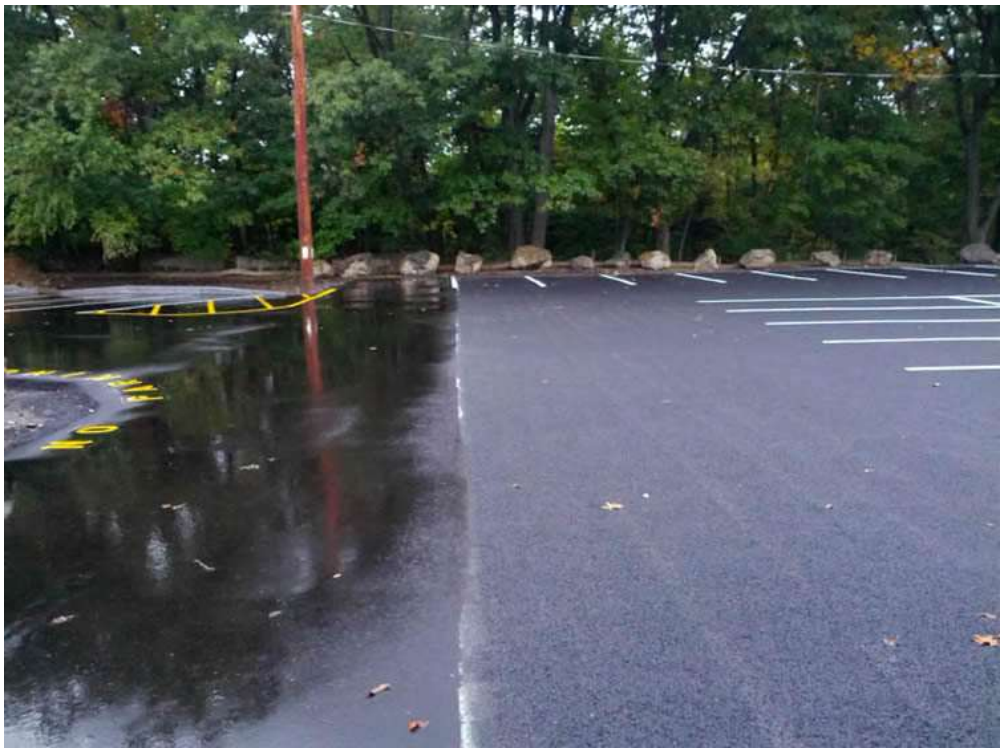
Description of Practice

Upon closer inspection, porous asphalt looks like a somewhat courser version of traditional asphalt, or "blacktop". Porous and traditional asphalt are made in a similar fashion, but the fine particles are left out of the porous asphalt mix. Without the fines, air becomes trapped in the asphalt mix creating pore space through which water can migrate.

Below the porous asphalt layer, a stone layer acts as a reservoir to store water before it infiltrates into the native soil. An underdrain system may also be included

Figure 4-5 provides a picture of a parking lot in which half was paved using porous asphalt (right side of photo) and the other half was paved using traditional asphalt (left side of photo).

Figure 4-5 - Porous Asphalt Parking Lot



(Source: https://www.epa.gov/soakuptherain/soak-rain-permeable-pavement)

Applicability to The Project

Porous pavement has been used successfully for decades to reduce ponding , flooding, and stormwater discharges. Many combined sewer cities are now using porous pavement as part of their CSO control strategy. Porous asphalt should be considered when roads or parking lots are to be constructed or repaved.

Limitations

Porous pavement requires additional maintenance, including regular service with a vacuum truck to help maintain the open pore space. The use of salt or sand for snow melting is also discouraged. Applications of porous asphalt are typically not recommended in high traffic or heavy industrial sites due to the increased sediment and pollutant loads.

Construction Costs

The cost for porous asphalt can vary significantly based upon whether it new surface or a retrofit. The NJDEP guidance document "Review of GI as a Component of LTCPs" provides a range of $12/sf to $25/sf for construction costs, in 2016 dollars, compiled from projects across the United States. For wide-scale green infrastructure planning, costs are often normalized to units of dollars per impervious acre controlled. Using a 2:1 loading ratio, this range of construction costs is $260,000 to $545,000 per acre controlled which is in-line with local project experience.

4.2.2 Pervious Concrete

Description of Practice

Pervious concrete is a concrete mix containing little or no sand, which creates pore space through which water can migrate. Pervious concrete functions similarly to porous asphalt in that water migrates through the pavements void space down into an underlying stone bed, and either infiltrates to the natural soil or enters an underdrain system. A photo of a pervious concrete application is shown in Figure 4-6. Pre-fabricated pervious concrete panels were installed in the parking stalls.

Figure 4-6 – Pervious Concrete Panels

Applicability to The Project

Pervious concrete pavement has been used successfully for decades to reduce ponding, flooding, and stormwater discharges. Many combined sewer cities are now using pervious concrete as part of their CSO control strategy. Pervious concrete can be considered for sidewalks, courtyards, or anywhere else that traditional concrete may be used.

Limitations

Pervious concrete requires additional maintenance, including regular service with a vacuum truck and pressure washing to help maintain the open pore space. The use of salt or sand for snow melting is also discouraged.

Construction Costs

The cost for pervious concrete can vary significantly based upon the type of application. The NJDEP guidance document "Review of GI as a Component of LTCPs" provides a range of $14/sf to $28/sf for construction costs, in 2016 dollars, compiled from projects across the United States. For wide-scale green infrastructure planning, costs are often normalized to units of dollars per impervious acre controlled. Using a 2:1 loading ratio, this range of construction costs is $305,000 to $610,000 per acre controlled which is in-line with local project experience.

4.2.2 Permeable Interlocking Concrete Pavers (PICP)

Description of Practice

Unlike pervious concrete, permeable pavers do not allow water to pass through the concrete. Instead, the joints between the impervious concrete pavers are filled with a permeable medium such as small stone or sand, allowing water to infiltrate between the pavers. The subsurface includes as stone base and an underdrain, if required.

A photo of a Philadelphia parking lot utilizing concrete permeable pavers is shown in Figure 4-7.

Figure 4-7 – Permeable Interlocking Concrete Pavers (source: EPA)

Applicability to The Project

As with the other types of permeable pavements, permeable interlocking concrete pavers are being used across the country for stormwater control.

Limitations

Permeable interlocking concrete pavers require regular service with a vacuum truck. Proper erosion control is required on the surrounding areas to prevent additional loading to the pavers and clogging.

Construction Costs

The cost for permeable pavers can vary significantly based upon the desired design and type of application. The NJDEP guidance document "Review of GI as a Component of LTCPs" provides a range of $12/sf to $34/sf for construction costs, in 2016 dollars, compiled from projects across the United States. For wide-scale green infrastructure planning, costs are often normalized to units of dollars per impervious acre controlled. Using a 4:1 loading ratio, this range of construction costs is $130,000 to $370,000 per acre controlled which is in-line with local project experience.

Section 5

Water Conservation

Reducing overall water consumption can provide some reduction in CSO discharge volume by providing additional wet weather capacity in the collection system and helping to alleviate the stress on the existing wastewater treatment facilities. It is difficult to quantify the CSO reduction provided through water conservation practices without modeling, and this Technical Guidance Manual does not attempt to do so. The CSO reduction benefits provided through water conservation measures will be dependent upon the coincidence of wet weather events and the highs and lows of daily water usage

Water consumption reduction can be achieved through a variety of measures including public outreach and education; distribution system leak detection and repair; water efficient landscaping; and water efficient plumbing fixtures (i.e., toilets and urinals, faucets, and showerheads). Assuming that nearly all water use inside residences and commercial users will ultimately be disposed of in the sewer, outside water use, such as lawn watering and leaks in the distribution system will not be addressed in the TGM.

This section will focus on water efficient plumbing fixtures and discuss the water saving and costs while implementing water efficient plumbing fixtures.

5.1 Water Efficient Toilets and Urinals

Nearly one-third of total water consumption returns to the sewer system through flushed toilets and urinals. Many plumbing fixtures still in use today were designed at a time when little concern was given to water conservation. Prior to 1950, typical toilets consumed 7-gallons-per-flush (gpf). Toilets installed between 1950 and 1994 consumed 4-5 gpf. Federal laws enacted in 1994 required that residential toilets use no more than 1.6 gpf. A similar limit was established for commercial toilets in 1997, and urinals were limited to 1.0 gpf by the 1997 requirements.

Average water savings by using low-volume toilets compared to high-volume ones is shown for residential households in Table 5-1, and for industrial and commercial facilities in Table 5-2. Average water savings by using low-volume urinals compared to high-volume ones in industrial and commercial facilities only is shown in Table 5-3.

Table 5-1 - Estimated Water Savings Provided by Low Volume Toilets in Households

Year Installed	Average Toilet Water Use Rate (gpf)	Estimated Water Use (gal/household/day)	Estimated Water Use Annually (gal/household/year)	Estimated Annual Water Savings (gal/household/year)
1994 - Present	1.6	32	11,680	-
1980-1994	4.0	80	29,200	17,520
1950s - 1980	5.0	100	36,500	24,820
Pre-1950s	7.0	140	51,100	39,420

Notes: Assume a 4-person household at 5 uses per person per day.

Table 5-2 - Estimated Water Savings Provided by Low Volume Toilets in Commercial and Industrial Facilities

Year Installed	Average Toilet Water Use Rate (gpf)	Average Daily Use (gal/toilet/day)	Estimated Water Use Annually (gal/toilet/year)	Estimated Annual Water Savings (gal/toilet/year)
1997 - Present	1.6	38.4	14,016	-
1980-1994	4.0	96	35,040	21,024
1950s - 1980	5.0	120	43,800	29,784
Pre-1950s	7.0	168	61,320	47,304

Notes: Assume an average daily use of 24 times per toilet per day.

Table 5-3 - Estimated Water Savings Provided by Low Volume Urinals in Commercial and Industrial Facilities

Year Installed	Average Toilet Water Use Rate (gpf)	Estimated Average Daily Use (gal/urinal/day)	Estimated Water Use Annually (gal/urinal/year)	Estimated Annual Water Savings (gal/urinal/year)
1997 - Present	1	16	5,840	-
1980-1994	2.0	32	11,680	5,840
Pre 1980	5.0	80	29,200	23,360

Notes: Assume an average daily use of 16 times per urinal per day.

An estimate of the typical costs associated with replacing a toilet or urinal was developed using construction cost estimating database such as R.S. Means. In 2017 dollar, the equipment and labor costs were:

- Residential Floor Mounted Toilets = $645 per fixture
- Commercial Wall Hung Toilets = $1,225 per fixture
- Urinals = $615 per fixture

5.2 Water Efficient Faucets and Showerheads

Significant amounts of water and energy can be wasted through use of non-water efficient faucets and showerheads. Even a brief five-minute shower can consume 15-35 gallons of water with a conventional showerhead with a flow rate of 3-7 gpm.

Prior to 1980, typical faucets had a flowrate of 4 gpm. Faucets installed between 1980 and 1994 flowed at approximately 3 gpm. Federal guidelines in 1994 required that all lavatory and kitchen faucets and replacement aerators use no more than 2.5 gpm measured at normal water pressure (typically 80 pounds per square inch, psi). A similar limit was established for showerheads in 1994, which reduced the typical flowrate of a showerhead from 3-7 gpm to 2.5 gpm.

Average water savings by using low-flow faucets compared to high-flow ones is shown for residential households in Table 5-4, and for industrial and commercial facilities in Table 5-5. Average water savings by using low-flow showerheads compared to high-flow ones in residential households is shown in Table 5-6.

Table 5-4 - Estimated Water Savings Provided by Low Flow Faucets in Households

Year Installed	Average Faucet Flowrate (gpm)	Estimated Faucet Use (gal/household/day)	Estimated Water Use Annually (gal/household/year)	Estimated Annual Water Savings (gal/household/year)
1994 - Present	2.5	100	36,500	-
1980-1994	3.0	120	43,800	7,300
Pre-1980s	4.0	160	58,400	21,900

Notes: Assume a 4-person household at 10-minutes uses per person per day.

Table 5-5 - Estimated Water Savings Provided by Low Flow Faucets in Commercial and Industrial Facilities

Year Installed	Average Faucet Flowrate (gpm)	Average Daily Use (gal/faucet/day)	Estimated Water Use Annually (gal/faucet/year)	Estimated Annual Water Savings (gal/faucet/year)
1994 - Present	2.5	180	65,700	-
1980-1994	3.0	216	78,840	13,140
Pre-1980s	4.0	288	105,120	39,420

Notes: Assume an average daily use of 72 minutes per faucet per day.

Table 5-6 - Estimated Water Savings Provided by Low Flow Showerheads in Households

Year Installed	Average Showerhead Flowrate (gpm)	Average Daily Use (gal/household/day)	Estimated Water Use Annually (gal/household/year)	Estimated Annual Water Savings (gal/household/year)
1997 - Present	2.5	62.5	22,813	-
1980-1994	3.0	75	27,375	4,563
Pre 1980	7.0	175	63,875	41,063

Notes: Assume a 4-person household at 25-minutes uses per person per day.

An estimate of the typical costs associated with replacing a toilet or urinal was developed using construction cost estimating database such as R.S. Means. In 2017 dollar, the equipment and labor costs were:

- Residential Faucet Replacement = $189
- Residential Showerhead Replacement (including built-in, head, arm, and 2.5 gpm valve) = $350

Commercial Faucet Replacement (with automatic sensor and operator) = $675

Appendix A

Climber Screens® Installation List

(Source: Suez, formerly Infilco Degremont, Inc.)

July 2017

Climber Screen® Installation List
Type IIS and IIIAS
NJ, NY, PA
2000-2015

Serial Number	Contract#	State	Location	Name	Year	Qty	Type	Design Flow Rate	Unit of Measure	Channel Width	Channel Depth	Max. Water Depth	Clear Spacing	Channel Invert to Operating Floor	Material - Non Wetted	Material - Wetted
CS-1445	00012	NY	Brooklyn	Red Hook WPCP (Replaced 84-949)	2000	1	IIS	70.0	MGD	72	100.25	72	1	429.5	316SS	316SS
CS-1446	00012	NY	Brooklyn	Red Hook WPCP (Replaced 84-949)	2000	1	IIS	70.0	MGD	72	100.25	72	1	429.5	316SS	316SS
CS-1447	00012	NY	Brooklyn	Red Hook WPCP (Replaced 84-949)	2000	1	IIS	70.0	MGD	72	100.25	72	1	429.5	316SS	316SS
CS-1448	00012	NY	Brooklyn	Red Hook WPCP (Replaced 84-949)	2000	1	IIS	70.0	MGD	72	100.25	72	1	429.5	316SS	316SS
CS-1478	00103	PA	Erie	Erie WWTP - East Headworks	2000	1	IIS	58.0	MGD	72	120	90	1	120	Carbon Steel	304SS
CS-1479	00103	PA	Erie	Erie WWTP - East Headworks	2000	1	IIS	58.0	MGD	72	120	90	1	120	Carbon Steel	304SS
CS-1480	00103	PA	Erie	Erie WWTP - East Headworks	2000	1	IIS	58.0	MGD	72	120	90	1	120	Carbon Steel	304SS
CS-1499	01138	NY	Albany	Albany County WWTP	2001	1	IIS	50.0	MGD	48	88	82	1	450	Carbon Steel	304SS
CS-1500	01138	NY	Albany	Albany County WWTP	2001	1	IIS	50.0	MGD	48	88	82	1	450	Carbon Steel	304SS
CS-1501	01138	NY	Albany	Albany County WWTP	2001	1	IIS	50.0	MGD	48	114	108	1	474	Carbon Steel	304SS
CS-1502	01138	NY	Albany	Albany County WWTP	2001	1	IIS	50.0	MGD	48	114	108	1	474	Carbon Steel	304SS
CS-1503	01137	NY	Suffolk County	Bergen Point STP	2001	1	IIS			72	258		0.75			
CS-1527	01205	NY	Bronx	Hunts Point WPCP (Replaced 84-904)	2001	1	IIIAS	80.0	MGD	84	144	132	0.5	144	Carbon Steel	304SS
CS-1528	01205	NY	Bronx	Hunts Point WPCP (Replaced 84-904)	2001	1	IIIAS	80.0	MGD	84	144	132	0.5	144	Carbon Steel	304SS
CS-1529	01205	NY	Bronx	Hunts Point WPCP (Replaced 84-904)	2001	1	IIIAS	80.0	MGD	84	144	132	0.5	144	Carbon Steel	304SS
CS-1530	01205	NY	Bronx	Hunts Point WPCP (Replaced 84-904)	2001	1	IIIAS	80.0	MGD	84	144	132	0.5	144	Carbon Steel	304SS
CS-1531	01205	NY	Bronx	Hunts Point WPCP (Replaced 84-904)	2001	1	IIIAS	80.0	MGD	84	144	132	0.5	144	Carbon Steel	304SS
CS-1539	02253	NY	Binghamton	Binghamton-Johnson County WWTP	2002	1	IIS		MGD	48	270		0.75	381	Carbon Steel	304SS
CS-1540	02253	NY	Binghamton	Binghamton-Johnson County WWTP	2002	1	IIS		MGD	48	270		0.75	381	Carbon Steel	304SS
CS-1559	01137	NY	Suffolk County	Bergen Point STP	2001	1	IIS		MGD	72	258	135	0.75	414	304SS	304SS
CS-1560	01137	NY	Suffolk County	Bergen Point STP	2001	1	IIS		MGD	72	258	135	0.75	414	304SS	304SS
CS-1594	04401	NY	Brooklyn	Coney Island WPCP (Replaced 84-927 CS-32	2004	1	IIS		MGD	60	218.438		0.75	218.4375	Carbon Steel	304SS

Climber Screen® Installation List
Type IIS and IIIAS
NJ, NY, PA
2000-2015

July 2017

Serial Number	Contract#	State	Location	Name	Year	Qty	Type	Design Flow Rate	Unit of Measure	Channel Width	Channel Depth	Max. Water Depth	Clear Spacing	Channel Invert to Operating Floor	Material - Non Wetted	Material - Wetted
CS-1595	04401	NY	Brooklyn	Coney Island WPCP (Replaced 84-927 CS-32	2004	1	IIS		MGD	60	218.438		0.75	218.4375	Carbon Steel	304SS
CS-1596	04401	NY	Brooklyn	Coney Island WPCP (Replaced 84-927 CS-32	2004	1	IIS		MGD	60	218.438		0.75	218.4375	Carbon Steel	304SS
CS-1599	05462	NJ	Sayreville	Sayreville PS	2005	1	IIS	100.0	MGD	60	296.5		1	440.5	304SS	304SS
CS-1600	05462	NJ	Sayreville	Sayreville PS	2005	1	IIS	100.0	MGD	60	296.5		1	440.5	304SS	304SS
CS-1601	05462	NJ	Sayreville	Sayreville PS	2005	1	IIS	100.0	MGD	60	296.5		1	440.5	304SS	304SS
CS-1602	05462	NJ	Sayreville	Sayreville PS	2005	1	IIS	100.0	MGD	60	296.5		1	440.5	304SS	304SS
CS-1604	04451	NY	Brooklyn	Owls Head WPCP (Replaced 84-926 Coarse)	2004	1	IIIAS			81	174		1.25	336	Carbon Steel	304SS
CS-1605	04451	NY	Brooklyn	Owls Head WPCP (Replaced 84-926 Coarse)	2004	1	IIIAS			81	174		1.25	336	Carbon Steel	304SS
CS-1606	04451	NY	Brooklyn	Owls Head WPCP (Replaced 84-926 Coarse)	2004	1	IIIAS			81	174		1.25	336	Carbon Steel	304SS
CS-1607	04451	NY	Brooklyn	Owls Head WPCP (Replaced 84-926 Coarse)	2004	1	IIIAS			81	174		1.25	336	Carbon Steel	304SS
CS-1608	04451	NY	Brooklyn	Owls Head WPCP (Replaced 84-926 Fine)	2004	1	IIIAS			81	174		0.75	336	Carbon Steel	304SS
CS-1609	04451	NY	Brooklyn	Owls Head WPCP (Replaced 84-926 Fine)	2004	1	IIIAS			81	174		0.75	336	Carbon Steel	304SS
CS-1610	04451	NY	Brooklyn	Owls Head WPCP (Replaced 84-926 Fine)	2004	1	IIIAS			81	174		0.75	336	Carbon Steel	304SS
CS-1611	04451	NY	Brooklyn	Owls Head WPCP (Replaced 84-926 Fine)	2004	1	IIIAS			81	174		0.75	336	Carbon Steel	304SS
CS-1621	05476	NJ	Camden County	Camden County WWTP	2005	1	IIS	150.0	MGD	72	276	126	1	276	Carbon Steel	304SS
CS-1622	05476	NJ	Camden County	Camden County WWTP	2005	1	IIS	150.0	MGD	72	276	126	1	276	Carbon Steel	304SS
CS-1623	05476	NJ	Camden County	Camden County WWTP	2005	1	IIS	150.0	MGD	72	276	126	1	276	Carbon Steel	304SS
CS-1624	04441	NY	New York	13th St. Manhattan PS (Replaced 85-032)	2004	1	IIIAS	100.0	GPM	66	144	120	1	522	Carbon Steel	316SS
CS-1625	04441	NY	New York	13th St. Manhattan PS (Replaced 85-032)	2004	1	IIIAS	100.0	GPM	66	144	120	1	522	Carbon Steel	316SS
CS-1626	04441	NY	New York	13th St. Manhattan PS (Replaced 85-032)	2004	1	IIIAS	100.0	GPM	66	144	120	1	522	Carbon Steel	316SS
CS-1627	04441	NY	New York	13th St. Manhattan PS (Replaced 85-032)	2004	1	IIIAS	100.0	GPM	66	144	120	1	522	Carbon Steel	316SS
CS-1629	05486	NY	Onondaga County	Baldwinsville Senera Knolls	2005	1	IIS		MGD	48	66		1	360	304SS	304SS

Climber Screen® Installation List
Type IIS and IIIAS
NJ, NY, PA
2000-2015

July 2017

Serial Number	Contract#	State	Location	Name	Year	Qty	Type	Design Flow Rate	Unit of Measure	Channel Width	Channel Depth	Max. Water Depth	Clear Spacing	Channel Invert to Operating Floor	Material - Non Wetted	Material - Wetted
CS-1630	05486	NY	Onondaga County	Baldwinsville Senera Knolls	2005	1	IIS		MGD	48	66		1	360	304SS	304SS
CS-1631	05486	NY	Onondaga County	Ley Creek PS	2005	1	IIS		MGD	48	260.5		1	260.5	304SS	304SS
CS-1632	05486	NY	Onondaga County	Ley Creek PS	2005	1	IIS		MGD	48	260.5		1	260.5	304SS	304SS
CS-1633	05486	NY	Onondaga County	Metropolitan Syracuse Effluent Channel	2005	1	IIS		MGD	71	203.5		0.75	203.5	304SS	304SS
CS-1634	05486	NY	Onondaga County	Metropolitan Syracuse Effluent Channel	2005	1	IIS		MGD	71	203.5		0.75	203.5	304SS	304SS
CS-1635	05486	NY	Onondaga County	Metropolitan Syracuse Effluent Channel	2005	1	IIS		MGD	72	150.625		1.5	150.625	304SS	304SS
CS-1636	05486	NY	Onondaga County	Metropolitan Syracuse Effluent Channel	2005	1	IIS		MGD	72	150.625		1.5	150.625	304SS	304SS
CS-1650	05504	NJ	Rahway	Rahway Valley WWTP	2005	1	IIS	52.5	MGD	72	145	72	3	369	Carbon Steel	304SS
CS-1651	05504	NJ	Rahway	Rahway Valley WWTP	2005	1	IIS	52.5	MGD	72	145	72	3	369	Carbon Steel	304SS
CS-1652	05504	NJ	Rahway	Rahway Valley WWTP	2005	1	IIS	52.5	MGD	72	145	72	3	369	Carbon Steel	304SS
CS-1653	05504	NJ	Rahway	Rahway Valley WWTP	2005	1	IIS	52.5	MGD	72	145	72	3	369	Carbon Steel	304SS
CS-1654	05504	NJ	Rahway	Rahway Valley WWTP	2005	1	IIS	52.5	MGD	72	145	72	3	369	Carbon Steel	304SS
CS-1655	05504	NJ	Rahway	Rahway Valley WWTP	2005	1	IIS	52.5	MGD	72	145	72	3	369	Carbon Steel	304SS
CS-1657	05509	NY	Brooklyn	Paerdegat PS	2005	1	IIIAS	333.0	MGD	108	322	168	1.25	322	Carbon Steel	316SS
CS-1658	05509	NY	Brooklyn	Paerdegat PS	2005	1	IIIAS	333.0	MGD	108	322	168	1.25	322	Carbon Steel	316SS
CS-1659	05509	NY	Brooklyn	Paerdegat PS	2005	1	IIIAS	333.0	MGD	108	322	168	1.25	322	Carbon Steel	316SS
CS-1660	05509	NY	Brooklyn	Paerdegat PS	2005	1	IIIAS	333.0	MGD	108	322	168	1.25	322	Carbon Steel	316SS
CS-1661	05509	NY	Brooklyn	Paerdegat PS	2005	1	IIIAS	333.0	MGD	108	322	168	1.25	322	Carbon Steel	316SS
CS-1662	05509	NY	Brooklyn	Paerdegat PS	2005	1	IIIAS	333.0	MGD	108	322	168	1.25	322	Carbon Steel	316SS
CS-1690	08610	NY	Brooklyn	Newtown Creek WPCP (Replaced 86-119)	2008	1	IIIAS	100.0	MGD	78	148.5	86	1	496.5	Carbon Steel	316SS
CS-1691	08610	NY	Brooklyn	Newtown Creek WPCP (Replaced 86-119)	2008	1	IIIAS	100.0	MGD	78	148.5	86	1	496.5	Carbon Steel	316SS
CS-1692	08610	NY	Brooklyn	Newtown Creek WPCP (Replaced 86-119)	2008	1	IIIAS	100.0	MGD	78	148.5	86	1	496.5	Carbon Steel	316SS

Climber Screen® Installation List
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NJ, NY, PA
2000-2015

July 2017

Serial Number	Contract#	State	Location	Name	Year	Qty	Type	Design Flow Rate	Unit of Measure	Channel Width	Channel Depth	Max. Water Depth	Clear Spacing	Channel Invert to Operating Floor	Material - Non Wetted	Material - Wetted
CS-1693	08610	NY	Brooklyn	Newtown Creek WPCP (Replaced 86-119)	2008	1	IIIAS	100.0	MGD	78	148.5	86	1	496.5	Carbon Steel	316SS
CS-1720	09657	NY	New York	Powell's Cove PS (Replaced 84-937)	2009	1	IIS		MGD	54	90		1.25	408	Carbon Steel	316LSS
CS-1739	09671	NY	Albany	Albany North & South WWTP	2009	1	IIS		MGD	60	114		1	468	Carbon Steel	304LSS
CS-1740	09671	NY	Albany	Albany North & South WWTP	2009	1	IIS		MGD	48	88		1	444	Carbon Steel	304LSS
CS-1751	10700	NY	Brooklyn	Newtown Creek WPCP (Secondary)	2010	1	IIS	70.0	MGD	76	276	156	0.375	276	Carbon Steel	304SS
CS-1752	10700	NY	Brooklyn	Newtown Creek WPCP (Secondary)	2010	1	IIS	70.0	MGD	76	276	156	0.375	276	Carbon Steel	304SS
CS-1753	10700	NY	Brooklyn	Newtown Creek WPCP (Secondary)	2010	1	IIS	70.0	MGD	76	276	156	0.375	276	Carbon Steel	304SS
CS-1754	10700	NY	Brooklyn	Newtown Creek WPCP (Secondary)	2010	1	IIS	70.0	MGD	76	276	156	0.375	276	Carbon Steel	304SS
CS-1755	10700	NY	Brooklyn	Newtown Creek WPCP (Secondary)	2010	1	IIS	70.0	MGD	76	276	156	0.375	276	Carbon Steel	304SS
CS-1756	10700	NY	Brooklyn	Newtown Creek WPCP (Secondary)	2010	1	IIS	70.0	MGD	76	276	156	0.375	276	Carbon Steel	304SS
CS-1757	10700	NY	Brooklyn	Newtown Creek WPCP (Secondary)	2010	1	IIS	70.0	MGD	76	276	156	0.375	276	Carbon Steel	304SS
CS-1758	10700	NY	Brooklyn	Newtown Creek WPCP (Secondary)	2010	1	IIS	70.0	MGD	76	276	156	0.375	276	Carbon Steel	304SS
CS-1759	10700	NY	Brooklyn	Newtown Creek WPCP (Secondary)	2010	1	IIS	70.0	MGD	76	276	156	0.375	276	Carbon Steel	304SS
CS-1760	10700	NY	Brooklyn	Newtown Creek WPCP (Secondary)	2010	1	IIS	70.0	MGD	76	276	156	0.375	276	Carbon Steel	304SS
CS-1761	10700	NY	Brooklyn	Newtown Creek WPCP (Secondary)	2010	1	IIS	70.0	MGD	76	276	156	0.375	276	Carbon Steel	304SS
CS-1762	10700	NY	Brooklyn	Newtown Creek WPCP (Secondary)	2010	1	IIS	70.0	MGD	76	276	156	0.375	276	Carbon Steel	304SS
CS-1768	10703	NY	Brooklyn	26th Ward WPCP (Replaced 89-441)	2010	1	IIIAS	45.0	MGD	66	98.5	98.5	1	300.5625	Carbon Steel	304SS
CS-1769	10703	NY	Brooklyn	26th Ward WPCP (Replaced 89-441)	2010	1	IIIAS	45.0	MGD	66	98.5	98.5	1	300.5625	Carbon Steel	304SS
CS-1770	10703	NY	Brooklyn	26th Ward WPCP (Replaced 89-441)	2010	1	IIIAS	45.0	MGD	66	102	102	1	288	Carbon Steel	304SS
CS-1771	10703	NY	Brooklyn	26th Ward WPCP (Replaced 89-441)	2010	1	IIIAS	45.0	MGD	66	93	93	1	413.25	Carbon Steel	304SS
CS-1772	10703	NY	Brooklyn	26th Ward WPCP (Replaced 89-441)	2010	1	IIIAS	45.0	MGD	66	93	93	1	413.25	Carbon Steel	304SS
CS-1773	10703	NY	Brooklyn	26th Ward WPCP (Replaced 89-441)	2010	1	IIIAS	45.0	MGD	66	88	88	1	413.25	Carbon Steel	304SS

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July 2017

Serial Number	Contract#	State	Location	Name	Year	Qty	Type	Design Flow Rate	Unit of Measure	Channel Width	Channel Depth	Max. Water Depth	Clear Spacing	Channel Invert to Operating Floor	Material - Non Wetted	Material - Wetted
CS-1794	11751	NY	Troy	Rensselear County District #1 WWTP	2011	1	IIS	30.0	GPM	48	119	119	0.75	119	Carbon Steel	304SS
CS-1795	11751	NY	Troy	Rensselear County District #1 WWTP	2011	1	IIS	30.0	GPM	48	119	119	0.75	119	Carbon Steel	304SS
CS-1799	11762	NJ	Sayreville	MCUA Sayreville PS	2011	1	IIS	56.0	GPM	72	297		0.625	471	304SS	304SS
CS-1800	11762	NJ	Sayreville	MCUA Sayreville PS	2011	1	IIS	56.0	GPM	72	297		0.625	471	304SS	304SS
CS-1801	11762	NJ	Sayreville	MCUA Sayreville PS	2011	1	IIS	56.0	GPM	72	297		0.625	471	304SS	304SS
CS-1806	11771	NY	Jamaica	Jamaica WPCP (Replaced 88-271)	2011	1	IIIAS	67.0	MGD	99	112.5	112.5	1	398.5	Carbon Steel	304SS
CS-1807	11771	NY	Jamaica	Jamaica WPCP (Replaced 88-271)	2011	1	IIIAS	67.0	MGD	99	112.5	112.5	1	398.5	Carbon Steel	304SS
CS-1808	11771	NY	Jamaica	Jamaica WPCP (Replaced 88-271)	2011	1	IIIAS	67.0	MGD	99	112.5	112.5	1	398.5	Carbon Steel	304SS
CS-1809	11771	NY	Jamaica	Jamaica WPCP (Replaced 88-271)	2011	1	IIIAS	67.0	MGD	99	112.5	112.5	1	398.5	Carbon Steel	304SS
CS-1816	13819	PA	Allentown	Kline's Island WWTP	2013	1	IIS	88.0	MGD							
CS-1817	13819	PA	Allentown	Kline's Island WWTP	2013	1	IIS	88.0	MGD							
CS-1818	13821	NY	Syracuse	Metro Grit Facility	2013	1	IIS	45.0	MGD							
CS-1819	13821	NY	Syracuse	Metro Grit Facility	2013	1	IIS	45.0	MGD							
CS-1820	13821	NY	Syracuse	Metro Grit Facility	2013	1	IIS	45.0	MGD							
CS-1839	14846	NY	Hempstead	Bay Park STP	2014	1	IIS	80.0	MGD	66						
CS-1840	14846	NY	Hempstead	Bay Park STP	2014	1	IIS	80.0	MGD	66						
CS-1841	14846	NY	Hempstead	Bay Park STP	2014	1	IIS	80.0	MGD	66						
CS-1842	14846	NY	Hempstead	Bay Park STP	2014	1	IIS	80.0	MGD	66						
CS-1850	15866	NY	Astoria	Bowery Bay WPCP	2015	1	IIIAS	80.0	MGD	84	102	102	1	255	Carbon Steel	304SS
CS-1851	15866	NY	Astoria	Bowery Bay WPCP	2015	1	IIIAS	80.0	MGD	84	102	102	1	255	Carbon Steel	304SS
CS-1852	15866	NY	Astoria	Bowery Bay WPCP	2015	1	IIIAS	80.0	MGD	84	102	102	1	255	Carbon Steel	304SS
CS-1862	15893	NY	Flushing	Flushing Bay CSO	2015	1	IIIAS	280.0	MGD	138	367		1.25	367	Carbon Steel	304SS

July 2017

Climber Screen® Installation List
Type IIS and IIIAS
NJ, NY, PA
2000-2015

Serial Number	Contract#	State	Location	Name	Year	Qty	Type	Design Flow Rate	Unit of Measure	Channel Width	Channel Depth	Max. Water Depth	Clear Spacing	Channel Invert to Operating Floor	Material - Non Wetted	Material - Wetted
CS-1863	15893	NY	Flushing	Flushing Bay CSO	2015	1	IIIAS	280.0	MGD	138	367		1.25	367	Carbon Steel	304SS
CS-1864	15893	NY	Flushing	Flushing Bay CSO	2015	1	IIIAS	280.0	MGD	138	367		1.25	367	Carbon Steel	304SS
CS-1865	15893	NY	Flushing	Flushing Bay CSO	2015	1	IIIAS	280.0	MGD	138	367		1.25	367	Carbon Steel	304SS
CS-1866	15893	NY	Flushing	Flushing Bay CSO	2015	1	IIIAS	280.0	MGD	138	367		1.25	367	Carbon Steel	304SS
				Total Number:		**106**										

Appendix B

ROMAG™ Installation List

(Source: WesTech Engineering, Inc.)

Job No.	Year	Location				Qty	Size	Equipment/Model
20855	2009	MUNCIE, IN WPCF	MUNCIE	IN	US	1		ROMAG CSO SCREEN RSW854
21335	2012	10TH STREET PUMP STATION	JEFFERSONVILLE	IN	US	1	1 Meters	ROMAG CSO SCREEN RSW115.54
21629	2013	FOURTH CREEK WWTP	KNOXVILLE	TN	US	1	1 Meters	ROMAG CSO SCREEN RSW-K1034
22138	2014	ARCHBALD WWTF	JERMYN	PA	US	1	1 Meters	ROMAG CSO SCREEN RSW724
22156	2014	CLINTON CSO LONG TERM CONTROL PLAN PHASE 1	CLINTON	IN	US	1	4 Meters	ROMAG CSO SCREEN RSW724
22430	2015	GLENS FALLS WWTP	GLENS FALLS	NY	US	1	16 MGD	ROMAG CSO SCREEN RSW-K724
22440	2015	LANCASTER NORTH PUMPING STATION	LANCASTER	PA	US	2	160 MGD	ROMAG CSO SCREEN RSW1254
22463	2016	TOWN BRANCH WET WEATHER STORAGE FACILITY	LEXINGTON	KY	US	1	57 MGD	ROMAG CSO SCREEN RSW864
22596	2016	WOLF RUN WET WEATHER STORAGE FACILITY	LEXINGTON	KY	US	1	7.3 MGD	ROMAG CSO SCREEN RSW824
22676	2016	KENTUCKY AVENUE INTERCEPTOR SEWER IMPROVEMENTS	FRANKFORT	KY	US	1	20 MGD	ROMAG™ CSO SCREEN RSW634
22742	2016	LOWER CANE RUN WET WEATHER STORAGE	LEXINGTON	KY	US	1	20 MGD	ROMAG™ CSO SCREEN RSW634
23133	2017	JOLIET CSO WET WEATHER TREATMENT FACILITY	JOLIET	IL	US	1		ROMAG™ CSO Screen RSW884
			Total Qty =			**13**		

Appendix C

Storm King® Vortex Separator Installation List

(Source: Hydro International)

Storm King Installation List

Plant / Job Name	Start-up Date	Contact	Plant Peak Flow, mgd	Equipment	Engineer	Rep	Appl
Hartford, CT WPCP	Jun-95		60.0	(2) 30' Storm King®	Blasland & Bouck Engineers	Aqua Solutions	CSO
Columbus, GA 19th Street - Uptown Park WRF Advanced Demostration Facility	Dec-95	Mike Burch 706-617-4981 mburch@cwwga.org	48 4.9	(6) 32' Storm King® (1) 8.5' FSU Grit King® (1) Classifier	Parsons Engineering Science	PEI	CSO-HW
Columbus, GA State Docks WRF South Commons	Sep-95	Mike Burch 706-617-4981 mburch@cwwga.org	48.0 4.0	(6) 35' Storm King® (2) 8' FSU Grit King® (2) Classifier	JJ & G	PEI	CSO
Lemont, IL WRP Wet Weather Treatment Facility and Reservoir	Jun-15		7.0	(1) 24' Storm King®	CH2M Hill	Drydon	CSO
Round Lake Beach, IL Round Lake Sanitary District	Jan-16		25.0	(1) 30' Storm King®	Christopher Burke Engineering 9575 W. Higgins Road, # 600 Rosemont, IL 60018	Drydon	CSO
Boonville, IN CSO North and South Basin	Feb-12		84.0	(2) 44' Storm King®	Midwestern Engineers	HPT	CSO
Bucksport, ME CSO	Apr-08	David Michaud, Opterator (207)469-0021 DEMichaud@aquaamerica.com	2.9	(1) 18' Storm King®	Wright Pierce Engineers	Aqua Solutions	CSO
Saco, ME CSO Treatment Facility	Nov-06	John Hart Superintendent (207) 282-3564	5.6 8.6	(1) 22' Storm King® (1) 12' ISU Grit King® (1) Type 2 Classifier	Deluca-Hoffman Associates	Aqua Solutions	HW/CSO
Redford, MI Rogue River CSO Retention Basin	Oct-96		61.0	(1) 35' Storm King®		Pumps Plus	CSO
New York, NY Corona Avenue	Oct-01		130.0	(1) 43' Storm King®	URS		CSO
Browndale, PA Clinton WWTP	Feb-06	Glenn Butler Bill Stanvitch Mike Dodgson	15.0	(1) 32' Storm King® (1) 6' ISU Grit King® (1) 12" Classifier	Montgomery Watson Harza	Sherwood Logan	CSO
Conyngham Borough, PA CSO	Nov-99	Jamie Wasilewski Operator (570)788-0608 ext.1	2.0	(1) 18' Storm King®	RDK Engineering	Sherwood Logan	CSO
Hazelton, PA Greater Hazelton JSC - CSO 002	May-11		14.0	(1) 30' Storm King®	Gannett Fleming	Sherwood Logan	CSO
Hazelton, PA Sixth & Ridge CSO	Jun-08	Chris Carcia Director of Operations (570)454-0851	2.6	(1) 18' Storm King®	Gannett Fleming		CSO

Appendix D

HYDROVEX® FluidSep Vortex Separator Installation List

(Source: Veolia Water Technologies)

VEOLIA
4105 Sartelon, Saint-Laurent, Québec, Canada, H4S 2B3
T: 514-334-7230
F: 514-334-5070
cso@veolia.com | www.hydrovex.com

HYDROVEX® FluidSep Vortex Separator
Installation List

	Country	Project	Qty	Type	Diameter (m)	Diameter (ft)	Inlet Flow Rate (L/s)	Inlet Flow Rate (MGD)	Installation Year
1	USA	Burlington, Vermont	1	2.5	12.20	40.03	2629	60	1990
2	USA	Decatur, Illinois, Lincoln Park	4	2.5	13.40	43.96	18230	416	1990
3	USA	Decatur, Illinois, 7th Ward	1	3	13.40	43.96	4951	113	1990
4	USA	Decatur, Illinois, Oakland Park	1	1.35	8.10	26.57	920	21	1991
5	USA	Saginaw, Michigan, 14th Street	3	2.5	11.00	36.09	8500	194	1991
6	USA	Saginaw, Michigan, Weiss	1	3	11.00	36.09	2848	65	1992
7	USA	Cincinnati, Ohio, Daly Rd.	1	3	12.20	40.03	2973	68	1993
8	USA	New York City, C80 #3	1	3	13.10	42.98	5663	129	1994
9	USA	Richmond, Virginia	1	1	2.60	8.53	150	3	1995
10	Canada	The Regional Municipality of Niagara, ON	2	2	12.00	39.37	2000	46	2006
11	USA	Riley Creek CSO, Mattoon, IL	1	2	6.40	21.00	657	15	2016
		Total	**17**	**Units**					

®Hydrovex®
www.hydrovex.com

Revised: July 13, 2017

Appendix E

SanSep Installation List

(Source: Echelon Environmental)

SANSEP^{tm} INSTALLATION & CONTACT LIST

Oct 2013

YEAR INSTALLED	LOCATION	OWNER	ENGINEER	DETAILS
1999	LOUISVILLE, KY CSO 50	LOUISVILLE & JEFFERSON CTY MSD Roddy Williams (now works for Strand Associates in Louisville) Derek Guthrie (now works for HDR in Louisville)	HDR (OMNI ENGINEER'ING) Gary Boblett Louisville & Jefferson Cty MSD Darren Thompson	Single PCS50_50; 10 cfs
2000	LOUISVILLE, KY CSO 108	LOUISVILLE & JEFFERSON COUNTY MSD	HDR (OMNI ENGINEERING)	Twin PCS70_70; 38 cfs
2002	AKRON, IN CITY LAKE CSO TREATMENT FACILITIES	AKRON, IN PUBLIC WORKS DEPT Marty Gearhart, Superintendent (574) 893-4674	COMMONWEALTH ENGINEERS Mark Sullivan, PE 7256 Company Drive Indianapolis, IN 46237 (317) 888-1177	PCSC56_40; 10 cfs. PCSC30_30; 4 cfs
2004	COHOES, NY N. NIAGARA AVE CSO OUTFALL	CITY OF COHOES, NY PUBLIC WORKS DEPT. Billy Kane, Maintenance Mgr. Office - (518) 488-8622 ALBANY REGIONAL SEWER DIST. Timothy S. Murphy, Permit Compliance Mgr. Office - (518) 447-1614	MALCOLM PIRNIE Robert E. Ostapczuk, PE 855 Route 146 Suite 210 Clifton Park, NY 12065 Office – (518) 250-7305	PCS100_100; 42 cfs
2004	WEEHAUKEN, NJ W5	NORTH HUDSON SEWER DISTRICT, WEEHAUKEN, NJ CONTRACT OPERATOR – OMI SERVICES JAMES HOWEY, Regional Mgr. 10 Brondesbury Drive Cherry Hill, NJ 08003 856-751-0213 Mohankumar Boraiah CH2M Hill 1600 Adams Street Hoboken, NJ 07030 Ph: 201-386-9847 Cell: 201-344-2783	CH2M-HILL Vincent Rubino, PE Kelly O'Connor, PE 119 Cherry Hill Road Parsippany, NJ 07054-1102 973-316-9300	Twin PCS70_80; 64 cfs

SANSEP^{tm} INSTALLATION & CONTACT LIST

Oct 2013

YEAR INSTALLED	LOCATION	OWNER	ENGINEER	DETAILS
2006	NIAGARA FALLS, ON, CANADA MUDDY RUN PUMP STA. HRT COMPARISON	NIAGARA FALLS REGION AUTHORITY		Single PCS40_30 Demonstration site with StormKing 8 ft diameter unit.
2008	FORT WAYNE CSO 58, FORT WAYNE, IN.	FORT WAYNE PUBLIC UTILITIES Wendy Reust, PE, CSO Program Mgr. One Main St., Room 480 Fort Wayne, IN 46801-1804 Office - 260-427-1367	CDM Karl E. Tanner, PE 151 N. Delaware St. Suite 1520 Indianapolis, IN 46204 Office - 317-637-5424	Twin PCS70_70; 10 cfs
2013	CSO 026 – HARBOR BROOK WETLANDS PILOT PROJECT	ONONDAGA COUNTY DEPT OF WATER ENVIRONMENT	CHA – CH2M-HILL JOINT Rich DeGuida, PE (CHA) 441 S Salina St. Syracuse, NY 13202 Office – 315-471-3920	Double 80-80, 44 cfs
2015	Taylorville, Illinois	City of Taylorville	Crawford, Murphy and Tilly Jeffery Large 217 572-1131	Single 70_70 with gravity underdrain
EUROPEAN INSTALLATIONS				
2005	LONDON	LONDON SEWER DEPT		PCS70_70; 450 l/sec
PACIFIC RIM				
1998	SYDNEY, AUSTRALIA		CDS TECHNOLOGIES PTY LTD.	PCS100_100; 1000 l/sec
2002	BRISBANE, AUSTRALIA		CDS TECHNOLOGIES PTY LTD.	PCS65_65; 400 l/sec
2002	SEOUL, S. KOREA, CHUNG GAE CSO FACILITY	SEOUL PUBLIC WORKS DEPT	KOGET ENVIRONMENTAL TECH.	6 each PCS100_100, 1,000 l/sec each

PWTech
PROCESS WASTEWATER TECHNOLOGIES LLC

Process Wastewater Technologies LLC • 105 Springbrook Place • Cary, NC 27518 • Phone: 919-858-8887 • Fax: 919-858-8983 • www.PWTech.us

Appendix F

ACTIFLO® Ballasted Flocculation Unit Installation List

(Source: Veolia Water Technologies)

KRÜGER

ACTIFLO Wet Weather Installation List

Jul-17

Installation Number	Name	Application	Location	Year Startup	Total Capacity	Number of Trains
1	St. Bernard, LA	ACTIFLO	At WWTP	2001	10	1
		BIOACTIFLO	At WWTP	2011	7.5	1
2	Bremerton, WA	ACTIFLO	Satellite	2001	10	1
3	Lawrence, KS	ACTIFLO	At WWTP	2003	40	2
4	Fort Smith, AR (P Street)	ACTIFLO	At WWTP	2004	31	1
5	Port Clinton, OH	Dual Mode ACTIFLO*	At WWTP	2004	24	2
6	Greenfield, IN	Dual Mode ACTIFLO*	At WWTP	2004	8	2
7	Fort Worth, TX	ACTIFLO	At WWTP	2005	110	2
8	Port Orchard, WA	ACTIFLO	At WWTP	2006	6.7	1
9	Cincinnati SSO 700, OH	ACTIFLO	Satellite	2006	15	1
10	Heart of the Valley (HOV) Kaukauna, WI	Dual Mode ACTIFLO*	At WWTP	2007	60	2
11	Salem, OR	ACTIFLO	Satellite	2007	50	2
12	Cincinnati, OH Sycamore Creek	ACTIFLO	At WWTP	2008	32	2
13	Tacoma, WA	ACTIFLO	At WWTP	2008	76	2
14	Geneva, NY	ACTIFLO	Satellite	2008	23	1
15	Nashua, NH	ACTIFLO	At WWTP	2008	60	2
16	Fort Smith, AR (Sunnymede Pump Station)	ACTIFLO	Satellite	2010	25	1
17	Newark, OH	ACTIFLO	At WWTP	2011	28	2
18	Wilson Creek, TX Phase 1	Dual Mode BIOACTIFLO*	At WWTP	2012	36	1
	Wilson Creek, TX Phase 2 (under construction)		At WWTP	2017	36	1
19	Lowell, IN	ACTIFLO	At WWTP	2013	10	1
20	Rock Creek, OR	Dual Mode ACTIFLO*	At WWTP	2013	30	2
21	Knoxville, TN	BIOACTIFLO	At WWTP	2013	11	2
22	Terra Haute, IN	ACTIFLO	Satellite	2016	16.5	1
23	Nappanee, IN (under construction)	ACTIFLO	Satellite	2017	5	1
24	Cox Creek, MD (under construction)	BIOACTIFLO	At WWTP	2017	12	1
25	McHenry, IL (under construction)	BIOACTIFLO	At WWTP	2017	10	1
26	DC Water (under construction)	ACTIFLO	At WWTP	2018	250	3

* Note: Dual mode means the ACTIFLO treatment train is used during dry weather flows for either primary or tertiary treatment.

Appendix G

DensaDeg® Ballasted Flocculation Installation List

(Source: Suez)

DENSADEG CSO EXPERIENCE

SUEZ has been providing high rate solids contact system for over 85 years. The new DensaDeg XRC™ has been born out of decades of improvements, starting with the original solids-contact clarifier, the Accelator, which was the first to incorporated internal sludge recycling. In the late 1980's the original DensaDeg clarifier was introduced to the market and continues to lead the industry for high-rate sludge ballasted and solids recirculation systems. While the DensaDeg XRC™ is recently introduced in 2015, it is merely an improvement upon a history of existing installations and operating principles, including over 2,400 installations over this span.

DENSADEG XRC

A year-long pilot study was conducted at Petersburg WWTP, VA, which included testing of the primary influent and secondary effluent from the plant. A case study summary is provided in **Addendum 3** of this proposal.

CSO/SSO REFERENCES

Below you will find a list of select installations for the original DensaDeg in CSO/SSO applications.

1 – McLoughlin Point WWTP, British Columbia, Canada – 64.5 MGD, 2019

2 – Shreveport WWTP, Louisiana – 40 MGD, 2006

3 – Toledo WWTP, Ohio – 232 MGD, 2006

Mr. Alan Ruffle, 419-727-2618

4 – Halifax WWTP, Nova Scotia, Canada – 92 MGD, 2005

5 – Edinborough, Scotland, UK -- 2002

6 – Aix-En-Provence (De La Pioline) WWTP, France – 25MGD, 2001

7 – Bourg-End-Bresse (De Majornas) WWTP, France – 22MGD, 2000

8 – Limoges WWTP, France – 23.8 / 33.6 MGD, 2000

9 – Meru (De L'Eau D'Amont) WWTP, France – 3.2MGD, 1999

10 – Saint-Chamond WWTP, France – 63.5MGD, 1999

11 – Colombes (Seine Centre) WWTP, France – 277MGD, 1998

12 – Bonneuil-En-France WWTP, France – 81.5 MGD, 1996

13 – Metz (Station Nord) WWTP, France – 68.5MGD, 1995

Appendix H

FlexFilter Installation List

(Source: WesTech Engineering, Inc.)

WESTECH

WWETCO FlexFilter™

Installation and Reference List

This partial list is composed of our key installations for this product. If you would like an expanded or more customized installation or reference list, please contact WesTech Engineering, Inc.

Plant Name	Location City/Sate	Quantity Size	Capacity Equipment Application	Contact Information
Springfield WWTP	Springfield, Ohio	11 30 ft. x 27 ft.	100 MGD Flex Filters CSO Treatment	Bill Young: Plant Superintendent, Springfield WWTP P: (937) 328.7626 E: byoung@springfieldohio.gov
Choctaw Pines	Dry Prong, Louisiana	2 2 ft. x 2 ft.	60 gpm FlexFilters Tertiary Treatment	Russell Turnage: Owner, Turnage Environmental Services P: (318) 447.5291 E: russellturnage@aol.com
Lamar WWTP	Lamar, Missouri	3 6 ft. x 6 ft.	2 MGD FlexFilter Lagoon Effluent Filtration	Rick Hornbeck: Water Plant Superintendent, City of Lamar P: 417-682-4480 E: rhornbeck@cityoflamar.org
Heard County	Franklin, Georgia	2 4 ft. x 4 ft.	0.75 MGD FlexFilters Tertiary Treatment	Jimmy Knight: Director, Heard County Water Authority P: (706) 594.2486 E: jknight@myhcwa.com
Weracoba Creek	Columbus, Georgia	3 6 ft. x 18 ft.	10 MGD FlexFilters Stormwater Treatment	Lynn Campbell: Vice President, Division of Water Resources, Operations, Columbus Waterworks P: (706) 649.3459 E: lcampbell@cwwga.org

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WESTECH

WWETCO FlexFilter™

Installation List

This partial list is composed of our key installations for this product. If you would like an expanded or more customized installation or reference list, please contact WesTech Engineering, Inc.

Plant Name	Location City/Sate	Quantity Size	Capacity Equipment Application
Solvay Polymer	Marietta, Ohio	3 6 ft. Diameter	1.44 MGD, Flex Filters Tertiary Treatment
Hope East WWTP	Hope, Arkansas	3 6ft. x13 ft	1.6 MGD, Flex Filters Tertiary Treatment
Hope West WWTP	Hope, Arkansas	3 6ft. x16 ft	2 MGD, Flex Filters Tertiary Treatment
Upper Tuscarawas WWTP	Akron, Ohio	10 6 ft. x 10 ft.	100 MGD, Flex Filters CSO Treatment
Springfield WWTP	Springfield, Ohio	11 30 ft. x 27 ft.	100 MGD, Flex Filters CSO Treatment
Choctaw Pines	Dry Prong, Louisiana	2 2 ft. x 2 ft.	60 gpm, FlexFilters Tertiary Treatment
Lamar WWTP	Lamar, Missouri	3 6 ft. x 6 ft.	2 MGD, FlexFilter Lagoon Effluent Filtration
Heard County	Franklin, Georgia	2 4 ft. x 4 ft.	0.75, MGD FlexFilters Tertiary Treatment
Weracoba Creek	Columbus, Georgia	3 6 ft. x 18 ft.	10 MGD, FlexFilters Stormwater Treatment

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13 Borough of Fort Lee Development and Evaluation of Alternatives Report (Attached)

Development and Evaluation of Alternatives Report

Borough of Fort Lee

June 2019

Issue and Revision Record

Revision	Date	Originator	Checker	Approver	Description
0	6/25/2019	G. Grey	Y. Wu	G. Grey	Original report

Document reference:

Information class: Standard

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Certification

Fort Lee Development and Evaluation of Alternatives Report

Submitted on behalf of the following participating Permittee

by the Borough of Fort Lee

NJPDES Number NJ0034517 (Borough of Fort Lee)

NJPDES Certification:

"I certify under penalty of law that this document and all attachments were prepared either: (a) under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted; or (b) as part of a cooperative effort by members of a hydraulically connected system, as is required under the NJPDES Permit, to provide the information requested. Based on my inquiry of the person or persons who manage the system, or those persons directly responsible for gathering the information, the information submitted is, to the best of my knowledge and belief, true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for purposely, knowingly, recklessly, or negligently submitting false information."

Permittee: _____ 6/26/19

Alfred R. Restaino, Borough Administrator Date

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Executive Summary

This report is being provided as one of several reports required for the Fort Lee Combined Sewer Overflow (CSO) Long Term Control Plan (LTCP) under NJPDES Permit No. NJ0034517. It evaluates control alternatives but makes no claims as to the ultimate control that will be selected. Costs are also developed for some control alternatives but these are comparative costs developed solely for making comparisons of the capital and operating costs of the alternatives. Fort Lee has not yet committed to providing for the costs associated with control of their CSOs.

The borough of Fort Lee was issued a Combined Sewer Overflow (CSO) permit (NJPDES Permit No. NJ0034517) by the New Jersey Department of Protection in 2015. The permit requires the development of a Long Term Control Plan for reducing CSO's by June 1, 2020. There are several requirements for reports and other actions. This report, Development and Evaluation of Alternatives Report, discusses all the alternatives available for CSO reduction and selects alternatives that could be used to reduce CSOs in Fort Lee.

The Borough of Fort Lee comprises approximately 1,600 acres which is serviced by combined and separately sewered areas. The combined sewer system consists of approximately 640 acres discharging to three pump stations and two CSO outfalls. The CSO outfalls are activated in rain storms.

One significant improvement was made in 2016 to the combined collection system that impacts CSOs. In 2016 a sewer infrastructure project servicing the new Hudson Lights project revised the sewer collection system. Before 2016, the Lower Main Pump Station sent pumped flow to a 12 inch pipe by gravity to the Palisade Terrace Pump Station which would then pump to the interceptor. After 2016, the flow from the Palisade Terrace Pump Station was rerouted to a new 12 inch pipe that discharges directly to the BCUA interceptor. In addition to the rerouting the flow, the pump station capacity was upgraded from 2 MGD to 5 MGD. This change in the combined sewer system reduced annual CSOs at the Palisade Terrace outfall from 11.73 MGD to 4.17 MGD. This reduction in CSOs corresponds to 84.7% CSO capture which almost achieves EPA's CSO control policy objective of 85% capture.

The CSO outfalls are in a neighboring town, Edgewater, on the Hudson River. The Hudson River is an SE2 water body in the vicinity of Fort Lee with a fecal coliform criteria (geometric mean) of 770 cfu/100 mL. Currently, sampling programs show the water quality to be in compliance with this criteria; therefore, water quality is not a driver for CSO control.

If Fort Lee is required to reduce CSOs further, the alternatives that they could use to reduce or eliminate CSOs are gray infrastructure alternatives such as disinfection, high rate filtration with disinfection and storage tanks. Peracetic Acid (PAA) would be the likely disinfection process selected because of the long shelf life of PAA and the non-toxic nature of any residual concentration. Testing of the

PAA disinfection process with and without high rate filtration would likely be performed to gather design data.

Control alternatives that could reduce but not eliminate CSOs are sewer separation and green infrastructure. Fort Lee may use sewer separation or green infrastructure to increase CSO capture to 85%. If more CSO capture is required by NJDEP then we will consider the gray infrastructure alternatives of disinfection, high rate filtration with disinfection and storage tanks.

The lifecycle cost for achieving 85% capture by sewer separation or GI ranges from $6,250,000 to $10,000,000. For tank storage, the most expensive CSO control alternative considered, the range is $47,000,000 to $167,000,000. For filtration with disinfection the cost range from $36,000,000 to $85,000,000. If disinfection alone proves to be a viable option then costs may range from $3,720,000 to $7,270,000.These evaluations of alternatives will serve as a base for the consideration and development of final selected CSO control plan in Fort Lee. We believe the most cost effective solution for meeting water quality objectives and complying with the EPA CSO control policy will be GI, sewer separation or disinfection with PAA.

1 Introduction

This report is being provided as one of several reports required for the Fort Lee Combined Sewer Overflow (CSO) Long Term Control Plan (LTCP) under NJPDES Permit No. NJ0034517. It evaluates control alternatives but makes no claims as to the ultimate control that will be selected. Costs are also developed for some control alternatives but these are comparative costs developed solely for making comparisons of the capital and operating costs of the alternatives. Fort Lee has not yet committed to providing for the costs associated with control of their CSOs.

1.1 Sewage System Description

Fort Lee's wastewater services spreads to over 1,600 acres of which 640 acres are serviced by a combined system. The service area extends along the Palisade Ridge adjacent to the Hudson River. The Fort Lee CSS includes three (3) pump stations, their regulators, and two (2) discharge points. The three (3) pump stations are Palisade Terrace Pumping Station (PTPS), Lower Main Pumping Station (LMPS), and Bluff Road Pumping Station (BRPM). During the 2017 flow metering, flows at these pump stations were metered. The tributaries to these pumping stations are described in the following sections.

1.2 Service Area Land Use Data

The sanitary flow in Fort Lee is primarily residential with some commercial flow. Figure 1-1 displays the various land use types in Fort Lee. There are no planned changes to land use type in the future.

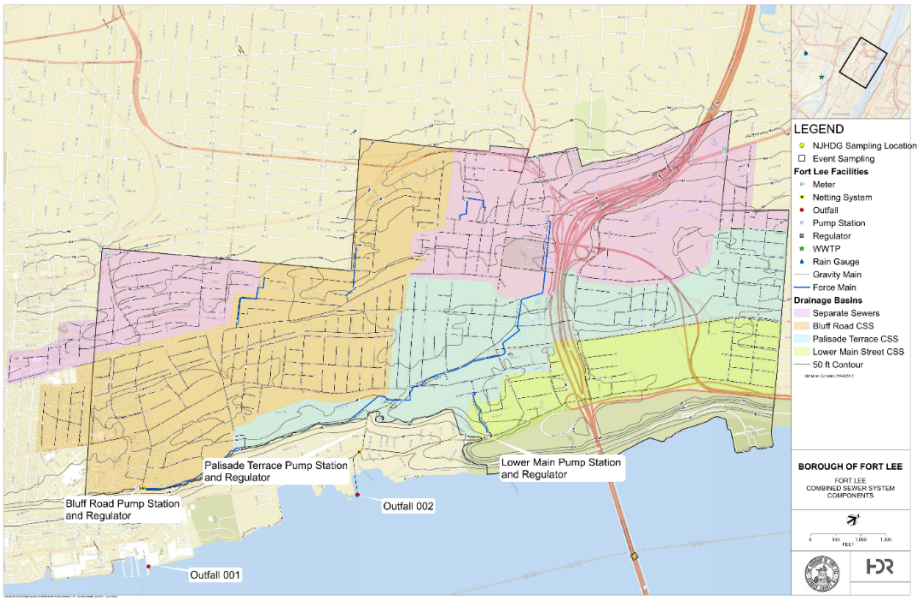

Figure 1-1. Land Use Type in Fort Lee

1.2.1 Palisade Terrace Pumping Station (PTPS)

The PTPS collects dry weather flows from the north-western portion of the Borough that has an area of about 340 acres. The land use in the drainage area is mostly residential. The PTPS drainage area has the population of approximately 9,100 people who contribute on average of 1.23 million gallons per day of dry weather flow.

Most of this area is separately sewered with the exception of McCloud Drive. Additional Inflow and Infiltration (I&I) sources (e.g., sump pumps, groundwater infiltration, etc.) contribute flows to the pump station during wet weather periods. The PTPS pumped flows discharge to the BCUA interceptor starting at the intersection of Route 4 and Edwin Avenue.

1.2.2 Lower Main Pumping Station (LMPS)

The LMPS collects flows from the north-eastern portion of the Borough and has a drainage area of about 167 acres. The drainage area is primarily residential. The LMPS has approximately 2,500 people who contribute on average 0.64 million gallons per day of dry weather flow. This drainage area is separately sewered with the exception of English and Cedar Street. I&I is also prevalent in this drainage area. Before 2016 the LMPS sent pumped flow to a 12 inch pipe on Parking Avenue, from which the flow traveled by gravity to the PTPS. After 2016, the flow from the pump station was rerouted to a 12 inch pipe that discharges to the BCUA interceptor. In addition to the rerouting the flow, the pump station capacity was upgraded from 2 MGD to 5 MGD.

1.2.3 Bluff Road Pumping Station (BRPS)

The BRPS collects dry weather flow from about 493 acres from the southern portion of the Borough. This drainage area is primarily residential. The BRPS has approximately 12,100 people who contribute on average 1.83 dry weather flow.

Unlike the other drainage areas, this area is serviced mostly by combined sewers with the exception of Anderson Avenue and the areas north of the street. The pump station can pump 6 MGD of flow.

1.2.4 Direct Drainage to BCUA Interceptor

Within the Borough of Fort Lee there are two drainage areas that drain directly to the BCUA interceptor Sewer. BCUA-1 combines with the Bluff Road Pumping Station and drains downstream of the Overpeck Valley Sewer. BCUA-1 has an approximate population of 3,500 with an average flow of 0.5 MGD. BCUA-2 directs to the Fort Lee East Interceptor Sewer and combines with both Lower Main and Palisades Pumping Station upstream of the Overpeck Valley Trunk

Sewer. BCUA-2 drainage area contains approximately 7,800 people with an average flow of 2 MGD. Both areas are separated and primarily residential.

1.2.5 Combined Sewer System Characteristics

Prior to 2016 the Fort Lee model discharged flow from the Lower Main pump station to the Palisades regulator. In 2016 this was revised with the construction of the Hudson Lights project. Lower Main Pump Station was upsized and a new 10" line was installed connecting it directly to the BCUA Interceptor. The overflow from the Lower Main Pump Station still combines with the overflow from the Palisades Pump Station and discharges to Outfall 2. The new pumps at Lower Main were built to handle additional flows from new housing development, Hudson Lights, in the Lower Main drainage area.

The changes to the Fort Lee model include the following:

1. Population and landuse updates.
2. Increased capacity of the Lower Main Station.
3. Redirection of the Lower Main Pump Station discharge from the Palisade Pump Station directly to the BCUA Interceptor.
4. Incorporation the Hudson Lights 16 acre redevelopment project.
5. Addition of seasonal variability to infiltration and inflow flows.

After these changes were made the model was recalibrated to flow data collected during October to December 2017 and validated to BCUA flow metering data from March 1, 2017 to August 27, 2017. Once the model was calibrated and validated a one year simulation was performed using the rainfall design year of 2004. The simulation was performed before and after the redirection of the Lower Main Pump Station discharge to the BCUA Interceptor to see the effect of this change. Overflows for both conditions are summarized below. Before the Lower Main flow was redirected ti the interceptor the simulation resulted in 38 overflows totaling 11.73 MGD at the Palisade netting facility. By redirecting the discharge directly to the interceptor, overflows were reduced to 22 and total volume was reduced to 4.17 MGD. This is a 42% reduction in overflows and a 64.5% reduction in overflow volume. This model revision simulates how the collection system currently works.

Table 1-1. Comparison of Model Results Before and After Revising the Operation of the Lower Main Pump Station

Condition	Outfall 001 (Bluff Road)		Outfall 002 (Palisade Terrace)	
	Overflows	Volume	Overflows	Volume
2004 before redirection of Lower Main	60	77.20	38	11.73
2004 after redirection of Lower Main	60	77.20	22	4.17

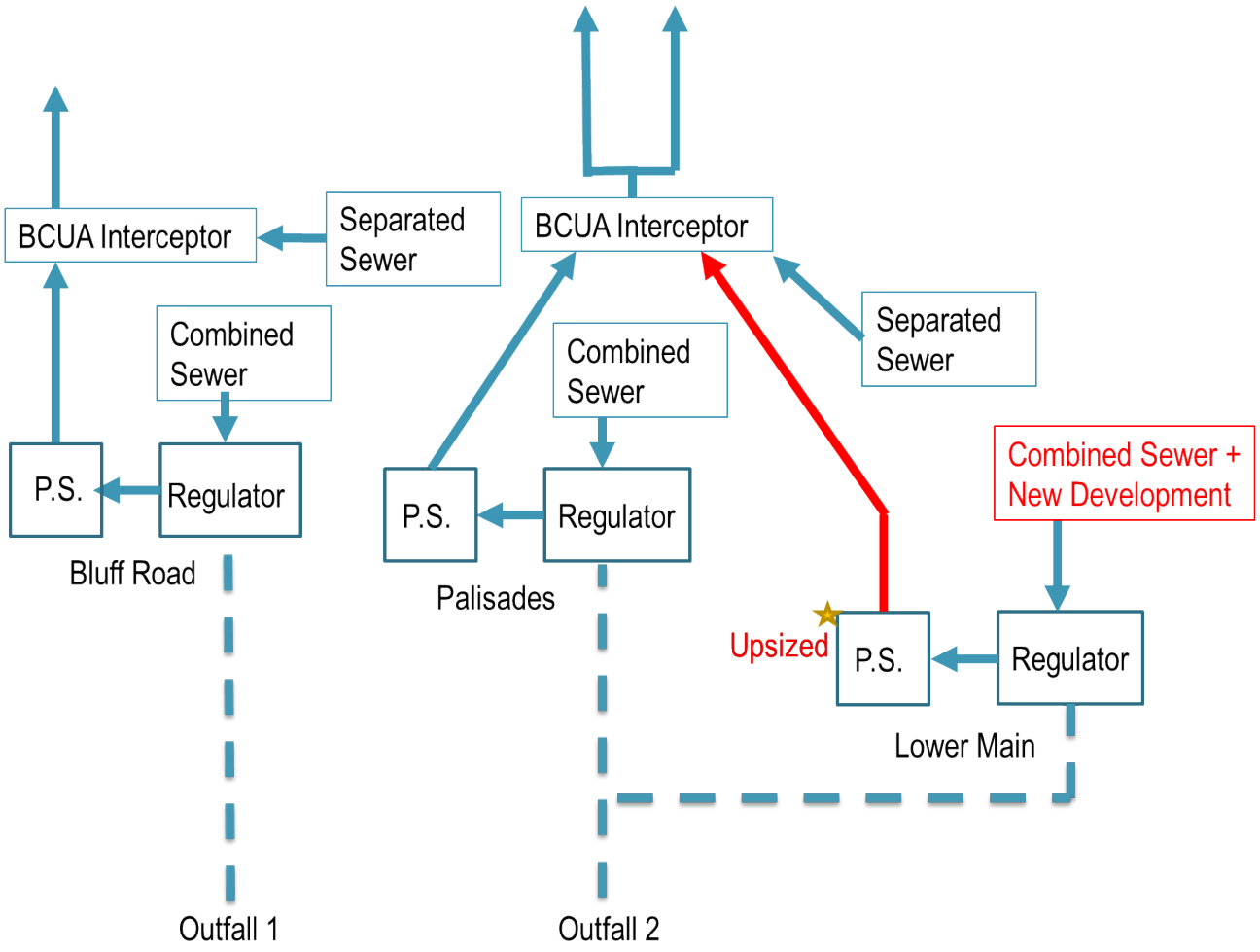

Figure 1-2. Present Configuration of Fort Lee Collect System

1.3 CSO Regulators and Control Facilities

Netting system are utilized by Fort Lee to remove floatables from the CSOs and meet USEPA "Nine Minimum Controls" and the Long Term Control Plan requirements. The netting systems were installed in 1998. The netting systems are inspected at least monthly by DPW personnel. Inspections are undertaken prior to anticipated storm events, and after significant storm events.

The netting systems are in-line TrashTrap systems manufactured by Fresh Creek Technologies, Inc. who is currently owned by Storm Trap. Each netting facility contains two nets. The netting units are installed in in-line chambers and are installed in line with the combined sewer system pipe. A fixed hydraulic relief

screen located upstream of the nets assures screening of the flow under all conditions and provides additional system capacity. The screen is inclined in the direction of flow so that any debris caught on this screen falls into the nets as the water level in the chamber recedes. The screen will also work as a weighted relief valve, if required, to reduce back pressure. Grating under the nets allows them to drain dry. Debris is captures and contained in disposable nets. The disposable nets and support frame are housed in a rack assembly installed in the chamber.

Net maintenance and change outs are scheduled based on periodic visual inspection. The system is maintained by Fresh Creek/Storm Trap through ground-level lockable access doors on the top of the netting chamber. There is no confined-space entry required during routine service because the disposable nets and frames are lifted from the chamber to perform the net change-out above ground. A hoist truck for changing the nets and a container for holding the full nets are used for maintenance. A crew of two typically accomplishes the net change-out. The full nets are disposed of at an approved facility.

1.4 Recent Reports or Plans

The 2004 NJPDES permit for Fort Lee required the Borough to develop a Long Term Control Plan (LTCP) in accordance with the National CSO Control Policy. This phase of the CSO program required development and evaluation of the feasibility of a range of control alternatives to reduce CSO frequency and pollutant loadings pursuant to the Federal Clean Water Act (CWA) goals and consistency with EPA's CSO Control Policy. It resulted in the following studies and reports:

- Interim Service Area and Land Use Report for Fort Lee, March 2007;
- Rainfall Monitoring Study Report for Borough of Fort Lee, March 2007;
- Interim System Inventory and Assessment Report for Borough of Fort Lee, March 2007;
- Interim Combined Sewer System Modeling Report for Borough of Fort Lee, March 2007;
- Combined Sewer Overflow Interim Monitoring Report for Fort Lee, March 2007; and
- Cost and Performance Analysis Report for Borough of Fort Lee, March 2007.

These reports presented the development of the Long Term Control Plan in 2007.

Two subsequent reports were used to define the reconfiguration for the combined sewer system:

- Engineer's Design Report For Lower Main Pumping Station and Force Main, March 2013; and
- Calculations Report for Flow Reduction to Lower Main Pump Station Due to Sewer-Storm Separation at Fort Lee Redevelopment Projects, April 2013.

The following reports were prepared for the development of the 2020 LTCP:

- New Jersey CSO Group Baseline Compliance Monitoring Program Report, June 30, 2018;
- Identification Of Sensitive Areas Report, May 2018;
- BCUA CSO Group Public Participation Program Report, June 27, 2018 revised January 4, 2019;
- Public Participation Program Report for the Borough of Fort Lee, January 18, 2019; and
- Sewer System Characterization Report for the Borough of Fort Lee, New Jersey, June 29, 2018 revised March 12, 2019.

1.5 Approach to Development and Evaluation of Alternative Controls

The USEPA recommends a three-step sequence for the development of alternatives:

1. Definition of water quality objectives.
2. Definition of a range of CSO control goals to meet the CSO component of the water quality goals.
3. Development of alternatives to meet the CSO control goals.

2 General Information

2.1 Public Participation Process

According to the NJPDES permit, each permittee is required to establish a Supplemental CSO team comprised of members of the public and other stakeholders. The Supplemental CSO Team works with the permittee's consultants and assigned staff to act as a liaison between the general public and the decision makers for the permittee. The Borough also created a local CSO Team to carry out the same functions. The goals of the Supplemental and Local CSO Teams consists of the following elements:

- Meet periodically to assist in the sharing of information and to provide input to the planning process;
- Review the proposed nature and extent of data and information to be collected during LTCP development;
- Provide input for consideration in the evaluation of CSO control alternatives; and
- Provide input for consideration in the selection of those CSO controls that will cost effectively meet the Clean Water Act requirements.

The BCUA CSO Group established a Supplemental CSO Team by posting an invitation on its website providing notification of the project and inviting individual members, or interest groups with the community to join. The website invitation was posted for approximately a one-month period, but there was no public response.

In an effort to obtain regional input the BCUA extended a personal invitation to the Hackensack River Keeper, who accepted. In addition, each member of the BCUA CSO Group was invited to designate two members of their municipality or supplemental team to join the Regional Team. The members identified by Fort Lee are:

- Jan Goldberg;
- Bob Applebaum; and
- Sal Pagano.

Meetings for the BCUA CSO Group were scheduled on a roughly quarterly basis. Attendance at each meeting was good, with most of the members attending each meeting. The CSO treatment objectives, and alternatives have been discussed at these meetings.

Mr. Goldberg, Mr. Applebaum, and Mr. Pagano are also on the Local CSO Team. Three local team meetings were held with all members in attendance at each. In

addition to the team members, the Mayor, Council members, Borough Engineer and Department of Public Works members also attend some meetings.

2.2 Approximated Future Conditions

One large redevelopment project in Fort Lee is The Modern and Hudson Lights complex which have been under construction since 2013. The Modern is a residential skyscraper complex with two buildings near the western end of the George Washington Bridge. The towers are part of a larger 16 acre urban renewal project for the long vacant parcel. They are separated by a 1.75-acre public park. An adjacent project called Hudson Lights will feature retail, including a three-screen movie theater, as well as a hotel and office space. The complex will contain approximately 1400 residential units.

Figure 2-1. Photo of the Modern A and B and Hudson Lights

2.2.1 Projections of Population Growth

The population growth was estimated for 2045 and was based on the New Jersey Transportation Planning Authority. The population for Fort Lee is projected to be 43,747.

3 Water Quality Objectives

The New Jersey CSO Group submitted a report to NJDEP presenting the results of the Baseline Compliance Monitoring Program that was conducted in 2016 and 2017. These efforts are described in Baseline Compliance Monitoring Program Quality Assurance Project Plan, 2016 and in NJCSO Group Compliance Monitoring Program Report, 2018.

3.1 Applicable Water Quality Standards

The sampling program was not designed to provide an adequate data volume for assessing attainment of water quality standards, which would have required five samples per month at each sampling location to compute monthly geometric means. However, a review of the data collected can indicate the likelihood of attainment in the Hudson River in the vicinity of Fort Lee.

Figure 3-1 presents the data that was collected during the sampling program for the Hudson River at Fort Lee. This location is a classified as a SE2 water body with a fecal coliform standard of 770 cfu/100 mL as a geometric mean. Data are presented as open circles for surface data, filled gray circles for mid-depth data, and filled black circles for bottom data. Secchi depth does not fall into a specific depth category, but is plotted with filled black circles. Turbidity is shown on the same panel as Secchi depth and is presented with filled green circles. All samples collected in this program were in compliance with fecal coliform standard of 770 CFU/100 mL.

3.2 Range of CSO Goals Being Evaluated

Fort Lee will specify its intention to comply with the Presumptive or Demonstration Approaches in its final LTCP. The Presumptive Approach implies compliance with the CSO treatment goals if one of three conditions are met. The Demonstration Approach allows a municipality to demonstrate compliance without meeting the criteria.

The preliminary indication is that it will comply with the Presumptive Approach with the understanding that capturing 85% of the CSO will be deemed to be in compliance with the NJPDES permit and EPA CSO Policy. The Policy states:

> *A program that meets any of the criteria listed below would be presumed to provide an adequate level of control to meet the water quality-based requirements of the CWA, provided the permitting authority determines that such presumption is reasonable in light of the data and analysis conducted in the characterization, monitoring, and modeling of the system and the consideration of sensitive areas described above. These criteria are provided because data and modeling of wet weather events*

often do not give a clear picture of the level of CSO controls necessary to protect WQS.

i. No more than an average of four overflow events per year...

ii. The elimination or the capture for treatment of no less than 85% by volume of the combined sewage collected in the CSS during precipitation events on a system-wide annual average basis...

iii. The elimination or removal of no less than the mass of pollutants, identified as causing water quality impairment..., for the volumes that would be eliminated or captured for treatment under paragraph ii... " (Section II.C.4.a.)

Fort Lee will need to understand NJDEP's thoughts on the Presumptive Approach criteria and if they are receptive to 85% CSO capture as being in compliance. It is understood that the post construction compliance sampling program, subsequent to the LTCP, will also have to demonstrate compliance.

Figure 3-1. Water Quality Results for the Hudson River in Fort Lee near George Washington Bridge

4 Screening of CSO Control Technologies

4.1 Introduction

This section of the report focuses on the technology screening process and the evaluation of CSO control alternatives as per the requirements of the NJPDES Permit. In order to determine the appropriate CSO control technologies, a preliminary comprehensive review of combined sewer overflow technologies was completed to determine those technologies that have the greatest potential to meet the requirements of the NJPDES Permit. This screening of technologies complies with the requirements of the Combined Sewer Overflows (CSOs) Control Policy Section II.C.4 and is consistent with the EPA's "Guidance for Long Term Control Plan." The Alternatives Evaluation shall consist of:

- Technology Screening Process; and
- Identification and Evaluation of Specific CSO Control Alternatives.

This screening of technologies does not consider cost or the cost effectiveness, and is only meant to exclude those CSO control technologies not technically or physically applicable to the service area. The results of this screening have brought several CSO control technologies forward for more detailed consideration in the development of the LTCP. These control technologies are further discussed in Section D of this report.

4.1.1 Water Quality and CSO Control Goals

With respect to water quality, control technologies are screened for their effectiveness on addressing pollutants of concern and CSO control goals to achieve compliance with the CWA, including:

- Reducing the count of fecal coliform colonies;
- Reducing the count of Enterococcus colonies;
- Reducing the count of Escherichia coli colonies; and
- CSO discharge volume reduction.

4.1.2 Evaluation Methodology Used for This Study

The CSO control technologies evaluated in this section have been assigned a value based on their effectiveness at reaching primary and secondary CSO control goals. Descriptions of the goal effectiveness categories are detailed below:

- High: The CSO control technology will have a significant impact on this CSO control goal and is among the best technologies available to achieve

that goal. These technologies may be considered for further evaluation for this reason.

- Medium: This technology is effective at achieving the CSO control goal, but is not considered among the most effective technologies to achieve that goal.
- Low: This technology will have a minor impact on this CSO control goal. These technologies will need other positive attributes to be considered for further evaluation.
- None: The CSO control technology will have zero or negative effect on the CSO control goals.

CSO control technologies will be recommended for further evaluation based on multiple factors. The first factor will be the goals effectiveness value that generally quantifies the impact a technology will have towards achieving a water quality goal. These goal-effectiveness values are described above. The second factor is whether or not the NJPDES Permit requires further investigation of a technology. The permit identifies certain technologies that must be evaluated further. The third and final factor in determining whether a technology will be evaluated further is the current or future implementation and operation of that technology. If the technology is currently in place, will be implemented, or is mandated by the Nine Minimum Controls, then an evaluation is unnecessary.

Potential CSO control technologies generally fall into the following broad categories:

- Source Controls: Green infrastructure; public and private infiltration and inflow (I/I) reduction and removal; sewer separation; and best management practices (BMPs)/Nine Minimum Controls, including floatables control.
- Collection System Controls: Gravity sewers; pump stations; hydraulic relief structures; in-line storage; outfall relocation/consolidation; and regulator/diversion structure modification.
- Storage Technologies: Above and below ground tanks; and tunnels.
- Treatment Technologies: Screening and disinfection; vortex separation; retention/treatment basins; high rate clarification; and satellite sewage treatment.

A CSO technology that is highly effective in one or all evaluation factors will likely be recommended for further investigation. A CSO technology that does not reach a “medium” effectiveness water quality goals will not be recommended for further evaluation.

The following discussion is structured to closely follow the order of CSO technologies listed in the NJPDES Permit. A summary of technologies

recommended for further investigation is provided in Section C.9 Screening of Control Technologies of this report.

4.2 Source Control

The EPA defines source controls as those that impact the quality or quantity of runoff entering the combined sewer system. Source control measures can reduce volumes, peak flows, or pollutant discharges that may decrease the need for more capital-intensive technologies downstream in the CSS. However, source controls typically require a high level of effort to implement on a scale that can achieve a measureable impact. Source controls discussed in the following section will include both quantity control and quality control measures.

4.2.1 Stormwater Management

Stormwater management controls consist of measures designed to capture, treat, or delay stormwater prior to entering the CSS.

4.2.1.1 Street/Parking Lot Storage (Catch Basin Control)

Street and parking lot storage can be accomplished by modifying catch basins to restrict the rate of stormwater runoff that enters the CSS. A portion of the stormwater runoff that would otherwise immediately enter the CSS is allowed to pond on streets or parking lots for a period of time before entering the CSS. This control measure can be very effective at reducing peak flows during wet weather events, when most CSOs occur. However, this practice typically faces strong public opposition and can lead to hazardous road conditions if not managed properly (e.g., hydroplaning, ice formation during winter months, etc.).

4.2.1.2 Catch Basin Modification (Floatables Control)

Catch basin modifications consist of various devices that prevent floatables from entering the CSS. Inlet grates can reduce the amount of street litter and debris that enters the catch basin. Other modifications such as hoods, submerged outlets and vortex valves alter the outlet pipe hydraulics and keep floatables from exiting the catch basin and continuing downstream. These devices also provide a water seal for containing sewer gas. The success of a catch basin modification program is dependent on having catch basins with sumps deep enough to install hood-type devices. A potential disadvantage of catch basin outlet modifications and other insert-type devices is the fact that retained materials could clog the outlet if cleaning is not performed regularly.

4.2.1.3 Catch Basin Modification (Leaching)

Catch basin modifications for leaching consist of catch basin base and riser sections that permit infiltration of stormwater into the ground. Leaching catch basins are generally installed in a geotextile and crushed stone lined excavation.

Leaching catch basin installations are limited to highly permeable soils and should not be installed in series with other drainage structures. Leaching catch basins can be installed with or without an outflow pipe. Basins without an outflow can overflow into streets and parking lots and then freeze under excessive storm events or if soils decrease permeability over time. These control measures function much like an infiltration basin without an emergency overflow pipe. In order to avoid this adverse feature, an outflow pipe should be necessary in all leaching modified catch basins unless there is minimal flow to the basin, and a low overflow damage risk to the surrounding area.

4.2.2 Public Outreach Program

Public education and outreach is a non-structural control measure aimed at limiting the negative effects of certain human behavior on the CSS. Promoting certain human actions and discouraging others can impact the quality and quantity of water discharged to the CSS. A collaboration of entities who own and operate combined sewer systems within the Passaic Valley Sewerage Commission (PVSC) and North Bergen Municipal Utilities Authority (NBMUA) services areas have established the Clean Waterways, Healthy Neighborhoods initiative. The initiative aims to foster public awareness by keeping the public informed of the efforts being taken to reduce the water quality impact of CSOs on the receiving waters in the area. Additional information is available on the following website: https://www. nicleanwaterwavs.com/.

4.2.2.1 Water Conservation

Water conservation in CSS areas can reduce the volume of direct discharges to the system. Water conservation measures include the installation of low-flow fixtures, education to reduce water waste, leak detection and correction, and other programs. Although this measure has the potential to decrease CSS flows, it has very little impact on peak flows, which cause most CSOs.

4.2.2.2 Catch Basin Stenciling

Stenciling consists of marking catch basins with symbols and text such as, “Drains to the River” or “Only Rain Down the Storm Drain”. This measure can help increase public awareness of the sewer system and discourage the public from dumping trash into the CSS, which can cause blockages and lead to CSOs.

Catch basin stenciling is only as effective as the public’s understanding and acceptance of the program. Catch basin inlet grates have the equivalent effect while not relying on public cooperation.

4.2.2.3 Community Cleanup Programs

Community cleanup programs are an inexpensive and effective way to reduce floatables entering the CSS and provided educational benefits to the community.

Cleanup activities can be organized by local businesses, non-profit organizations, and student chapters at all levels. It is a great way to raise the sense of community spirit and environmental awareness.

4.2.2.4 Public Outreach Programs

Public outreach programs help raise citizens' awareness of water quality and other environmental issues. Programs educate citizens about CSS's and encourage people to do their part to reduce the grease, toxic chemicals, and floatables from entering local waterways. These items are currently discussed during the project Supplemental CSO Team Meetings (public meetings) and information presented in meetings is available as handouts.

4.2.2.5 FOG Program

Fats, oils and grease (FOG) are not water soluble and will buildup and clog sewer and drainage pipes, resulting in messy, costly sanitary sewer overflows. These overflows are bad for commercial and retail businesses, the environment, and public health. FOG programs often consist of food service establishment inspection, installation of Grease Removal Devices (GRDs) and development of a preferred pumper program for proper maintenance of GRDs. However, FOG programs have little effect on the amount of bacteria in the collection system and do not provide any flow reductions.

4.2.2.6 Garbage Disposal Restrictions

Garbage disposals provide a convenient means for residences and businesses to dispose of food waste. However, the use of garbage disposals increases the amount of food scrap entering the sewer system and is known to cause blockages and decrease the flow capacity in the CSS. Restricting garbage disposal usage has the potential to decrease the number of blockages that occur each year. Garbage disposal restrictions require an increased allocation of resources for enforcement and can face considerable public resistance. Furthermore, this practice does very little to reduce wet weather CSO events or decrease bacteria loads.

4.2.2.7 Pet Waste Management

When pet waste is not properly disposed of, it can be carried away by stormwater runoff and washed into storm drains or nearby streams. Since storm drains do not always connect to treatment facilities, untreated animal feces often end up in waterways, causing significant water pollution. An effective pet waste management program can help increase public awareness and encourage proper waste disposal. This is a low cost, long term program that has the potential to reduce bacteria loads to both the CSS and directly to local streams.

4.2.2.8 Lawn and Garden Maintenance

Failure to apply chemical treatments to lawns or gardens per USEPA guidelines may lead to ineffective treatment and contamination of the waterways through runoff or groundwater. A public outreach program that explains the guidelines and the reasons they exist may help reduce waterway contamination. This information is currently available to the public on the following USEPA website: https://www.epa.gov/safepestcontrol/lawn-and-garden. Runoff that contains chemical treatments can contribute to decreased water quality downstream of the CSS in the receiving waters.

4.2.2.9 Hazardous Waste Collection

Improperly disposed hazardous waste can find its way into stormwater runoff and into storm drains and waterways. Hazardous waste that ends up in waterways does not necessarily end up in a treatment facility and can cause significant surface water pollution. To prevent this, household hazardous waste collection events can be scheduled a few times every year to allow the community to properly dispose of any hazardous waste.

4.2.3 Ordinance Enforcement

4.2.3.1 Construction Site Erosion & Sediment Control

Construction site erosion and sediment control involves management practices aimed at controlling the transport of sediment and silt by stormwater from disturbed land. Erosion and sediment control has the potential to reduce sediment loads to both the CSS and directly to streams, and can help reduce server cleanout Operation and Maintenance (O&M) costs. The N.J.S.A. 4:24-39, NJ Soil Erosion and Sediment Control Act, requires all construction activities greater than 5,000 square feet to complete an application for certification of an erosion and sediment control plan for activities during construction.

4.2.3.2 Illegal Dumping Control

Illegal dumping is the disposal of trash or garbage by dumping, burying, scattering, or unloading trash in an unauthorized place, such as public or private property, streets or alleys, or directly into the CSS. When it occurs, illegal dumping contributes a considerable amount of floatables to stormwater runoff, as well as a moderate amount of bacteria, settleable solids, and other pollutants. Enforcement of illegal dumping regulations is being led by State Park Police & Conservation Officers and the NJDEP Department of Compliance & Enforcement.

4.2.3.3 Pet Waste Control

As described in the previous section, pet waste can be a significant contributor of bacteria to stormwater. Public education and outreach programs can help raise public awareness and reduce the level of improper waste disposal. Additional gains can be made through enforcement of the pet waste ordinances, which can be an effective tool in achieving public compliance. Significant resources would need to be devoted to enforcement to achieve similar improvements to Pet Waste Management, which requires very few resources to implement.

4.2.3.4 Litter Control

Litter consists of waste products that have been disposed of improperly in an inappropriate area. Litter is easily washed into the collection system during wet weather events, which increases the amount of floatables in the system. Strict enforcement of the litter control ordinances can help to curb violations and decrease the amount of floatables that make their way into the CSS. Similar to Pet Waste Control, public outreach and education is a more effective use of resources to achieve similar water quality improvements.

4.2.3.5 Illicit Connection Control

An illicit discharge is any discharge to the municipal separate storm sewer system (MS4) that is not composed entirely of storm water, except for discharges allowed under a NPDES permit or waters used for firefighting operations. Illicit connections can contribute polluted water, solids, and trash to the stormwater system, where it is eventually discharged to the environment without receiving proper treatment. These connections can be reduced through the implementation of an illicit discharge detection and elimination (IDDE) program. Although this measure does not directly target the CSS, it can have significant impacts on local water quality that can help to address Total Maximum Daily Loads (TMDLs). Illicit connection control is not particularly effective at achieving any of the primary goals of the LTCP.

4.2.4 Good Housekeeping

4.2.4.1 Street Sweeping/Flushing

Municipal street cleaning enhances the aesthetic appearance of streets by periodically removing the surface accumulation of litter, debris, dust and dirt, which prevents these pollutants from entering storm or combined sewers. Common methods of street cleaning are manual, mechanical and vacuum sweepers, and street flushing. However, the total public area accessible to street sweepers is limited, and generally does not include sidewalks, traffic islands, and congested street parking areas. Although street sweeping/flushing can reduce the concentration of floatables and pollutants in storm runoff that originate from the street, the measure has minimal impact on bacteria or CSO volume reduction.

4.2.4.2 Leaf Collection

Leaf collection is an important part of stormwater management because it not only keeps leaves out of the stormwater system to maintain its maximum flow capacity, but also benefits water quality by reducing nutrients such as phosphorous and nitrogen that can originate from the decomposition of leaves. In most municipalities, this long term stormwater management measure is scheduled based on seasonal patterns, and is an effective tool to maintain capacity in both the separate storm sewer and the CSS.

4.2.4.3 Recycling Programs

Recycling programs provide a means for the public to properly dispose of items that may otherwise end up entering the CSS, such as motor oil, anti-freeze, pesticides, animal waste, fertilizers, chemicals, and litter. These programs are usually effective in reducing floatables and toxins.

4.2.4.4 Storage/Loading/Unloading Areas

Industrial and commercial users would be required to designate and use specific areas for loading and unloading operations. This would concentrate the potential for loading and unloading related waste to a few locations on site, making it easier to manage waste. The effectiveness of this technology is limited to the number of industrial users upstream of CSO regulators. If there are no industrial users in the CSS, then this is technology is not applicable.

4.2.4.5 Industrial Spill Control

Industrial users would be required to utilize spill control technologies like containment berms and absorbent booms to mitigate the risk of contaminants entering the waterway or collection system. Similar to Storage/Loading/Unloading Areas, the effectiveness of this technology is limited to the number of industrial users upstream of CSO regulators.

4.2.5 Green Infrastructure

Green infrastructure is a source control that reduces runoff volumes, peak flows, and/or pollutant loads. GI utilizes the processes of infiltration, evapotranspiration, and capture for re-use to reduce the amount of runoff volume (USEPA, 2014). It is effective at increasing the time of concentration of remaining runoff and reducing pollutant loads through sedimentation and filtration. This technology can be used alone in a scalable manner, or it can be used in conjunction with gray infrastructure to reduce its size and cost.

GI's benefits extend beyond reducing the flow of water into CSSs during wet weather events. Through mimicking a more naturalized system, GI can deliver a broad range of ecosystem services or benefits to people, some of which include:

improved community livability (aesthetics and property values), human health, air quality, water quality, groundwater recharge, wildlife habitats and connectivity, reduced heat island effects, reduced energy use, green jobs, and recreational opportunities (USEPA, 2014). It can also help reduce flooding and is flexible for addressing climate change (droughts or increased precipitation). As described in Greening CSO Plans: Planning and Modeling Green Infrastructure for Combined Sewer Overflow (CSO) Control (USEPA, 2014), the EPA requires that any incorporation of GI into a LTCP include analysis in two areas:

1. Community and political support for GI; and
2. Realistic potential for GI implementation.

PVSC and the Permittees will assess the public support from stakeholders in the community and government for the GI alternatives through the implementation of the LTCP Public Participation Plan. The realistic potential for the implementation will first be screened within this memorandum and refined further in the alternatives evaluation.

There are a wide range of potential GI technologies currently in use throughout the country, and many of these include numerous design variations incorporated into a variety of documents and design manuals. The intent of this section is to summarize important aspects of the relevant practices, rather than to provide a comprehensive catalog or detailed design documents.

In addition, there are watershed-scale GI options that are not appropriate for PVSC Service Area due to highly urbanized nature of the CSS area or improper resources to maintain the technology. These include land conservation efforts and creation, preservation, or restoration of riparian buffers, flood plains, wetlands, open space, and forests. These GI options should be encouraged when land use can easily be converted for this intention with minimal upkeep, but this report will not consider these technologies to reduce runoff volume and bacterial loading. With the above considerations in mind, feasible and appropriate GI technologies were evaluated for implementation in buildings, impervious areas, and pervious areas in PVSC Service Area publically-owned property.

4.2.5.1 Green Roofs

Green roofs have bioretention media that collect runoff to promote evapotranspiration and achieve water quality through soil media filtration. They are typically shallow in depth (4-8”) based on the ability of the building to support the weight of the media, plantings, and captured rainfall. Green roofs may be built in layers on a roof or installed as cells in crates. An example green roof section can be found in Figure 4-1.

Green roofs are recommended for use on buildings with flat roofs (recommended 1-2% slope) that have the structural capacity to support the weight of the media, plantings and water. Structural improvements to an existing building to support

the additional weight associated with a green roof are not typically recommended; therefore this technology is more feasible on new construction. Green roofs can be installed in a section or across an entire roof. An overflow system is typically installed. The vegetation may require irrigation during the first 1-2 years to establish growth. Recommended maintenance for green roofs includes semi-annual maintenance of vegetation.

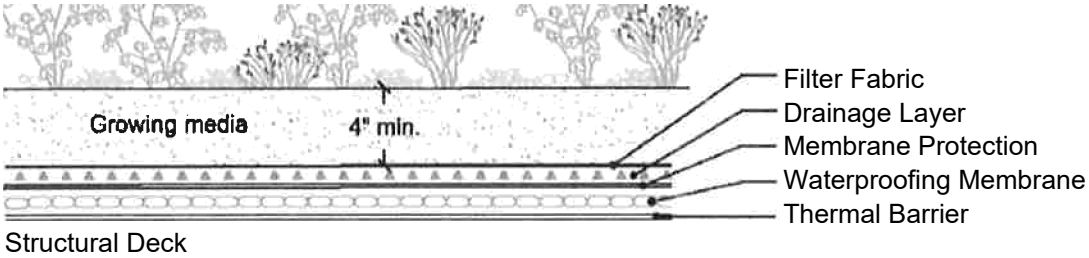

Figure 4-1. Example Green Roof Section

Many rooftop retrofits are required for this GI technology to have measureable impact. Most of the buildings in the CSS are privately owned. Implementing this technology on a scale that would have a measureable impact would require retrofits on private property.

4.2.5.2 Blue Roofs

Blue roofs collect runoff to promote evaporation (they do not have plantings) through detention. They are typically shallow in depth (4-8”) based on the ability of the building to support the weight of the media and captured rainfall. Blue roofs may be built in layers on a roof or installed as cells in crates.

Unlike green roofs, a blue roof may not provide any water quality benefits, unless filters or storage media are used specifically for this purpose. The water detained from blue roofs may be used on-site instead of being released with the appropriate modifications.

Blue roofs are recommended for use on buildings with flat roofs (recommended 1-2% slope) that have the structural capacity to support the weight of the media and water. Structural improvements to an existing building to support the additional weight associated with a blue roof are not typically recommended; therefore this option is more feasible on new construction. Blue roofs can be installed in a section or across an entire roof. An overflow system is typically installed to direct the detained water off of the roof. Recommended maintenance for blue roofs includes semi-annual maintenance for clearing of debris.

Similar to green roofs, blue roofs would require implementation on private property to have a measureable impact.

4.2.5.3 Rainwater Harvesting

Rainwater harvesting is the collection and storage of rainfall from buildings to delay or eliminate runoff. The reduction in runoff volume varies based on the size of the rain barrel or cistern storage unit, and the reuse of the stored rainfall. A few typical reuse options are irrigation and vehicle washing. Indoor reuse options, such as toilet flushing and heating and cooling, may be possible if coordinated with building policies.

Rainwater harvesting is applicable to all types of buildings with gutters and downspouts but may be reserved for buildings where green or blue roofs are not appropriate (roof slopes greater than 2%). Storage units may be sized and installed for each downspout or for the building as a whole. Rain barrels, such as those in Figure 4-2, are typically used for residential installations and larger cisterns are typically used for non-residential applications. They are typically placed at grade but can be buried below grade if a pumping system for water reuse is provided. An overflow system is typically installed. Recommended maintenance for rainwater harvesting includes semi-annual maintenance for clearing of debris in the piping or storage unit.

Similar to green and blue roofs, this technology is limited by the number of available roofs, most of which are private. Private residential uses of cisterns are much less common than on private commercial properties, but are encouraged to help reduce combined sewer overflows.

Figure 4-2. Rain Barrels

4.2.5.4 Permeable Pavements

Permeable pavements promote runoff infiltration and rely on a permeable substrate (engineered soils) to store runoff and remove pollutants. There are different types of permeable pavements, most commonly constructed with asphalt, concrete, or pavers. Permeable asphalt and concrete are similar to traditional mixes except that the amount of fine aggregates is reduced or eliminated. Permeable pavers are individual paver units laid together to create a paved surface. The depth of the permeable substrate, anywhere from 3-10 feet, will have the largest impact on runoff volume reduction. Substrate design may incorporate stormwater retention chambers to increase storage volume. Underdrains may be necessary depending on the local soil types, depth of substrate, and groundwater elevation.

Permeable pavements are recommended for low traffic and low speed traffic areas such as sidewalks, parking lanes, parking lots, driveways, and alleys. Figures 4-3, 4-4, and 4-5 show slightly different pavement includes semi-annual inspection and vacuuming. Preventative maintenance is also necessary to minimize the introduction of soil and other fine particles that could clog the pavement pores.

This GI technology can be very effective when implemented in parking lots, parking lanes, and narrow sidewalks where planter boxes cannot be implemented.

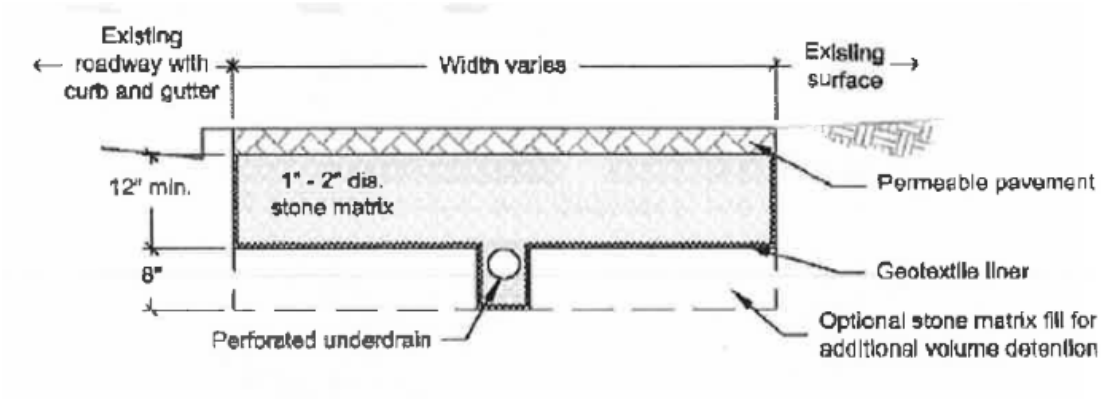

Figure 4-3. Example Permeable Sidewalk Section

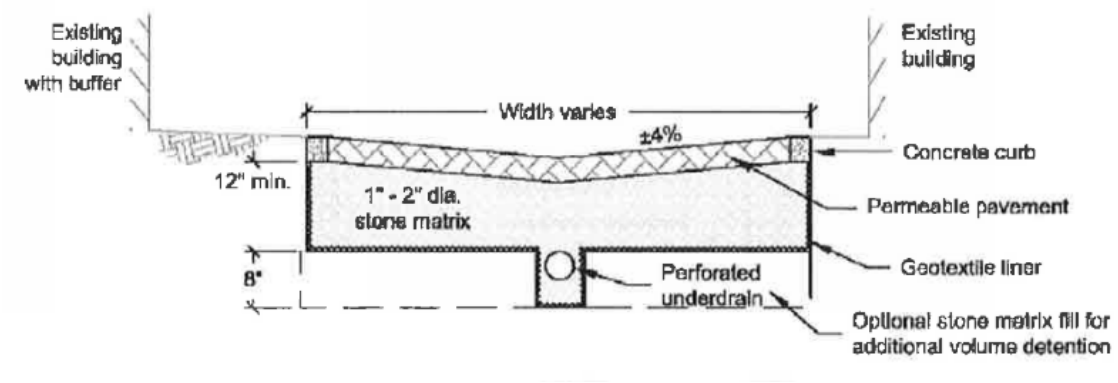

Figure 4-4. Example Permeable Parking Lane Section

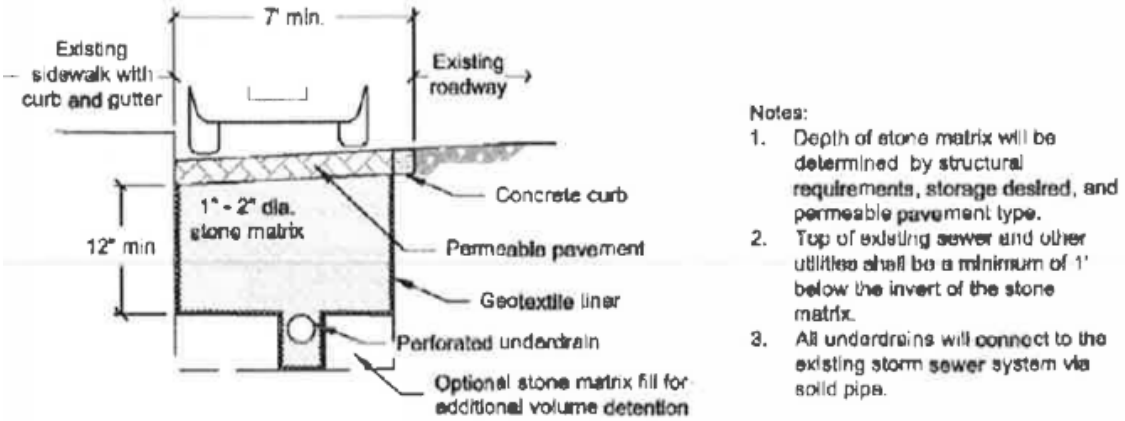

Figure 4-5. Example Permeable Alley Section

4.2.5.5 Planter Boxes

Planter boxes are bioretention cells that collect runoff and promote runoff Infiltration. These walled units are similar to free-form rain gardens as vegetated depressions (12-24”) that rely on ponding and a permeable substrate (engineered sols) to store runoff and remove pollutants. The depth of the permeable substrate, anywhere from 3-10 feet, will have the largest impact on runoff volume reduction. An Example Planter Bumpout Section can be found in Figure 4-6. Substrate design may incorporate stormwater retention chambers.to increase storage volume. Properly designed planter boxes limit ponding to 3-6 hours after a storm. Ponding overflow pipes and/or underdrains may be necessary depending on the local soil types, depth of substrate, and groundwater elevation. The vegetation promotes evapotranspiration to reduce the volume of the stored runoff.

There are two primary sizes of planter boxes for use based on the drainage pattern in developed areas: sidewalk planter boxes and bumpout planter boxes. Sidewalk planter boxes may also be more specifically referred to as a Tree Well Best Management Practice (BMP), a Tree Well with Soil Panels, a Continuous Planting Strip, Mid-Sidewalk BMP, or a Back of Sidewalk BMP. Sidewalk planter boxes are depressed below the elevation of the existing sidewalk. Bumpout planter boxes are larger units that extend from the sidewalk curb into an area of a parking lane. An example of this design can be found in Figure 4-6. Curb cuts into planter boxes allow roadway runoff to enter the cells and overflow to street inlets once the maximum ponding depth has been reached. Planter boxes are recommended for use in regularly spaced intervals in the downstream drainage path in areas of impervious cover.

Recommended maintenance for planter boxes includes semi-annual inspections and improvements to vegetation and mulch, and annual inspection of overflow pipes and underdrains, if applicable. Inspection after a large storm is also recommended. If there is evidence of ponding after 48 hours, mulch replacement or overflow pipe cleaning may be necessary.

Planter boxes are well suited for highly developed areas where space allows. They can be installed block by-block to contain, infiltrate, and evapotranspirate stormwater runoff.

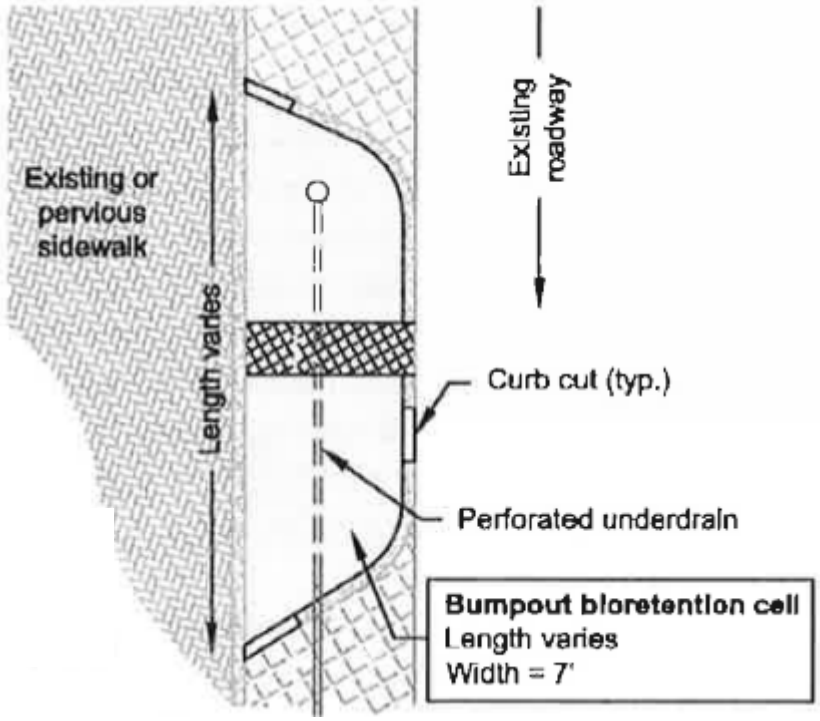

Figure 4-6. Example Planter Bumpout Section

4.2.5.6 Bioswales

Bioswales are vegetated channels that reduce runoff velocity and promote runoff infiltration. These are linear channels with shallow depressions (6-12”) that incorporate vegetation and a permeable substrate (engineered soils). As a channel, runoff not infiltrated does not pond, but flows through the swale and is conveyed elsewhere. The channels, especially those with slopes greater than 6%, may incorporate check dams to assist in reducing runoff velocity and promote infiltration and pollutant removal. A design example for a bioswale is found in Figure 4-7.

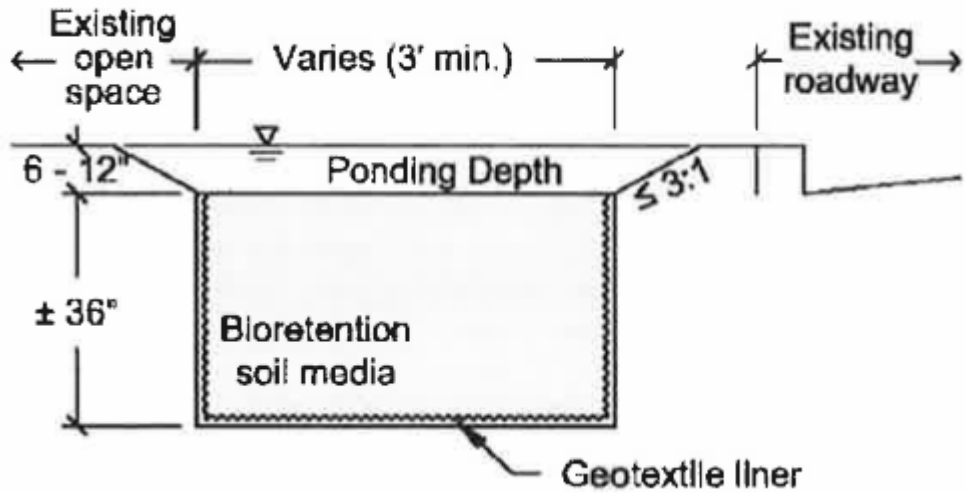

Figure 4-7. Example Bioswale Detail

Bioswales are recommended for use in parks and areas of natural cover since they primarily reduce runoff velocity and have a low volume reduction per square foot. Due to their linear nature, bioswales may also be effective in the buffer between open space areas and impervious areas with high volumes of runoff such as roads and parking lots. Recommended maintenance for bioswales includes semi-annual inspections and improvements to vegetation and mulch.

This technology incorporates both stormwater treatment and stormwater conveyance. While not as flexible as planter boxes, there may be locations in within the community where a bioswale could be effective.

4.2.5.7 Free-Form Rain Gardens

Rain gardens are bioretention basins that collect runoff and promote runoff infiltration. These are vegetated depressions (12-24”) that rely on ponding and a permeable substrate (engineered soils) to store runoff and remove pollutants. The size and shape of rain gardens can be tailored to site- specific needs, but the depth of the permeable substrate (anywhere from 3-10 feet) will have the largest impact on runoff volume reduction. Substrate design may incorporate

stormwater retention chambers to increase storage volume. Properly designed rain gardens limit ponding to 3- 6 hours after a storm. Ponding overflow pipes and/or underdrains may be necessary depending on the local soil types, depth of substrate, and groundwater elevation. The vegetation promotes evapotranspiration to reduce the volume of the stored runoff, and infiltration helps improve water quality. An example of a rain garden is found in Figure 4-8.

Rain gardens are recommended for use in low points in parks and areas of natural cover so they can blend in seamlessly with a grassed buffer and enhance the vegetation without appearing to be a stormwater control mechanism. Locations near the transition from pervious to impervious cover can provided runoff reduction for nearby impervious areas.

Recommended maintenance for rain gardens includes semi-annual inspections and improvements to vegetation and mulch and annual inspection of overflow pipes and underdrains, if applicable.

Annual inspection after a large storm is also recommended. If evidence of ponding exists after 48 hours, mulch and/or soil replacement or overflow pipe cleaning may be necessary.

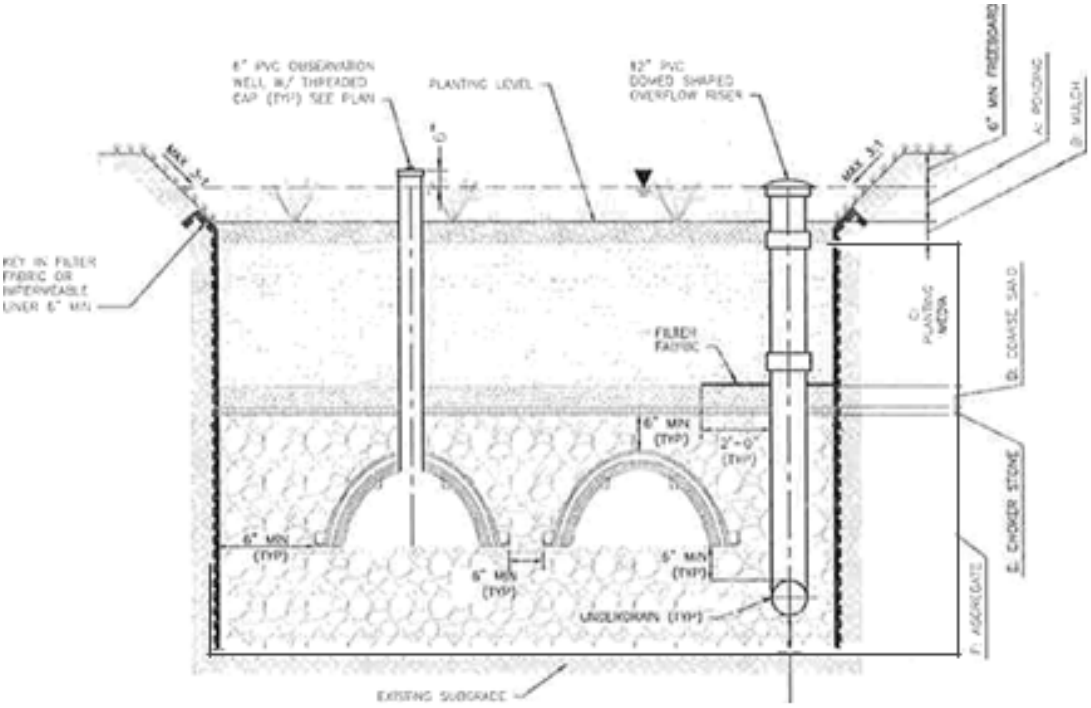

Figure 4-8. Example Rain Garden Section

Rain gardens are very effective at capturing and treating stormwater and have versatile footprints that make them advantageous for use in highly developed urban environments.

4.3 Infiltration and Inflow Control

4.3.1 Infiltration/Inflow (I/I) Reduction

Excessive infiltration and inflow can consume the hydraulic capacity of a collection system and increase overall operations and maintenance costs. Inflow comes from sources such as roof drains, manhole covers, cross connections from storm sewers, catch basins, and surface runoff. Within a CSS, surface drainage is the primary source of inflow. Infiltration comes from groundwater that seeps in through leaking pipe joints, cracked pipes, manholes, and other similar sources. The flow from infiltration tends to be constant, but at a lower volume than that of inflow.

Identifying I/I sources is labor intensive and requires specialized equipment. Significant I/I reductions can also be difficult and expensive to achieve. However, the benefit of a good I/I control program is that it can save money by extending the life of the system, reducing the need for expansion, and lowering treatment costs. I/I reduction for combined sewers provides limited gains, since water tends to find another way into the system. However, I/I reductions in sanitary sewers can have significant impacts on increasing the available capacity in the downstream CSS.

4.3.2 Advanced System Inspection & Maintenance

System inspection and maintenance programs can provide valuable knowledge about the condition of the CSS infrastructure, which is beneficial for planning, inspection, and maintenance activities. This can help ensure design flow capacity is consistently available to prevent CSO events. This technology offers relatively minor advances towards meeting the primary and secondary goals of the LTCP.

4.3.3 Combined Sewer Flushing

This type of operation and maintenance (O&M) practice re-suspends solids that have settled in the CSS and flushes them downstream. This practice consists of introducing a controlled volume of water over a short duration at key points in the collection system using external water from a tank truck, pressurized feed, or by detaining the CSS flow for a period, and then releasing it. Overall, this practice helps reduce the amount of settled solids that are resuspended and discharged during significant wet weather events. This measure is most effective when applied to flat collection systems since solids are more likely to become deposited on flat grades.

4.3.4 Catch Basin Cleaning

Catch basin cleaning reduces the transport of solids and floatables to the CSS by regularly removing accumulated catch basin deposits. Methods to clean catch

basins include manual, bucket, and vacuum removal. Catch basin cleaning can be effective in reducing floatables in combined sewer; however, it is not effective at bacteria reduction or volume reduction, nor is it particularly effective at BOD reduction.

4.4 Sewer System Optimization

Sewer system optimization involves collection system controls and modifications that affect CSO flows and loads once the runoff has entered the collection system. Options for system optimization include measures that maximize the volume of flow stored in the collection system or maximize the capacity of the system to convey flow to the treatment plant. Sewer system optimization techniques have no impact on water quality, but do have the potential to reduce the volume of CSO events.

4.4.1 Increased Storage Capacity in the Collection System

Options for increased storage capacity rely on maximizing the volume of flow stored in the collection system or increasing the conveyance capacity of the system. Maximizing the use of the existing system involves ongoing maintenance and inspection of the collection system, and can include minor modifications/repairs to existing structures to increase the volume of flow retained in the system. Increasing conveyance capacity is typically achieved by providing additional conveyance pipes or upsizing the existing conveyance system to handle a greater capacity.

4.4.1.1 Additional Conveyance

Conveyance is a technology that transports the combined sewage out of a particular area to a location where the flow can be stored, treated, or discharged where direct public contact with the water is less likely. Conveyance is accomplished by providing additional conveyance pipes or upsizing the existing conveyance pipe to a greater capacity. This practice can effectively reduce overflow volume and frequency in the affected areas. Large conveyance projects can be expensive and may require a lengthy permitting process.

4.4.1.2 Regulator Modifications

A CSO regulator can be uniquely configured to control combined sewer overflow frequency and volume. The existing overflow control structures may be modified based on site-specific conditions. For example, regulator modifications may include increasing the overflow weir height and length or raising the overflow pipe elevation. This technology is especially effective for CSO outfalls with high overflow frequency and low overflow volume, because the additional volume held back in the system is small and less likely to have negative impacts on upstream conditions.

4.4.1.3 Outfall Consolidation/Relocation

Consolidation of one or multiple outfalls can help eliminate CSO discharges in sensitive areas. Outfall consolidation may require modification or relocation of an outfall, the installation of additional conveyance to accommodate new flow configurations, and may also require additional permitting with government agencies. This practice typically lowers O&M requirements for the CSS by limiting the number of outfall structures that need to be monitored. Outfall consolidation works best in areas where outfalls are located in close proximity to each other and require limited additional conveyance. Similar to regulator modifications, outfall consolidation is especially effective at reducing high frequency, low volume CSOs. This practice typically doesn't add a significant amount of extra capacity to the CSS (depending on the amount of conveyance pipe associated with the consolidation project), so its impact on infrequent, large volume CSO events can be limited. The Hydrologic and Hydraulic (H&H) Model can determine the level of impact that outfall consolidation will have in terms of reducing the number of CSO events.

4.4.1.4 Real Time Control

Real Time Control (RTC) is a highly automated system in which sewer level and flow data are measured at key points in the sewer system and used to operate systems controls to maximize the storage capacity of the CSS and limit overflows. The collected data is typically transferred to a control device where program logic is used to operate gates, pump stations, inflatable dams and other control components. Local dynamic controls are used to control regulators to prevent flooding and system wide dynamic controls are used to implement control objectives, such as maximizing flow to the treatment plant or transferring flows from one portion of the CSS to another to fully utilize the system. Predicative control, which incorporates use of weather forecast data, is an optional feature, but it should be noted that it is complex and requires sophisticated operational capabilities. Additionally, it is important to note that RTC involves the installation of numerous mechanical control, which require upkeep and maintenance, and can only reduce CSO volumes where in-system storage capacity is available.

4.5 Storage

The objective of storage is to reduce overflows by capturing and storing wet weather flows, greater than CSS conveyance/treatment plant capacity, for controlled release back into the system once treatment and conveyance capacity have been restored. A storage facility can attenuate peak flows in the CSS and provide a relatively constant flow into the treatment plant after peak events. Storage technologies do not prevent water from entering the CSS or treat bacterial loads in CSO discharge, but are very effective at reducing or eliminating CSO events. Storage technologies typically have fairly high construction and O&M costs compared to other CSO control technologies, but are a very reliable means of achieving CSO control goals.

4.5.1 Linear Storage

Linear storage is provided by underground storage facilities that are sized to detain peak flows during wet weather events for controlled release back into the system after the event. In-line linear storage (storage in series with the CSS) can be provided by over-sizing the existing interceptors for conveyance, as described in the previous section, whereas off-line linear storage (storage parallel to the CSS) can be provided by installing new facilities such as tunnels and pipelines.

4.5.1.1 Pipelines

Large diameter parallel pipelines or conduits can provide significant storage in addition to the ability to convey flow. Pipelines are typically constructed between an overflow point and a pump station or treatment facility. The pipelines include discharge controls to allow flow to be stored within the pipeline during wet weather events, and slowly released by gravity following the event. The pipelines' conveyance to the desired endpoint depends on the additional capacity necessary to handle the increased flow and is developed concurrently with the pipeline. A force main pipeline constructed from a pump station relies heavily on the increased flow capacity as the storage benefits are negligible. Pipelines have the advantage of requiring less area for construction compared to point storage. If trenchless technologies can be utilized, such as horizontal directional drilling (HDD), land requirements can be reduced even further.

One disadvantage of pipelines is that a larger volume is typically required to accommodate combined sewer storage needs. The installation of large diameter pipelines is typically less cost effective than tunneling, and the installation of smaller diameter pipes typically requires a significant length in order to provide adequate storage. Additionally, the installation of pipelines is very disruptive, typically requiring open trenches and the temporary closure of public streets.

4.5.1.2 Tunnels

Tunnels provide more storage volume than pipelines, while maintaining the ability to convey flow. Tunnel excavation is accomplished completely underground, and therefore results in minimal surface disruption and requires little right-of-way, outside of drop shafts and conveyance piping to the drop shafts. Overall costs for tunnels can be high, but their cost per million gallons of storage is fairly reasonable compared to other storage technologies, depending on local geology. Tunnels are typically used in congested urban areas where available land is scarce and connections to most, if not all, of the CSO regulators can be made.

4.5.2 Point Storage

Point storage can be provided by above-ground or underground storage facilities such as tanks and equalization basins. These off-line facilities are placed at

specific points in the system to detain peak flows for controlled return back to the system, reducing CSO discharge volume and bacterial loading.

4.5.2.1 Tanks

This technology reduces overflow quantity and frequency by storing all or a portion of diverted wet weather combined flows in off-line storage tanks. Stored flows are returned to the interceptor for conveyance to the treatment plant once system capacity becomes available. Storage tanks are generally fed by gravity and the stored flow is typically pumped back to the interceptor the storm. The benefit of off-line storage tanks is that they are well suited for early action projects at critical CSO outfalls. Storage tanks capture the most concentrated first flush portion wet weather peak flow and help to reduce the downstream capacity needs for conveyance and treatment.

A disadvantage of off-line storage tanks is that they typically require large land area for installation, which may not be available in congested urban areas. Off-line storage tanks typically have higher costs per volume captured compared to other technologies. Additionally, if the existing sewers are deep, then the storage tank must also be deep, which results in additional construction costs. Operation and maintenance costs can also be high, especially if the application includes provisions for partial treatment and discharge, rather than simple storage and bleed-back to the sewer. Depending on the application, odor problems may also be an Issue. However, storage tanks can be a very effective means of CSO control.

4.5.2.2 Industrial Discharge Detention

This technology would require industrial users to build and maintain storage basins to hold industrial discharge during wet weather events and subsequently release it back to the CSS. This would limit the peak wet weather flow to the WRTP. The effectiveness of this technology is limited to the number of industrial users upstream of CSO regulators. If there are no industrial users in the CSS, then this is technology is not applicable.

4.6 STP Expansion or Storage at the Plant

4.6.1 Additional Treatment Capacity

CSOs can potentially be reduced by increasing the treatment capacity of plant. Other technologies can make use of this increased treatment capacity by providing more flow to the plant instead of CSO outfalls.

4.6.2 Wet Weather Blending

Blending is the practice of allowing portions of the wet weather peak flow to bypass certain treatment facilities at the plant. In blending, wet weather flows are

typically routed through primary treatment, allowed to bypass secondary and tertiary treatment, and then recombined with effluent from all processes prior to disinfection and discharge to the environment. This practice may require increasing the capacity of primary treatment and disinfection facilities, but doesn't require the upsizing of secondary treatment facilities, which can be the more costly components. Other technologies can make use of the increased wet weather peak flow capacity by providing more flow to the plant instead of CSO outfalls.

4.7 Sewer Separation

4.7.1 Roof Leader Disconnection

Roof leaders may directly be connected to the CSS. Roof leaders can be disconnected in order to divert stormwater elsewhere and/or to delay its entry into the CSS. Depending on the neighborhood, roof leaders may be run to dry well, vegetation bed, lawn, storm sewer, or street. This technology typically has limited benefits in dense urban areas due to the lack of pervious areas available to divert flow for infiltration. Unfortunately, the most feasible rain leader disconnection scheme in these areas is usually diversion to the street. In this case, disconnection can lead to nuisance street flooding and is only able to briefly delay the water from entering the CSS through catch basins. Roof leader disconnection is typically much more effective in areas with separate sewers where the roof leader was previously connected to a sanitary sewer, since the diverted rainwater does not have a direct path back into the system. Roof leader disconnection can be effective for both sanitary and storm sewers; however, the effect of this measure is highly contingent upon the extent of roof leaders in the system, site specific conditions, and the ability to find an adequate location to divert stormwater flow from the roof leader.

4.7.2 Sump Pump Disconnection

Buildings with basements below the ground water table sometimes are kept dry by using dewatering pumps. In many cases, these pumps discharge to the CSS or sanitary sewers. Sump pump disconnection diverts this pumped groundwater flow to a location other than these sewers. Sump pump disconnection programs are typically more effective in separate sewer areas and are subject to the same limitations as roof leader disconnection programs (extent, site conditions, diversion options, etc.). There are many limitations to the effectiveness of this approach in terms of the resources, impact on the public and difficulties implementing.

4.7.3 Combined Sewer Separation

Sewer separation is the conversion of a CSS into a system of separate storm sewers and sanitary sewers. This can be accomplished by installing a new

sanitary sewer and using the existing combined sewer as a storm sewer or vice versa. This practice can be very expensive, disruptive to the public, and difficult to implement, especially in downtown areas or other densely developed urban environments. It typically requires closure of public streets for construction while the new pipes are installed and the sewer is separated.

4.8 Treatment of CSO Discharge

Treatment technologies are intended to reduce the pollutant loads to receiving waters by treating wet weather flows prior to discharging to the environment. Specific technologies can address different pollutant constituents, such as settleable solids, floatables, or bacteria.

4.8.1 Treatment — CSO Facility

4.8.1.1 Vortex Separators

Vortex separation is a process that removes floatables and settleable solids from a wastewater stream by directing influent flow tangentially into a cylindrical tank, thereby creating a vortex. The vortex action causes settleable solids to move toward the center of the tank where they are concentrated with a fraction of the influent flow and directed to the underflow at the bottom of the tank. The underflow is then conveyed downstream to the treatment plant. The remaining influent flow travels under a baffle plate, which traps any floatables, and then over a circular baffle located in the center of the tank. It is then discharged to receiving waters or conveyed to storage or treatment devices for further processing. This technology does not address CSO volume or bacteria reduction, and would only help meet water quality and CSO control goals only if used in combination with other technologies.

4.8.1.2 Screens and Trash Racks

Screens and trash racks consist of a series of vertical and horizontal bars or wires that trap floatables while allowing water to pass through the openings between the bars or wires. They can be installed at select points within a CSS to capture floatables and prevent their discharge during CSO events. Due to limited hydraulic capacity, screens are most suitable for small outfalls. Trash racks or static screens can be located on top of an overflow weir or near the outfall. These devices are inexpensive but usually incur high maintenance costs due to their tendency to become clogged. Frequent cleaning (after every storm) is usually required to prevent clogging, which can cause serious flooding and sewer backups.

Mechanical screens can remove floatables and some solids without frequent manual cleaning. This can be a significant advantage when compared to the maintenance requirements and the potential for flooding caused by a clogged static screen. However, most mechanical screens (climber screens, cog screens,

or rake screens) require structural modifications to the outfall chamber to house and protect the screens. If weir-mounted mechanical screens are used instead, they require much less headroom and can be retrofitted into an existing overflow chamber with little to no structural modifications.

As this technology does not address CSO volume or bacteria reduction, it would do little to meet Water quality and CSO control goals.

4.8.1.3 Netting

Netting systems involve mesh nets that are attached to a CSO outfall to capture floatable material as the CSO discharges into the receiving water. The nets are nylon mesh bags that can be concealed inside the CSO outfall until an overflow occurs. The advantage of this technology is that it captures floatables inexpensively, and can provide a base level of control at some CSO sites.

However the operation and maintenance requirements are high and it has some negative aesthetic impacts associated with the visibility of collected trash in the waterbody. This technology is strictly for floatables control and will not address water quality and CSO control goals alone.

4.8.1.4 Containment Booms

A containment boom is a temporary floating barrier used to contain floatables entering into the waterway from a CSO outfall. Containment booms are used to reduce the spread of floatables and reduce the level of effort for post-storm cleanup. These devices are very simple to install, but can be difficult to maintain. Also, there are some negative aesthetic impacts associated with visibility of collected trash in a waterbody. This technology is strictly for floatables control and will not address water quality and CSO control goals alone.

4.8.1.5 Baffles

Baffles are simple floatables control devices that are typically installed at flow regulators within the CSS. They consist of vertical steel plates or concrete beams that extend from the top of the sewer to just below the top of the regulating weir. During an overflow event, floatables are retained by the baffles while water passes under the baffles, over the regulator, and into the receiving water body. When the flow recedes below the bottom of the baffle, floatable material is carried downstream to the treatment plant. Baffles are easy to install and require little maintenance, but do require proper hydraulic configuration. This technology is strictly for floatables control and will not address water quality and CSO control goals alone.

4.8.1.6 Disinfection and Satellite Treatment

This technology consists of disinfecting and treating sewer overflows at a local facility near the CSO outfall. Disinfection is very effective at reducing bacteria through inactivation, but provides only limited opportunities for volume reduction. Disinfection alone cannot provide reductions in TSS, floatables, and nutrient loads unless other processes (e.g., screening, high-rate clarification, etc.) are provided upstream of the disinfection facility. The combination of these other processes with disinfection can provide a satellite location that helps reduce pollutants of concern.

Disinfection of wet weather flow is more challenging to design and control than traditional disinfection at a treatment plant, because of the complex characteristics of the flow. Intermittent occurrences and highly variable flowrates make it more challenging to regulate the addition of disinfectant. One way to address the variable flow issue is to provide flow retention facilities that provide for disinfectant contact time and capture through storage of the first flush of TSS, floatables and nutrients.

Wet weather flows can vary widely in temperature, suspended solids concentrations, and bacterial composition. Therefore, pilot studies are usually needed to characterize the range of conditions that exist for a particular area and the design criteria that need to be considered. Experience has shown that the long contact time required for conventional wastewater treatment is not appropriate for the treatment of wet weather flows. Disinfection can be achieved by providing an increased disinfection dosage and intense mixing to ensure disinfectant contact with the maximum number of microorganisms.

Various disinfection technologies are available, both with and without chlorine compounds. In addition to disinfection effectiveness, many factors should be considered when selecting a disinfectant, including potential toxic effects to the environment, regulations for residuals, safety precautions, and ease of operation and maintenance. Ultraviolet (UV) light and peracetic acid (PAA) are two alternatives to chlorine compounds for wet weather disinfection.

- Ultraviolet Light - The main advantages of UV include its ability to quickly respond to flow variation and the absence of a disinfectant residual, among others. The size of the UV system mainly depends on the UV transmittance (i.e., the ability of wastewater to transmit UV light) and TSS concentrations in the wastewater. One of the challenges for UV disinfection is determining how to manage the disinfection of effluent during a power outage. In addition, UV typically has higher capital cost compared to chlorine disinfection systems.
- Peracetic Acid - The main advantage of PAA over sodium hypochlorite is its long “shelf life” without product deterioration. Due to the intermittent nature of CSO flows, stored sodium hypochlorite may degrade over time

if not used. However, PAA systems generally have higher operating costs than chlorine systems.

4.8.1.7 High Rate Physical/Chemical Treatment (ActiFlo)

High rate physical/chemical processes, such as Veolia's Actiflo or Infilco-Degremont's DENSADEG, are treatment facilities that require a much smaller footprint than conventional processes. These two competing products have very similar applications, but have processes that differ from each other considerably. For brevity, only one of these processes (Actiflo) is described in detail below and shown in Figure 4-9.

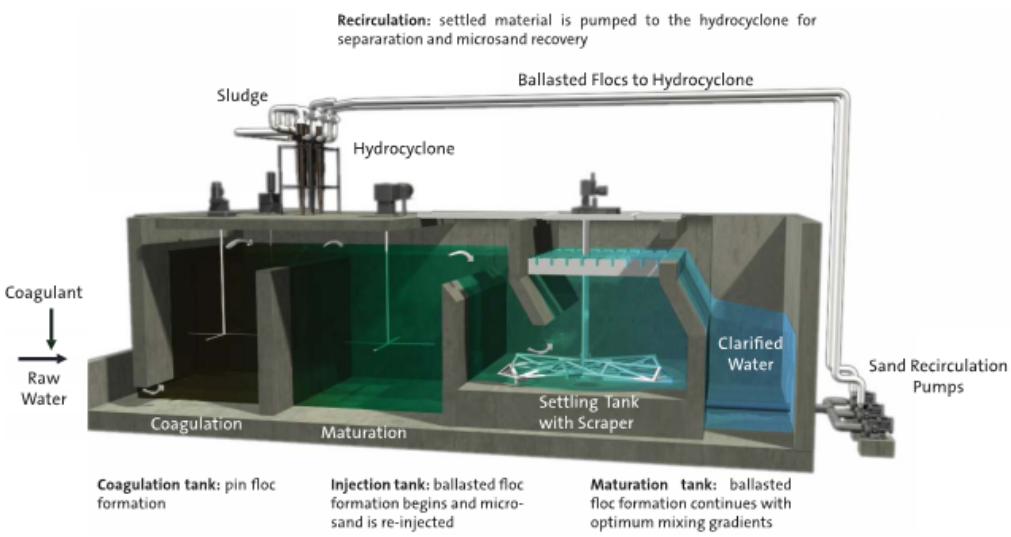

Figure 4-9. Actiflo Process

Fundamentally, the Actiflo process is very similar to conventional coagulation, flocculation, and sedimentation water treatment technology. Both processes use coagulant for suspended solid destabilization and flocculent aid (polymer) for the aggregation of suspended materials. The primary difference between Actiflo and conventional processes is the addition of microsand for the formation of high-density flocs that have a higher-density nucleus and thus settle more rapidly.

Clarified water exits the process by flowing over a weir in the settling tank. I he sand and sludge mixture that remains is collected at the bottom of the settling tank and pumped to a hydrocyclone which separates the sludge from the microsand. Sludge is discharged out of the top of the hydrocyclone while the sand is recycled back into the Actiflo process for further use. This process requires upstream screening to ensure that particles larger than 3 to 6 mm do not clog the hydrocyclone.

Actiflo performance varies, but in general removal rates of 80 - 95% for TSS and 30 - 60% for BOD are typical. Phosphorous and nitrogen are also removable with this process, although the removal efficiencies are dependent on the solubility of these compounds present in the wastewater.

Phosphorous removal is typically between 60-90%, and nitrogen removal is typically between 15-35%. Removal efficiencies are also dependent on start-up time. Typically the Actiflo process takes about 15 minutes before optimum removal rates are achieved.

The LTCP primary goals are bacteria reduction and CSO volume reduction. While high rate physical/chemical treatment reduces bacteria somewhat, its principal purpose is TSS reduction Disinfection would be required downstream for bacteria inactivation. Additionally, while disinfection can be enhanced with upstream treatment, it can be adequately accomplished without high rate physical/chemical treatment. As such, these processes do not add significant value compared to disinfection alone. Furthermore, while technologies such as Actiflo or DENSADEG reduce the footprint of conventional treatment, they still require a significant amount of available space for implementation.

4.8.1.8 High Rate Physical Treatment (FlexFilter)

The FlexFilter by Schreiber or the WesTech WWETCO FlexFilter shown in Figure 4-10 is an innovative filtration technology that used a compressible filter media that allows for a much smaller footprint than conventional filtration (footprint reductions of nearly 90%). Both technologies use a synthetic fiber media, as opposed to granular media such as sand, which can handle increased flux rates (up to 30-40 gpm/sf). Additionally, the process uses compressed air scour with influent flow for filter backwashing which eliminates the need for storage tanks. The filter removes up to 80% of influent particles down to 4 microns in diameter. Overall, this is a relatively low maintenance process, which requires periodic lubrication and detergent addition for media washing.

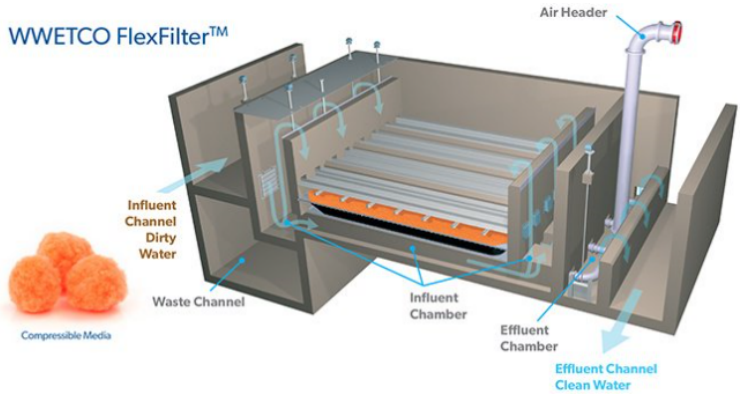

Figure 4-10. WWETCO FlexFilter

This technology is designed for TSS reduction and does not address the primary goals of the LTCP (bacteria reduction and overflow volume reduction). Since downstream disinfection would be required for bacteria inactivation, this technology provides little benefit compared to disinfection alone. Additionally, although this technology decreases the footprint of conventional filtration, it still requires a substantial footprint for implementation.

4.9 Screening of Control Technologies

The tables below summarize the preceding controls technologies and identify what technologies are being considered in Fort Lee. Those technologies that are not being considered in this report may still be considered in local ordinance's and Best Practices.

Table 4-1. Source Control Technologies

Source Control Technologies							
Technology Group	Practice	Primary Goals		Implementation & Operation Factors	Consider Combining w/ Other Technologies	Being Implemented	Evaluated in this Report
		Bacteria Reduction	Volume Reduction				
Stormwater Management	Street/Parking Lot Storage (Catch Basin Control)	Low	Low	Flow restrictions to the CSS can cause flooding in lots, yards and buildings; potential for freezing in lots; low operational cost. Effective at reducing peak flows during wet weather events but can cause dangerous conditions for the public if pedestrian areas freeze during flooding.	No	No	No
	Catch Basin Modification (for Floatables Control)	Low	None	Requires periodic catch basin cleaning; requires suitable catch basin configuration; potential for street flooding and increased maintenance efforts. Reduces debris and floatables that can cause operational problems with the mechanical regulators.	No	Yes	No
	Catch Basin Modification (Leaching)	Low	Low	Can be installed in new developments or used as replacements for existing catch basins. Require similar maintenance as traditional catch basins. Leaching catch basins have minor effects on the primary CSO control goals.	No	No	No
Public Education and Outreach	Water Conservation	None	Low	Water purveyor is responsible for the water system and all related programs in the respective City. However, water conservation is a common topic for public education programs. Water conservation can reduce CSO discharge volume, but would have little impact on peak flows.	Yes	Yes	No
	Catch Basin Stenciling	None	None	Inexpensive; easy to implement; public education. Is only as effective as the public's acceptance and understanding of the message. Public outreach programs would have a more effective result.	Yes	No	No
	Community Cleanup Programs	None	None	Inexpensive; sense of community ownership; educational BMP; aesthetic enhancement. Community cleanups are inexpensive and build ownership in the city.	Yes	Yes	No
	Public Outreach Programs	Low	None	Public education program is ongoing. Permittee should continue its public education program as control measures demonstrate implementation of the NMC.	Yes	Yes	No
	FOG Program	Low	None	Requires communication with business owners; Permitee may not have enforcement authority. Reduces buildup and maintains flow capacity. Only as effective as business owner cooperation.	Yes	No	No
	Garbage Disposal Restriction	Low	None	Permitee may not be responsible for Garbage Disposal. This requires an increased allocation of resources for enforcement while providing very little reduction to wet weather CSO events.	Yes	No	No
	Pet Waste Management	Medium	None	Low cost of implementation and little to no maintenance. This is a low cost technology that can significantly reduce bacteria loading in wet weather CSO's.	Yes	No	No

Table 4-1. Source Control Technologies

Source Control Technologies							
Technology Group	Practice	Primary Goals		Implementation & Operation Factors	Consider Combining w/ Other Technologies	Being Implemented	Evaluated in this Report
		Bacteria Reduction	Volume Reduction				
	Lawn and Garden Maintenance	Low	Low	Requires communication with business and homeowners. Guidelines are already established per USEPA. Educating the public on proper lawn and garden treatment protocols developed by USEPA will reduce waterway contamination. Since this information is already available to the public it is unlikely to have a significant effect on improving water quality.	Yes	No	No
	Hazardous Waste Collection	Low	None	The N.J.A.C prohibits the discharge of hazardous waste to the collection system.	Yes	Yes	No
Ordinance Enforcement	Construction Site Erosion & Sediment Control	None	None	In building code; reduces sediment and silt loads to waterways; reduces clogging of catch basins; little O&M required; contractor or owner pays for erosion control. A Soil Erosion & Sediment Control Plan Application or 14-day notification (if Permitee covered under permit-by-rule) will be required by NJDEP per the N.J.A.C.	Yes	Yes	No
	Illegal Dumping Control	Low	None	Enforcement of current law requires large number of code enforcement personnel; recycling sites maintained. Local ordinances already in place can be used as needed to address illegal dumping complaints.	Yes	Yes	No
	Pet Waste Control	Medium	None	Requires resources to enforce pet waste ordinances. Public education and outreach is a more efficient use of resources, but this may also provide an alternative to reducing bacterial loads.	Yes	No	No
	Litter Control	None	None	Aesthetic enhancement; labor intensive; City function. Litter control provides an aesthetic and water quality enhancement. It will require city resources to enforce. Public education and outreach is a more efficient use of resources.	Yes	Yes	No
	Illicit Connection Control	Low	Low	Site specific; more applicable to separate sanitary system; new storm sewers may be required; interaction with homeowners required. The primary goal of the LTCP is to meet the NJPDES Permit requirements relative to POCs. Illicit connection control is not particularly effective at any of these goals and is not recommended for further evaluation unless separate sewers are in place.	Yes	Yes	No
Good Housekeeping	Street Sweeping/Flushing	Low	None	Labor intensive; specialized equipment; doesn't address flow or bacteria; City function. Street sweeping and flushing primarily addresses floatables entering the CSS while offering an aesthetic improvement.	Yes	Yes	No
	Leaf Collection	Low	None	Requires additional seasonal labor. Leaf collection maximizes flow capacity and removes nutrients from the collection system.	Yes	Yes	No
	Recycling Programs	None	None	Most Cities have an ongoing recycling program.	Yes	Yes	No

Table 4-1. Source Control Technologies

Source Control Technologies							
Technology Group	Practice	Primary Goals		Implementation & Operation Factors	Consider Combining w/ Other Technologies	Being Implemented	Evaluated in this Report
		Bacteria Reduction	Volume Reduction				
	Storage/Loading/Unloading Areas	None	None	Requires industrial & commercial facilities designate and use specific areas for loading/unloading operations. There may be few major commercial or industrial users upstream of CSO regulators.	Yes	No	No
	Industrial Spill Control	Low	None	The POTW has established a pretreatment program for industrial users subject to the Federal Categorical Pretreatment Standards 40 CFR 403.1.	Yes	No	No
Green Infrastructure Buildings	Green Roofs	None	Medium	Adds modest cost to new construction; not applicable to all retrofits; low operational resource demand; will require the Permitee or private owners to implement; requires regular cleaning of gutters & pipes; upkeep of roof vegetation. Portions of Cities have densely populated areas, but this technology is limited to rooftops. Can be difficult to require on private properties.	Yes	No	Yes
	Blue Roofs	None	Medium	Adds modest cost to new construction; not applicable to all retrofits; low operational resource demand; will require the Permitees or private owners to implement; requires regular cleaning of gutters & pipes; upkeep of roof debris. Portions of the Cities have densely populated areas, but this technology is limited to rooftops. Can be difficult to require on private properties.	Yes	No	Yes
	Rainwater Harvesting	None	Medium	Simple to install and operate; low operational resource demand; will require the Permitees or private owners to implement; requires regular cleaning of gutters & pipes. Portions of the Cities have densely populated areas, but this technology is limited to capturing rooftop drainage. Capture is limited to available storage, which can vary on rainwater use. Can be difficult to require on private properties.	Yes	No	Yes
Green Infrastructure Impervious Areas	Permeable Pavements	Low	Medium	Not durable and clogs in winter; oil and grease will clog; significant O&M requirements with vacuuming and replacing deteriorated surfaces; can be very effective in parking lots, lanes and sidewalks. Maintenance requirements could be reduced if located in low-traffic areas, and can utilize underground infiltration beds or detention tanks to increase storage.	Yes	No	Yes
	Planter Boxes	Low	Medium	Site specific; good BMP; minimal vegetation & mulch O&M requirements with regular overflow and underdrain cleaning; effective at containing, infiltrating and evapotranspirating runoff in developed areas. Flexible and can be implemented even on a small-scale to any high-priority drainage areas. Underground infiltration beds or detention tanks can be utilized to increase storage.	Yes	No	Yes
Green Infrastructure Pervious Areas	Bioswales	Low	Low	Site specific; good BMP; minimal vegetation & mulch O&M requirements; not as flexible or infiltrate as much stormwater as planter boxes. Technology requires open space and is primarily a surface conveyance technology with additional storage & infiltration benefits. Can be modified with check dams to slow water flow. Limited open space in most Cities means land can be utilized in more effective ways with the existing infrastructure.	Yes	No	Yes

Table 4-1. Source Control Technologies

Source Control Technologies							
Technology Group	Practice	Primary Goals		Implementation & Operation Factors	Consider Combining w/ Other Technologies	Being Implemented	Evaluated in this Report
		Bacteria Reduction	Volume Reduction				
	Free-Form Rain Gardens	Low	Medium	Site specific; good BMP; minimal vegetation & mulch O&M requirements with regular overflow and underdrain cleaning; effective at containing, infiltrating and evapotranspirating diverted runoff. Rain Gardens are flexible and can be modified to fit into the previous areas. Underground infiltration beds or detention tanks can be utilized to increase storage.	Yes	No	Yes

Table 4-2. Collection System Technologies

Collection System Technologies							
Technology Group	Practice	Primary Goals		Implementation & Operation Factors	Consider Combining w/ Other Technologies	Being Implemented	Evaluated in this Report
		Bacteria Reduction	Volume Reduction				
Operation and Maintenance	I/I Reduction	Low	Medium	Requires labor intensive work; changes to the conveyance system require temporary pumping measures; repairs on private property required by homeowners. Reduces the volume of flow and frequency; Provides additional capacity for future growth; House laterals account for 1/2 the sewer system length and significant sources of I/I in the sanitary sewer.	Yes	Yes	Yes
	Advanced System Inspection & Maintenance	Low	Low	Requires additional resources towards regular inspection and maintenance work. Inspection and maintenance programs can provide detailed information about the condition and future performance of infrastructure. Offers relatively small advances towards goals of the LTCP.	Yes	Yes	Yes
	Combined Sewer Flushing	Low	Low	Requires inspection after every flush; no changes to the existing conveyance system needed; requires flushing water source. Ongoing: CSO Operational Plan; maximizes existing collection system; reduces first flush effect.	Yes	Yes	Yes
	Catch Basin Cleaning	Low	None	Labor intensive; requires specialized equipment. Catch Basin Cleaning reduces litter and floatables but will have no effect on flow and little effect on bacteria and BOD levels.	Yes	Yes	Yes
Combined Sewer Separation	Roof Leader Disconnection	Low	Low	Site specific; Includes area drains and roof leaders; new storm sewers may be required; requires home and business owner participation. The Cities are densely populated and disconnected roof leaders have limited options for discharge to pervious space. Disconnection may be coupled with other GI technologies but is not considered an effective standalone option.	Yes	Yes	Yes
	Sump Pump Disconnection	Low	Low	Site specific; more applicable to separate sanitary system; new storm sewers may be required; interaction with homeowners required. The Cities are densely populated and disconnected sump pumps have limited options for discharge to pervious space. Disconnection may be coupled with other GI technologies but is not considered an effective standalone option.	Yes	Yes	Yes
	Combined Sewer Separation	High	High	Very disruptive to affected areas; requires homeowner participation; sewer asset renewal achieved at the same time; labor intensive.	No	Yes	Yes
Combined Sewer Optimization	Additional Conveyance	High	High	Additional conveyance can be costly and would require additional maintenance to keep new structures and pipelines operating.	No	No	Yes
	Regulator Modifications	Medium	Medium	Relatively easy to implement with existing regulators; mechanical controls requires O&M. May increase risk of upstream flooding. Permitees have an ongoing O&M program and system wide replacement program for CSO regulators and tide gates.	Yes	No	Yes

Table 4-2. Collection System Technologies

Collection System Technologies							
Technology Group	Practice	Primary Goals		Implementation & Operation Factors	Consider Combining w/ Other Technologies	Being Implemented	Evaluated in this Report
		Bacteria Reduction	Volume Reduction				
	Outfall Consolidation/Relocation	High	High	Lower operational requirements; may reduce permitting/monitoring; can be used in conjunction with storage & treatment technologies. Combining and relocating outfalls may lower operating costs and CSO flows. It can also direct flow away from specific areas.	Yes	Yes	No
	Real Time Control	High	High	Requires periodic inspection of flow elements; highly automated system; increased potential for sewer backups. RTC is only effective if additional storage capacity is present in the system.	Yes	No	No

Table 4-3. Storage and Treatment Technologies

Storage and Treatment Technologies							
Technology Group	Practice	Primary Goals		Implementation & Operation Factors	Consider Combining w/ Other Technologies	Being Implemented	Evaluated in this Report
		Bacteria Reduction	Volume Reduction				
Linear Storage	Pipeline	High	High	Can only be implemented if in-line storage potential exists in the system; increased potential for basement flooding if not properly designed; maximizes use of existing facilities. Pipe storage for a CSS typically requires large diameter pipes to have a significant effect on reducing CSOs. This typically requires large open trenches and temporary closure of streets to install.	No	No	Yes
	Tunnel	High	High	Requires small area at ground level relative to storage basins; disruptive at shaft locations; increased O&M burden.	No	No	Yes
Point Storage	Tank (Above or Below Ground)	High	High	Storage tanks typically require pumps to return wet weather flow to the system which will require additional O&M; disruptive to affected areas during construction. Several CSO outfalls have space available for tank storage. There may be existing tanks in abandoned commercial and industrial areas to be converted to hold stormwater. Tanks are an effective technology to reduce wet weather CSO's.	No	No	Yes
	Industrial Discharge Detention	Low	Low	Requires cooperation with industrial users; more resources devoted to enforcement; depends on IUs to maintain storage basins. IUs hold stormwater or combined sewage until wet weather flows subside; there may be commercial or industrial users upstream of CSO regulators.	Yes	No	No
Treatment-CSO Facility	Vortex Separators	None	None	Space required; challenging controls for intermittent and highly variable wet weather flows. Vortex separators would remove floatables and suspended solids when installed. It does not address volume, bacteria or BOD.	Yes	No	Yes
	Screens and Trash Racks	None	None	Prone to clogging; requires manual maintenance; requires suitable physical configuration; increased O&M burden. Screens and trash racks will only address floatables.	Yes	No	No
	Netting	None	None	Easy to implement; labor intensive; potential negative aesthetic impact; requires additional resources for inspection and maintenance. Netting will only address floatables.	Yes	Yes	Yes
	Contaminant Booms	None	None	Difficult to maintain requiring additional resources. Contaminant booms will only address floatables.	Yes	No	No
	Baffles	None	None	Very low maintenance; easy to install; requires proper hydraulic configuration; long lifespan. Baffles will only address floatables.	Yes	No	No
	Disinfection & Satellite Treatment	High	None	Requires additional flow stabilizing measures; requires additional resources for maintenance; requires additional system analysis. Disinfection is an effective control to reduce bacteria and BOD in CSO's.	Yes	No	Yes

Table 4-3. Storage and Treatment Technologies

Storage and Treatment Technologies							
Technology Group	Practice	Primary Goals		Implementation & Operation Factors	Consider Combining w/ Other Technologies	Being Implemented	Evaluated in this Report
		Bacteria Reduction	Volume Reduction				
	High Rate Physical/Chemical Treatment (High Rate Clarification Process - ActiFlo)	None	None	Challenging controls for intermittent and highly variable wet weather flows; smaller footprint than conventional methods. This technology primarily focuses on TSS & BOD removal, but does not help reduce the bacteria or CSO discharge volume.	Yes	No	No
	High Rate Physical (FlexFilter)	None	None	Relatively low O&M requirements; smaller footprint than traditional filtration methods. This technology primarily focuses on TSS removal, but does not help reduce the bacteria or CSO discharge volume.	Yes	No	Yes
Treatment-WRTP	Additional Treatment Capacity	High	High	May require additional space; increased O&M burden.	No	No	No
	Wet Weather Blending	Low	High	Requires upgrading the capacity of influent pumping, primary treatment and disinfection processes; increased O&M burden. Wet weather blending does not address bacteria reduction, as it is a secondary treatment bypass for the POTW. Permittee must demonstrate there are no feasible alternatives to the diversion for this to be implemented.	Yes	No	No
Treatment-Industrial	Industrial Pretreatment Program	Low	Low	Requires cooperation with Industrial User's; more resources devoted to enforcement; depends on IU's to maintain treatment standards. May require Permits.	Yes	No	No

5 Costing

Costing was developed as a Class 5 Estimate with a range -50% to +100%. The purchase of land was not included in the costs because there is no land in the vicinity of the regulators available for use. Cost analysis was performed based on the following assumptions:

1. Sewer Separation Costs

    a. Capital costs based on a bid cost for partial separation from 2006 and developed into a unit cost and applied to the 954 acres of CSO drainage area. To convert to 2018 costs, a ratio of 10817:7630 was applied herein, based on the Engineering News Record (ENR) Construction Cost Index (CCI) values for 2018 and 2006, respectively.

    b. O&M costs are estimated at based on 2% of the capital cost (2019c, G&H).

2. Treatment Costs

    a. Capital and O&M costs for PAA disinfection are based on the latest available guidance for permittees (2018, G&H).

3. Storage Tank Costs

    a. Capital costs for tank-storage solutions are based on the latest available guidance for permittees (2018, G&H).

    b. O&M costs for tanks were compiled in accordance with the latest available guidance for permittees (2019c, G&H).

4. Green Infrastructure Costs

    a. Capital costs for various GI solutions are based on the latest available guidance for permittees (2018, G&H).

    b. O&M costs for Bioretention GI solutions were provided as $8,000 per managed acre (2019c, G&H).

    c. O&M costs for Porous Pavement GI solutions were assumed to be $1,250 per managed acre (2018, DEP).

5. Additional Cost Factors

    a. Present-value (PV) of life-cycle costs based on a 20-year period and an interest rate of 2.75% in accordance with the latest available guidance for permittees (2019a, G&H).

b. Based on experience on other similar CSO LTCP projects, HDR applied a capital-cost factor of 2.5 to calculate the probable total project cost (PTPC) of implementing each technology. The PTPC accounts for installation, non-component (electrical, piping, etc.), and indirect costs (freight, permits, etc.) for all storage and disinfection. A breakdown of how this factor was calculated is shown below.

   i. Installation was estimated at 20% of equipment costs based on historic data experienced by HDR and industry standards for typical plants of similar size and complexity.

   ii. Non-component costs including: electrical (10%), piping (10%), instrumentation and controls ($15,000), and civil site work (25%) were estimated based on factors or percentages of equipment costs. These factors account for standard installation commodities, accessories, steal supports and standard testing support.

   iii. Freight was estimated at a lump sum of $20,000.

   iv. Sales tax was estimates at 8%

   v. Permits were estimated at $20,000

   vi. Start up, performance testing, operator training and O&M manual were estimated at $50,000

   vii. Contract overhead and profit includes 29% for the following:

      a. Part time – Project management support, project controls, procurement, quality and safety support.

      b. Full time – Site construction manager (CM), site administration, standard CM travel pack.

   viii. Engineering, administration and legal fees were estimated at 10%

   ix. A contingency of 10% is included for the remaining equipment items and non-component costs

6 Available Land Analysis

The current netting chambers and pump stations for Bluff Road and Palisade Avenue/Lower Main are located on small parcels of land or rights of way. Bluff Road, which accounts for 95% of the discharged CSO, is located on a narrow (20 feet wide) strip of land on the Palisades shown in Figure 6-1. Large additional facilities cannot be located on these sites. The Bluff Road site is located on a narrow right of way at the end of Manatauck Avenue located along side but above Route 5 and the Palisades shown on Figure 6-2. Locating facilities at another distant site would require a large pumping station (30 to 85 mgd) to pump CSO to the tank. Also, Palisades bedrock, shown on Figure 6-3, is throughout the town and would increase the cost of construction.

Figure 6-1. Bluff Road Netting Facility Site

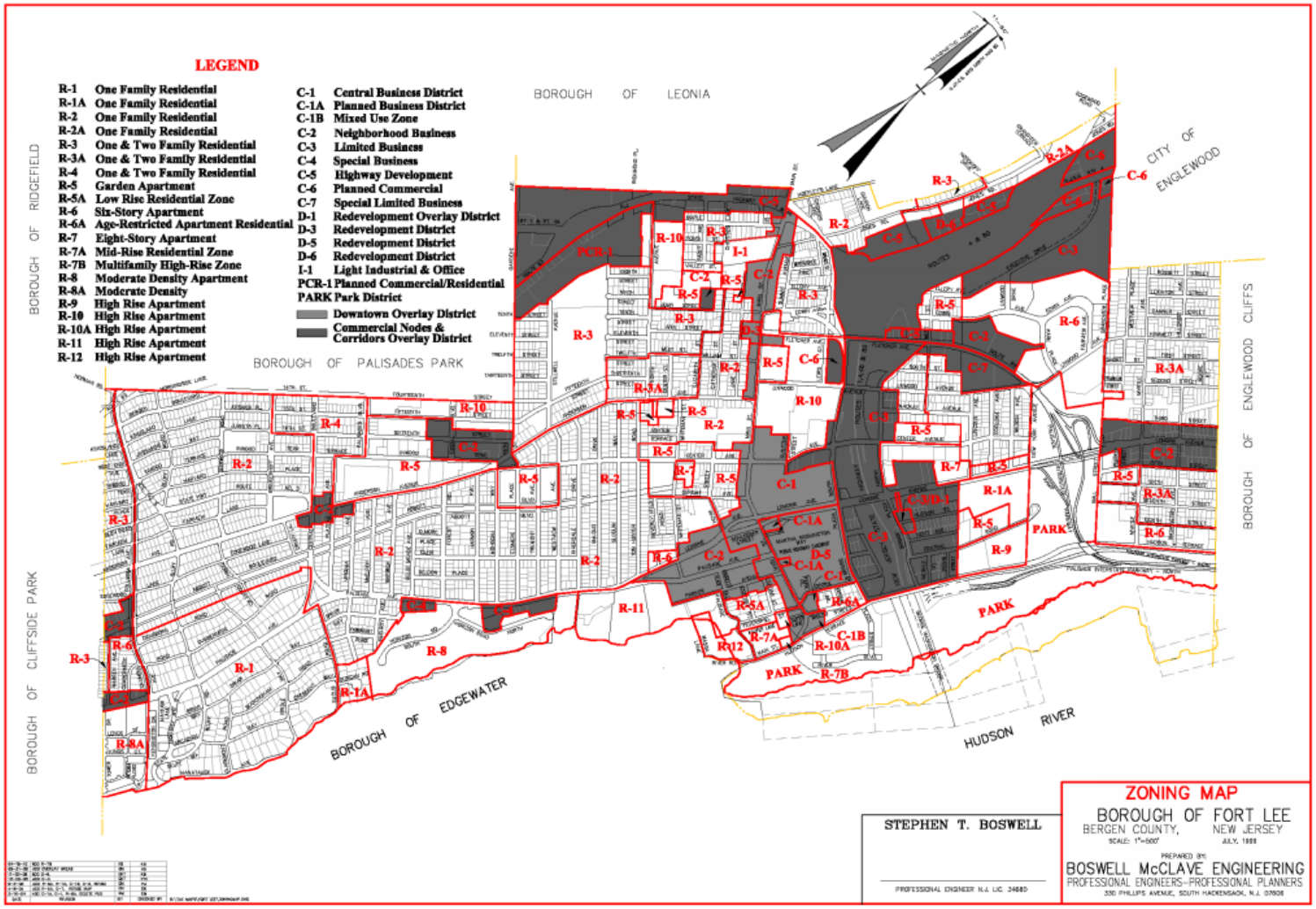

Figure 6-2. Fort Lee Zoning Map

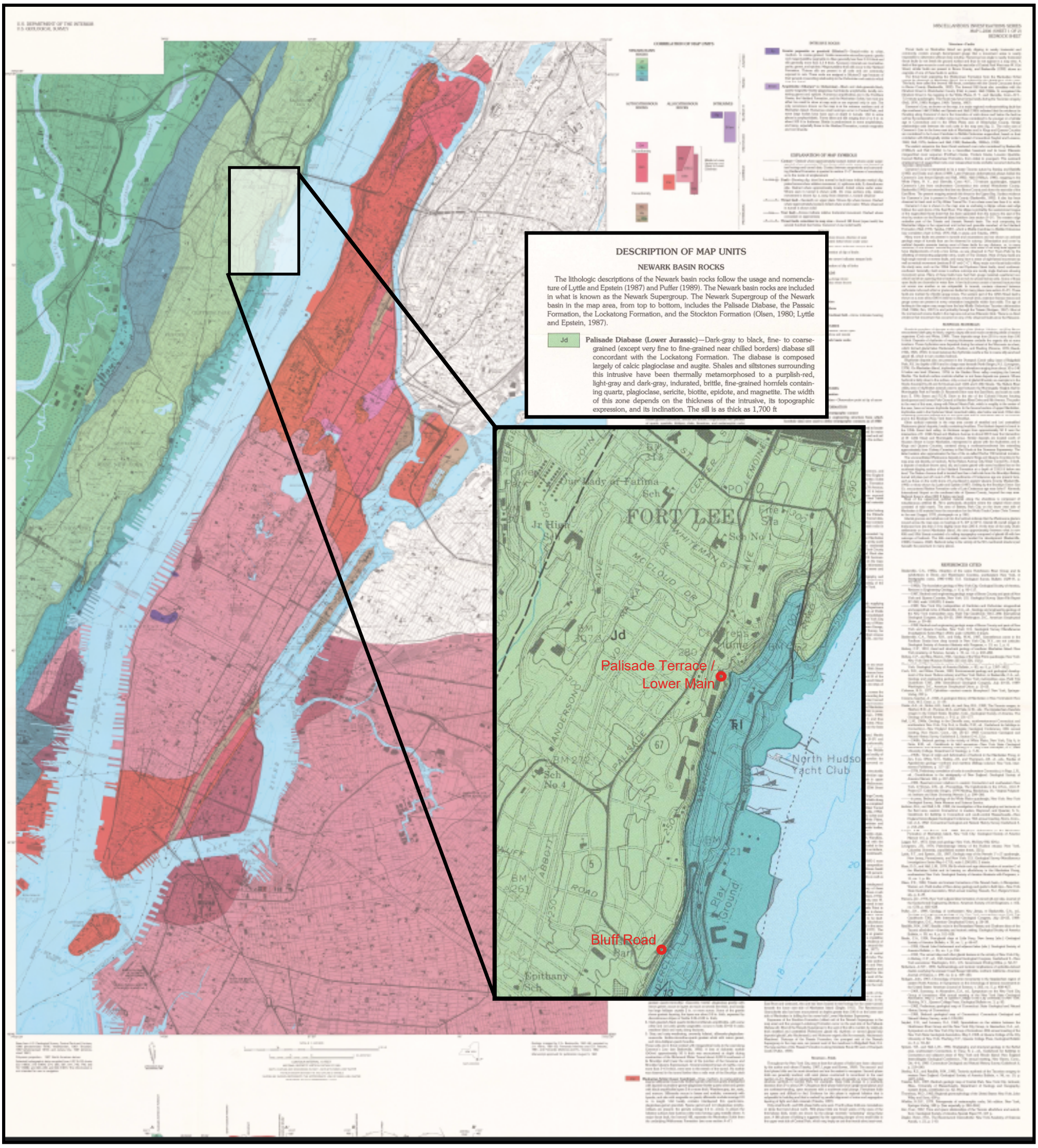

Figure 6-3. Bedrock Map of New York and Parts of Kings and Queens Counties, New York and Bergen and Hudson Counties, New Jersey

7 Alternatives Evaluation

7.1 Introduction

CSO treatment alternatives were investigated in the 2007 LTCP effort. For the current evaluation the 2007 model was updated to account for new developments and CSO improvements. The model was run to provided baseline results and alternatives were evaluated.

7.1.1 Targeted CSO Control Goal

The CSO control goals were:

- 85% CSO Capture;
- 20 CSO overflows per year;
- 12 CSO overflows per year;
- 8 CSO overflows per year; and
- 4 CSO overflows per year.

The control methods were sized to achieve these goals and costs were estimated based on the 2018 Technical Guidance Manual.

7.1.2 Single or Group Overflows

In the 2007 Preliminary Long Term Control Plan disinfection was selected as the preferred alternative if additional CSO control was required. Now, in this LTCP we are in some ways repeating the 2007 analysis but also performing water quality analyses to determine where CSO control or disinfection might be necessary.

7.1.3 Siting Issues

Siting is commonly a subject of most public debate on CSO control projects. Therefore, one of the key considerations in assessing the overall feasibility of a CSO control alternative is the identification of an appropriate site for new facilities. The Borough of Fort Lee is fully developed with not much available open space. Land availability can be an issue as most of the controls are preferred to be located near the waterfront, which is expensive and mostly developed in much of the city. It is recognized that issues involving facility location, land takings, and easements in both public and private lands can lead to disagreements among various stakeholders. Therefore, this alternatives evaluation focuses on the use of the city-owned available sites which have minimal impact on sensitive stakeholders, to be less likely controversial. The environmental, political,

socioeconomic, and regulatory impacts of locating a facility at a designated site will need to be evaluated in detail during the facilities planning and design phase.

Facilities siting in this evaluation is preliminary in nature and it is based on the space requirements. A buffer for roadways and access, potential conflicts with above ground existing utilities at the site, highways, and local streets are also part of the preliminary facility siting considerations.

7.1.4 Institutional Issues

Institutional constraints include matters related to political issues, public opinion, and other non-technical factors that could impact project approval. Institutional and political factors can influence CSO control projects as most part of such project is generally funded by tax payers or sewer rate payers. The general public must be convinced that the proposed project is cost-effective and for the public good, so that possible public rejection is minimized. This is important to support the fundraising needed for implementation of the project. The Borough of Fort Lee has continued raising public awareness about the LTCP project through ongoing public participation activities with PVSC, as stressed in the NJPDES permit, and EPA policy and related guidance for the LTCP. It is to be noted that the Borough of Fort Lee is a densely developed urban municipality with poverty levels at or above the state average. Therefore, it is acknowledged that negotiations amongst politicians, institutions, and other stakeholders and interested parties are necessary to ensure that CSO control measures that are technically feasible for the Borough of Fort Lee are also financially and politically feasible.

It is to be mentioned that budgetary constraints of the permittee and, indirectly, constituent rate payers are not explicitly considered in this analysis. It is recognized that while certain alternatives may provide measurable benefit within other evaluation criterion, it may be the case that overall costs prove to be prohibitive to implementation for those alternatives.

7.1.5 Implementability

In addition to the cost, performance and political and institutional aspects; several other factors can affect implementation of a potential alternative. The following are some of the key implementability issues that have been part of preliminary considerations in the alternatives evaluation, but they have not been reviewed or analyzed in depth. The considerations made in this evaluation are solely based on the available information obtained from various sources.

Environmental Issues: These issues may be related to land conservation, use and acquisition, zoning changes, easement, traffic and site access, noise and vibration, floodplains and zoning, wetland buffer zones, utilities relocation and loss of services, and short term impacts water or air quality. The Borough of Fort Lee has waterfront land on the Passaic River which is used to a limited extent

both commercially and for boating recreation. Alternatives that fit with existing land uses and favor City property will receive a positive consideration under this evaluation. Any specific permits that would be required to implement a CSO control alternative would be identified at the facility planning and design phase.

Consideration for no CSO discharges to sensitive areas is a requirement in the evaluation of the CSO control alternatives. The NJDEP approved sensitive area study report identified no such area for the Borough of Fort Lee's CSO receiving waters. Therefore, CSO discharges to sensitive areas is not an issue for this alternatives evaluation.

Constructability: This relates to the ease of construction. Constructability can be impacted by work site subsurface conditions. Adequate geologic data for the subsurface conditions is not currently available at the Borough of Fort Lee, so there is a large amount of uncertainty as to the rock and soil conditions. It is anticipated that alternatives with unsuitable soils, extensive rock or high groundwater requiring extensive dewatering or rerouting of drainage patterns may impose construction challenges. Alternatives involving complex designs and specialized construction would tend to drive up costs. Therefore, alternatives with few constructability issues will be preferred.

Reliability: Reliability of CSO control alternatives is a significant technical issue. The operating history of existing similar installations can help predict the reliability of a proposed solution. System components must function properly when required, particularly for CSO facilities that operate only on an intermittent basis. Alternatives that rely on simpler or less complex equipment and automation are inherently more reliable. Alternatives involving systems with unknown or poor track records will not be favored.

Ease of Operations: Operability issues involve both process and personnel related considerations. Alternatives involving equipment and system components that are relatively easy to operate and require reasonable operator assistance will be preferred. Unfavorable alternatives would involve highly specialized systems that require extensive training and staffing requirements.

Multiple Use Considerations: Multiple-use CSO control facilities can help to gain public and institutional acceptance. An alternative would be considered advantageous if it can serve another beneficial purpose while also mitigating CSOs. Examples include parking facilities over storage/treatment tanks, and recreational opportunities such as constructing bike paths over the routes of consolidation conduits or improving river access, which are possible enhancements that have been shown to provide additional public benefit.

Compatibility to Phased Construction: Given the cost of CSO control facilities, alternatives that can be implemented in smaller parts can be more affordable than a single large project. Phasing can lessen the immediate financial impact on rate payers with some immediate reliefs to CSO problems. Preferable

alternatives will need to meet current needs, but also will adapt to future conditions.

7.1.6 Public Acceptance

Community acceptance of a recommended solution is essential to its success. All permittees are required to involve the public, regulators, and other stakeholders throughout the LTCP development process. As such, the PVSC and the Borough of Fort Lee itself have continued raising public awareness of the LTCP development through ongoing public participation activities, as stressed in the NJPDES permit, and EPA policy and related guidance for the LTCP.

PVSC has held several quarterly regional supplemental CSO team public meetings over the course of the LTCP development effort. Local meetings were held in conjunction with the PVSC's regional supplemental CSO team meetings. The details of the public participation process and the associated outreach program activities have been documented in the January 2019 revision of the Public Participation Process Report submitted to NJDEP.

Thus far, the regional Supplemental CSO team public meetings have continued being held and the supplemental CSO team members have been encouraged to provide feedback on further LTCP development milestone deliverables, including the Development and Evaluation of Alternatives. Further, the City has presented its CSO alternatives evaluation approach in tandem with other permitees at the March 7, 2019 regional supplemental CSO pubic meeting (Session-11) held at the NJTPA's conference room. The majority of comments received thus far have been verbal comments, some of which are related to application of GI. To date, the Borough of Fort Lee has not received any comments on any of the draft LTCP submittals provided to the supplemental CSO team members for review and feedback. It is anticipated that the Borough of Fort Lee will present the results of the alternatives evaluation in one additional regional supplemental CSO team public meeting to discuss and address public comments in the NJDEP submittal as it would be necessary.

7.1.7 Performance

CSO control alternatives are generally evaluated using several measures, ranging from cost and performance to ancillary benefits and qualitative criteria. The EPA's CSO Policy requires CSO permittees to evaluate a reasonable range of control alternatives to reduce or eliminate CSO discharges to ensure that water quality standards are met. An alternative must include options to address all goals of the LTCP in a cost-effective manner relative to other options. The alternative must also be able to perform well under intermittent and variable flow conditions. A comprehensive set of reasonable alternatives with ranges of CSO control goals for percent capture or number of overflows or pathogen reduction with the ability to beneficially integrate with the hydraulically connected communities are among the considerations in this analysis.

7.2 Preliminary Control Program Alternative

Section 4 described the CSO control technology screening performed to identify the preliminary CSO control measures. The screened control measures were further evaluated and described in the following sections. The following section presents overview of various control alternatives developed for the Borough of Fort Lee. The preliminary alternatives with detailed evaluations are:

- Green infrastructure (GI);
- Storage tanks; and
- Treatment – Disinfection.

7.2.1 Baseline

A baseline analysis was performed to identify annually how many CSO discharge events occur, what volumes of CSO are discharged and what the percent capture of CSO is. 2004 was used as the design rainfall year. The results are shown in Table 7-1 and indicate that FL-001 and FL-002 accomplish 84.7% capture of CSO before any controls are selected.

Table 7-1. Baseline CSO Events and Volumes

Outfall	FL-001	FL-002
CSO Event Count	58	20
Volume (MG)	82.48	4.71
Total CSO Volume (MG)	87.19	
CSO Capture (%)	84.7	

There are several ways to compute CSO capture. The way that we computed it defines the wet weather period starting when the accumulated rainfall is greater than 0.1 inches and ends 12 hours after precipitation ends. The flow volume within this period is counted as wet weather. The annual wet weather flow volume is divided by the annual total flow volume to compute the percent capture.

This is the method being used by PVSC. This calculation approach is conservatively low and will produce lower estimates of capture than other methods. As an example using the method that New York City uses (using the total rain volume, not just the volume over 0.1 inches) would produce approximately 90% CSO capture. But regardless of the method used Fort Lee is near or above 85% capture.

The CSO Control Policy says:

"A program that meets any of the criteria listed below would be presumed to provide an adequate level of control to meet the water quality-based requirements of the CWA, provided the permitting authority determines that such presumption is reasonable in light of the data and analysis conducted in the characterization, monitoring, and modeling of the system and the consideration of sensitive areas described above. These criteria are provided because data and modeling of wet weather events often do not give a clear picture of the level of CSO controls necessary to protect WQS.

i. No more than an average of four overflow events per year...

ii. The elimination or the capture for treatment of no less than 85% by volume of the combined sewage collected in the CSS during precipitation events on a system-wide annual average basis...

iii. The elimination or removal of no less than the mass of pollutants, identified as causing water quality impairment..., for the volumes that would be eliminated or captured for treatment under paragraph ii... " (Section II.C.4.a.)

By this policy Fort Lee is almost in compliance with the 85% capture criteria. To attain the criteria and be in compliance with the policy Fort Lee will consider implementing some reasonable degree of additional CSO control. The following describes the levels of control that are being considered.

7.2.2 Inflow Control

GI can be used as a complementary CSO control technology in combination with other alternatives. This alternative was evaluated alone to find out if GI could have a significant impact on CSO volume and frequency reduction in the Bluff Road pumping station drainage area. Two different target levels of GI control were evaluated. One of them was to manage 1" of storm water runoff generated from 5% of impervious surfaces, another target level was to manage 1" of storm water runoff generated from 10% of impervious surfaces. Total impervious surface in Bluff Road pumping station drainage area is about 131 acres, GI control area will be about 6.5 acres and 13 acres with 5% and 10% control level respectively. Table 7-2 shows the CSO volume and frequencies before and after the implementation of GI. The CSO volume was reduced about 3% with 5% GI control and was reduced about 7% with 10% GI control comparing with the baseline condition. But the CSO frequencies did not decrease a lot. The type of runoff surface of Fort Lee is rock, there is not much infiltration capacity available for the GI thus the runoff generated will not be intercepted. This alternative will not feasible for CSO control. Because of the relatively small impact achievable with GI, HDR decided to evaluate all alternatives conservatively, without GI, with the assumption that any additional impact of GI, however minor, would be considered in the development of the final selected alternatives.

Table 7-2. Overflow Volumes and Frequencies with GI Alternative

GI Alternative								
	Baseline		5% GI-Bluff Road			10% GI-Bluff Road		
Outfall	CSO Volume	CSO Events	CSO Volume	CSO Events	Volume Reduction	CSO Volume (MG)	CSO Events	Volume Reduction
FL-001	82.5	58	79.8	57	3%	77.0	58	7%

7.2.3 Sewer System Separation

Sewer separation was considered in 2007 and costs were based on bid costs for a section of Fort Lee. These costs were escalated to 2018 costs based on ENR. The 2018 cost is estimated at $400,000,000. It considered rock construction costs because of high bedrock in Fort Lee.

Although this is a cost prohibitive alternative, a smaller sewer separation project(s) could be undertaken to improve Fort Lees CSO capture to 85%. It is estimated that separating approximately 10 acres could accomplish 85% CSO reduction. This cost would be approximately $6,250,000.

7.2.4 Satellite Storage

The conceptual evaluation of the storage tank for CSO reduction was performed. It is assumed that a storage tank would be located near the existing outfall and it would be below the ground. Storage tank was evaluated at outfall FL-001 and FL-002. CSO is stored in the tank during wet weather events. The stored CSO is pumped back to the interceptor for conveyance to the BCUA treatment plant during dry weather and when the system capacity is available. Five scenarios were analyzed to size the storage tank in order to achieve CSO frequencies of 0, 4, 8, 12, and 20 overflows per year. For example, in order to achieve 4 CSO events control target citywide per year, the sizing criteria for the storage tank is to capture the 5th biggest rainfall event during the typical year of 2004. Tank dewatering pump back rate is no more than 75% of the total average dry weather flows and the tank can be dewatered within 72 hours except for zero CSO control level at FL-001. Table 7-3 shows the size of the tank required at each CSO control level. Table 7-4 summarizes the CSO volumes and number of CSO events not captured and retained in the tank at each level of control. Storage tank alternative is considered as a primary solution for the CSO frequencies control because it is able to reach frequency control target without combining with other control alternatives.

Table 7-3. Storage Tank Size (MG) at Each Level of Control

CSO Event Target/yr	FL-001	FL-002
0	12.5	1.2
4	4.6	0.4
8	4.1	0.3
12	3.1	0.1
20	2.0	0.0

Table 7-4. Overflow Volumes and Events with Storage Tank Alternative

CSO Event Target/yr	CSO Volume (MG)				CSO Events			
	FL-001	FL-002	Total	Reduction	FL-001	FL-002	Total	Reduction
Baseline	82.5	4.7	87.2		58	20	60	
0	0.0	0.0	0.0	100%	0	0	0	100%
4	8.6	1.0	9.6	89%	4	3	4	93%
8	11.1	1.8	13.0	85%	8	6	8	87%
12	20.0	2.9	23.0	74%	12	11	12	80%
20	34.0	4.7	38.7	56%	20	20	20	67%

7.2.5 Treatment - Disinfection

Disinfection of combined sewer overflows is another option in the Borough of Fort Lee. Disinfection by Peracetic Acid (PAA) serves as the basis in the evaluation. Pathogens represent the primary pollutant of concern for CSO discharges. Disinfection facilities are sized based on the maximum CSO discharge flow rate for each event to fully treat all but 4, 8, 12, and 20 CSO discharges per year. For the target of 4 CSO events per year, the 5th largest storm in the typical year will be captured and disinfected. For the storm events larger than the 5th event, CSO discharges will be partially treated, full treatment is achieved only during times that CSO discharges are less than the maximum discharge rate. Where full treatment is achieved, disinfection is assumed to remove 99.9% of pathogens (a "3-log kill."). This preliminary disinfection alternative assumes that PAA disinfection will be implemented at locations between the existing regulators and the existing outfalls. Table 7-5 presents the peak flow rates at each CSO control target and Table 7-6 summarizes the volume of partially treated overflows at each target.

Table 7-5. CSO Peak Flow Rates (MGD) at Each Level of Control

CSO Event Target/yr	FL-001	FL-002
0	100.8	9.8
4	85.3	7.8
8	55.4	3.3
12	42.6	3.2
20	32.3	1.0

Table 7-6. Partially Treated CSO Volumes (MG) at Each Control Target

CSO Event Target/yr	FL-001	FL-002	Total	Volume Reduction
Baseline	82.5	4.7	87.2	
0	0.0	0.0	0.0	100%
4	3.6	0.3	3.9	96%
8	11.3	2.1	13.4	85%
12	15.8	2.1	18.0	79%
20	20.2	4.0	24.2	72%

It is possible that suspended solids removal may be required before disinfection is applied. Fort Lee's CSO's are very dilute with TSS ranging from 20 to 80 mg/L, so the added value of filtration may be investigated by bench and pilot testing. We have made cost estimates for disinfection and high rate suspended solids removal (FlexFilter). If we can meet the treatment objective without filtration then Fort Lee will use the lower cost treatment process of PAA disinfection alone.

7.3 Preliminary Selection of Alternative

Cost analysis was performed for GI, storage tank, and PAA disinfection with and without pretreatment (solids removal) in the Borough of Fort Lee. Assumptions used to estimate capital and O&M costs are described as follows.

1. Sewer Separation Costs
    a. Capital costs for sewer separation of the 935 acres combined area is about $400,000,000. This is based on a normalized cost of $235,233 per acre (2006, HMM). To convert to 2018 costs, a ratio of 10817:7630 was applied herein, based on the Engineering News Record (ENR) Construction Cost Index (CCI) values for 2018 and 2006, respectively.

b. O&M costs are estimated at $8,000,000 which is based on 2% of the capital cost (2019c, G&H).

2. Treatment Costs

   a. Capital and O&M costs for PAA disinfection with high rate filtration are based on the latest available guidance for permittees (2018, G&H) and are in Table 7-7

3. Storage Tank Costs

   a. Capital costs for tank-storage solutions are based on the latest available guidance for permittees (2018, G&H) and are in Table 7-7.

   b. O&M costs for tanks are based on operational costs at $235,000 and maintenance costs at 3% of the construction cost, in accordance with the latest available guidance for permittees (2019c, G&H) and are in Table 7-7.

4. Green Infrastructure Costs

   a. Capital costs for various GI solutions are based on the latest available guidance for permittees (2018, G&H) and are in Table 7-8.

   b. O&M costs for Bioretention GI solutions were provided as $8,000 per managed acre (2019c, G&H) and are in Table 7-8.

   c. O&M costs for Porous Pavement GI solutions were assumed to be $1,250 per managed acre (2018, DEP) and are in Table 7-8.

5. Additional Cost Factors

   a. Present-value (PV) of life-cycle costs based on a 20-year period and an interest rate of 2.75% in accordance with the latest available guidance for permittees (2019a, G&H).

   b. Based on experiences on other similar CSO LTCP projects, HDR applied a capital-cost factor of 2.5 to calculate the probable total project cost (PTPC) of implementing each technology. The PTPC accounts for installation, non-component (electrical, piping, etc.), and indirect costs (freight, permits, etc.) for all storage and disinfection. A breakdown of how this factor was calculated is shown below.

      i. Installation was estimated at 20% of equipment costs based on historic data experienced by HDR and industry standards for typical plants of similar size and complexity.

      ii. Non-component costs including: electrical (10%), piping (10%), instrumentation and controls ($15,000),

and civil site work (25%) were estimated based on factors or percentages of equipment costs. These factors account for standard installation commodities, accessories, steal supports and standard testing support.

iii. Freight was estimated at a lump sum of $20,000.

iv. Sales tax was estimates at 8%.

v. Permits were estimated at $20,000.

vi. Start up, performance testing, operator training and O&M manual were estimated at $50,000.

vii. Contract overhead and profit includes 29% for the following:

1. Part time – Project management support, project controls, procurement, quality and safety support.
2. Full time – Site construction manager (CM), site administration, standard CM travel pack.

viii. Engineering, administration and legal fees were estimated at 10%

ix. A contingency of 10% is included for the remaining equipment items and non-component costs

Table 7-7. Total Capital Cost, Total 20-yr O&M Cost, Raw and PTPC as 20-yr Present Value

	PAA Only	PAA w/ FlexFilter	Tanks
0 CSOs per year			
Raw Capital Cost ($M)	$ 1.35	$ 28.95	$ 50.64
20 yr PV O&M Cost ($M)	$ 3.90	$ 5.75	$ 30.29
Total 20 yr PV Cost ($M)	$ 5.25	$ 34.71	$ 80.94
Probable Total 20 yr PV Cost ($M)	$ 7.27	$ 78.14	$ 156.90
4 CSOs per year			
Capital Cost ($M)	$ 1.27	$ 24.67	$ 22.60
20 yr PV O&M Cost ($M)	$ 3.40	$ 5.01	$ 17.48
Total 20 yr PV Cost ($M)	$ 4.67	$ 29.68	$ 40.07
Probable Total 20 yr PV Cost ($M)	$ 6.58	$ 66.68	$ 73.97
8 CSOs per year			
Capital Cost ($M)	$ 1.07	$ 16.16	$ 20.11
20 yr PV O&M Cost ($M)	$ 2.38	$ 3.48	$ 16.34
Total 20 yr PV Cost ($M)	$ 3.45	$ 19.63	$ 36.45
Probable Total 20 yr PV Cost ($M)	$ 5.05	$ 43.87	$ 66.61
12 CSOs per year			
Capital Cost ($M)	$ 1.00	$ 12.97	$ 16.31
20 yr PV O&M Cost ($M)	$ 1.99	$ 2.89	$ 14.61
Total 20 yr PV Cost ($M)	$ 2.99	$ 15.85	$ 30.91
Probable Total 20 yr PV Cost ($M)	$ 4.50	$ 35.30	$ 55.37
20 CSOs per year			
Capital Cost ($M)	$ 0.85	$ 9.75	$ 11.25
20 yr PV O&M Cost ($M)	$ 1.60	$ 2.29	$ 8.72
Total 20 yr PV Cost ($M)	$ 2.44	$ 12.04	$ 19.97
Probable Total 20 yr PV Cost ($M)	$ 3.72	$ 26.66	$ 36.85

Table 7-8. Cost Summary for Green Infrastructure to Control 5% and 10% of Impervious Cover

	Green Infrastructure Type	Capital Cost Min PTPC ($M)	Capital Cost Max PTPC ($M)	20-Yr O&M Cost as PV ($M)	Min PTPC 20-Yr Life Cycle Cost as PV ($M)	Max PTPC 20-Yr Life Cycle Cost as PV ($M)
5% GI (~6.5 Acres)	Rain Garden	$ 1.58	$ 5.01	$ 0.80	$ 2.38	$ 5.81
	Right-of-Way Bioswale	$ 2.46	$ 8.21	$ 0.80	$ 3.26	$ 9.01
	Green Roof	$ 7.88	$ 40.08	$ 0.80	$ 8.68	$ 40.88
	Porous Asphalt	$ 4.27	$ 8.95	$ 0.13	$ 4.40	$ 9.08
	Pervious concrete	$ 5.01	$ 10.02	$ 0.13	$ 5.13	$ 10.14
	Permeable Interlocking Concrete Pavers	$ 2.14	$ 6.08	$ 0.13	$ 2.26	$ 6.20
10% GI (~13 Acres)	Rain Garden	$ 3.15	$ 10.02	$ 1.60	$ 4.75	$ 11.62
	Right-of-Way Bioswale	$ 4.93	$ 16.43	$ 1.60	$ 6.53	$ 18.03
	Green Roof	$ 15.77	$ 80.16	$ 1.60	$ 17.37	$ 81.76
	Porous Asphalt	$ 8.54	$ 17.90	$ 0.25	$ 8.79	$ 18.15
	Pervious concrete	$ 10.02	$ 20.04	$ 0.25	$ 10.27	$ 20.29
	Permeable Interlocking Concrete Pavers	$ 4.27	$ 12.16	$ 0.25	$ 4.52	$ 12.41

For the cost of GI, the latest guidance available to permittees (2018, G&H and 2019c, G&H) provides capital and O&M costs for a variety of GI technologies, O&M costs are available for porous-pavement technologies from the NJDEP (2018, NJDEP). As widespread implementation of GI could involve a variety of GI technologies depending on specific site conditions, a range of costs is provided in Tables 7-8 and Table 7-9. Table 7-7 shows the capital costs, O&M costs, and raw total 20-yr present value cost for each GI technology for implementation at 5% and 10% of impervious surfaces. Table 7-8 shows the raw and PTPC cost range of green infrastructure reported as $M/MG CSO controlled and $M/impervious acre controlled.

Table 7-9. Normalized Green Infrastructure Cost Ranges

	Green Infrastructure Type	Min $ Million/MG CSO Reduced	Max $ Million/MG CSO Reduced	Min $ Million/Acre GI	Max $ Million/Acre GI
Raw Capital Cost	Rain Garden	$ 0.53	$ 1.04	$ 0.22	$ 0.43
	Right-of-Way Bioswale	$ 0.66	$ 1.51	$ 0.27	$ 0.62
	Green Roof	$ 1.46	$ 6.23	$ 0.60	$ 2.56
	Porous Asphalt	$ 0.68	$ 1.37	$ 0.28	$ 0.56
	Pervious concrete	$ 0.79	$ 1.53	$ 0.32	$ 0.63
	Permeable Interlocking Concrete Pavers	$ 0.36	$ 0.95	$ 0.15	$ 0.39
Probable Capital Cost	Rain Garden	$ 0.88	$ 2.15	$ 0.36	$ 0.88
	Right-of-Way Bioswale	$ 1.21	$ 3.34	$ 0.50	$ 1.37
	Green Roof	$ 3.22	$ 15.14	$ 1.32	$ 6.22
	Porous Asphalt	$ 1.63	$ 3.36	$ 0.67	$ 1.38
	Pervious concrete	$ 1.90	$ 3.76	$ 0.78	$ 1.54
	Permeable Interlocking Concrete Pavers	$ 0.84	$ 2.30	$ 0.34	$ 0.94

7.3.1 Evaluation Factors

This preliminary evaluation considered several factors to gauge the technical feasibility and applicability for CSO controls in the Borough of Fort Lee in conjunction with the hydraulically connected communities. In general, the alternatives evaluation factors included but not limited to receiving water quality standards and uses and LTCP goals, sewer system characteristics and optimization opportunities, wet weather flow characteristics, hydraulic and pollutant loading, climate, implementation requirements (land, neighborhood, noise, disruption), and maintenance requirements. Pathogen reduction in CSO discharges and the frequency and volume of untreated CSO discharges are priorities for all alternatives along with their potential cost implications, and public acceptance and interests. The other significant factors considered in alternatives evaluation are:

- Performance capabilities and effectiveness under future (baseline) conditions.
- Applicability at a single CSO outfall or at grouped outfalls and capability to minimize number of new facilities required.

- Capability to beneficially integrate with hydraulically connected communities and the constraints involved.
- Community benefits (GI, as an example), and potential Social and environmental impacts.
- Risk and potential safety hazards to operators and public.
- LTCP Regulatory (EPA and NJSPDES) requirements

7.3.2 Regulatory Compliance

The alternatives evaluation included in the report was prepared in compliance with the LTCP regulatory (EPA and NJSPDES) requirements and associated guidance documents. The analysis was conducted in cooperation with PVSC and the permittees within the PVSC Sewer District. The evaluation considered a wide range of BMPs and CSO control measures, including all specified in Part IV G.4.e of the NJPDES permit, to identify the preliminary alternatives that will provide the levels of CSO controls necessary to develop a LTCP as required by the State and Federal regulations. The selection of the preliminary alternatives is based on multiple considerations including public input, water quality benefits and designated use, costs, and other aspects as previously outlined. The preliminary alternatives will result in full attainment of the existing pathogen water quality criteria providing the maximum bacterial reduction reasonably attainable. The remaining CSO discharges will not preclude the attainment of the water quality standards for bacteria or the designated uses of the receiving waters.

Further refinement and modifications of the alternatives is expected as the City further develops the LTCP through selection of the compliance approach in cooperation with the PVSC and hydraulically connected communities.

7.3.3 Selection of Preliminary Alternatives

The evaluation and screening of the range of control alternatives described above resulted in a trend toward the use of storage or disinfection technologies as the preliminary solutions based on the effectiveness of CSO volume and frequency control. Table 7-10 presents a summary of CSO volume and CSO events for disinfection and tank storage.

The lifecycle cost for achieving 85% capture by sewer separation or GI ranges from $6,250,000 to $10,000,000. We have also computed the lifecycle costs for 0 to 20 CSO overflows per year shown in Table 7-11. For tank storage, the most expensive CSO control alternative considered, the range is $47,000,000 to $167,000,000. For filtration with disinfection the cost range from $36,000,000 to $85,000,000. If disinfection alone proves to be a viable option then costs may

range from $3,720,000 to $7,270,000.These evaluations of alternatives will serve as a base for the consideration and development of final selected CSO control plan in Fort Lee. We believe the most cost effective solution for meeting water quality objectives and complying with the EPA CSO control policy will be GI, sewer separation or disinfection with PAA.

Table 7-10. CSO Event and Volume reduction for the CSO Control Alternatives

Alternatives	FL-001		FL-002		Total CSO (MG)	Volume Reduction Beyond 84.7% Capture	CSO Capture (%)
	AAOV(MG)	CSO Event	AAOV(MG)	CSO Event			
Baseline(Future)	82.5	58	4.7	20	87.2	0	84.7%
0 CSO-Tanks	0	0	0	0	0	100.00%	100.00%
4 CSOs-Tanks	8.6	4	1	3	9.6	89.00%	98.32%
8 CSOs-Tanks	11.1	8	1.8	6	13	85.10%	97.72%
12 CSOs-Tanks	20	12	2.9	11	23	73.70%	95.98%
20 CSOs-Tanks	34	20	4.7	20	38.7	55.60%	93.21%
0 CSO-PAA	0	0	0	0	0	100.00%	100.00%
4 CSOs-PAA	3.6	4	0.3	4	3.9	95.50%	99.31%
8 CSOs-PAA	11.3	8	2.1	8	13.4	84.60%	97.64%
12 CSOs-PAA	15.9	12	2.1	12	18	79.40%	96.85%
20 CSOs-PAA	20.2	20	4	20	24.2	72.20%	95.75%

Table 7-11. PV Cost Range for CSO Control Alternatives

CSO Event Target/yr	Maximum PV Cost ($M)			Minimum PV Cost ($M)		
	Tank Storage	GI of 5% of Impervious Surface	Total Cost	PAA Disinfection with FlexFilter	GI of 5% of Impervious Surface	Total Cost
0	$ 156.90	$ 10.14	$ 167.05	$ 76.40	$ 10.14	$ 86.55
4	$ 73.97	$ 10.14	$ 84.11	$ 44.38	$ 10.14	$ 54.52
8	$ 66.61	$ 10.14	$ 76.75	$ 42.84	$ 10.14	$ 52.99
12	$ 55.37	$ 10.14	$ 65.52	$ 34.46	$ 10.14	$ 44.61
20	$ 36.85	$ 10.14	$ 46.99	$ 26.01	$ 10.14	$ 36.15

14 City of Hackensack Development and Evaluation of Alternatives Report (Attached)

ARCADIS Design & Consultancy for natural and built assets

The City of Hackensack

COMBINED SEWER SYSTEM DEVELOPMENT AND EVALUATION OF ALTERNATIVES REPORT

NJPDES Permit No. NJ0108766

July 1, 2019

COMBINED SEWER SYSTEM DEVELOPMENT AND EVALUATION OF ALTERNATIVES REPORT

NJPDES Permit No. NJ0108766

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CERTIFICATION

City of Hackensack Development and Evaluation of Alternatives Report

Submitted on behalf of the City of Hackensack
NJPDES Number NJ0108766

Approval of Report

Permittee: 6/27/2019

Susan Banzon
Project Manager, City of Hackensack

Date

NJPDES Certification:

Per Part IV.D.1.b of the Permit, all reports submitted to the Department pursuant to the requirements of this permit shall comply with the signatory requirements of N.J.A.C. 7:14A-4.9., and contain the following certification:

"I certify under penalty of law that this document and all attachments were prepared under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons who manage the system, or those persons directly responsible for gathering the information, the information submitted is, to the best of my knowledge and belief, true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for purposely, knowingly, recklessly, or negligently submitting false information".

Permittee: 6/27/2019

Susan Banzon
Project Manager, City of Hackensack

Date

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Appendix A Detailed Total Capital, O&M, and Present Worth Costs

ACRONYMS AND ABBREVIATIONS

2007 Report	2007 LTCP Cost and Performance Analysis Report
Alternatives Report	Development and Evaluation of Alternatives Report
BCMP	Baseline Compliance Monitoring Program
BCUA	Bergen County Utilities Authority
CCI	Construction Cost Index
CFS	Cubic Feet per Second
Characterization Report	Sewer System Characterization Report
City	City of Hackensack
COD	Chemical Oxygen Demand
CSM	Combined Sewer Management
CSO	Combined Sewer Overflow
CSS	Combined Sewer System
CWA	Clean Water Act
DO	Dissolved Oxygen
ENR	Engineering News Record
EPA	United States Environmental Protection Agency
GPCD	Gallons Per Capita Daily
I/I	Inflow and Infiltration
LTCP	Long Term Control Plan
MG	Million Gallons
MGD	Million Gallons per Day
NJDEP	New Jersey Department of Environmental Protection
NJPDES	New Jersey Pollution Discharge Elimination System
NJTPA	North Jersey Transportation Planning Authority
NMS	National Marine Sanctuaries
O&M	Operations and Maintenance
ONRW	Outstanding National Resource Waters
PAA	Peracetic Acid
PACP	Pipeline Assessment Certification Program
Permit	Permit No. NJ0108766
PPP Report	Public Participation Process Report
PVSC	Passaic Valley Sewerage Commission
Rutgers Report	Green Infrastructure Feasibility Study for Hackensack, prepared by Rutgers Cooperative Extension Water Resources Program
SE1	Saline Estuary 1

STP	Sewage Treatment Plant
Supplemental Letter	Supplemental Letter Response – Review of Public Participation Process Report Required by Part IV.D.3.b.iii
TGM 2018	PVSC CSO Long Term Control Plan Updated Technical Guidance Manual, January 2018
TPW	Total Present Worth
TSS	Total Suspended Solids
Work Plan	Hackensack Sewer System Characterization Work Plan
WPCF	Water Pollution Control Facility

EXECUTIVE SUMMARY

Pursuant to the City of Hackensack's (City) New Jersey Pollution Discharge Elimination System (NJPDES) Permit No. NJ0108766 (Permit), this Development and Evaluation of Alternatives Report (Alternatives Report) evaluates a reasonable range of combined sewer overflow (CSO) control alternatives that will meet the water quality-based requirements of the Clean Water Act (CWA) using either the Presumption Approach or the Demonstration Approach in accordance with the City's Long Term Control Plan (LTCP) requirements.

The City's combined sewer system (CSS) consists of two subdrainage areas, which are referred to as Anderson and Court Street. The Anderson Street subdrainage area serves an area of approximately 470 acres and conveys sanitary and combined sewage to a regulator facility at Anderson Street. The Court Street subdrainage area serves an area of approximately 440 acres and conveys sewage to a regulator facility at Court Street. The regulators at Court Street and Anderson Street both overflow to the Hackensack River when the sewers have reached their wet weather capacities. These overflows are referred to as combined sewer overflows, or CSO. The dry weather flow from the City's CSS is conveyed to the Bergen County Utilities Authority (BCUA) Water Pollution Control Facility (WPCF) in Little Ferry, NJ.

The Permit specifies seven CSO control alternatives that should be evaluated by each permittee. Five of those alternatives are evaluated in this Alternatives Report: green infrastructure, increased storage capacity in the collection system, inflow and infiltration (I/I), sewer separation, and treatment of the CSO discharge. The two other alternatives are sewage treatment plant (STP) expansion and/or storage at the plant and CSO related bypass of the secondary treatment portion of the STP. As discussed in this Alternatives Report, the two other alternatives pertain to improvements that may be undertaken by the BCUA at the STP and that may impact the sizing of the five alternatives evaluated herein. The CSO control alternatives were prescreened to determine if certain CSO control alternatives were economical and feasible for the City to further evaluate. The prescreening process utilized the 2007 LTCP Cost and Performance Analysis Report (2007 Report), prepared by Malcolm Pirnie, Inc. (now Arcadis U.S., Inc.) that was required per the City's previous NJPDES Permit No. NJ0105023. City-wide sewer separation, STP expansion, CSO related bypass of secondary treatment at the STP, and maximizing the storage within the City's existing CSS network were prescreened out of further consideration due to outside factors, extensive costs, and limited capacity within the existing CSS.

Green infrastructure, satellite storage tanks, a regional storage tank, a tunnel, I/I reduction, and treatment of CSO discharge were further evaluated in this Alternatives Report. In the City's evaluation of the CSO control alternatives, the City used NJDEP approved hydrologic, hydraulic, and water quality models where applicable. The City utilized the hydrologic/hydraulic model to simulate the existing conditions and conditions as they are expected to exist after construction and operation of the chosen alternative(s). The City evaluated the practical and technical feasibility of the proposed CSO control alternative(s) and the water quality benefits of constructing and implementing various CSO controls or a combination of such controls. The CSO control alternatives were evaluated for 4, 8, 12, and 20 overflows per year as well as elimination or the capture for treatment of no less than 85% by volume of the combined sewage collected in the CSS during precipitation events for the 2004 typical year.

After further evaluation, the most effective CSO control alternatives to minimize overflows were satellite storage tanks or tunnel. Treatment of the CSO discharge was evaluated for disinfection alone and

disinfection with pretreatment. If disinfection alone is determined to be an adequate CSO control alternative, it is a candidate to be selected as a preliminary CSO control alternative for the City. However, because of the uncertainty of future pretreatment requirements, it is not known if this alternative will satisfy the water quality requirements for future permits. The green infrastructure and I/I reduction alternatives are beneficial for the City for a variety of reasons; however, they are not solely adequate to reach the water quality goals. Therefore, a combination of storage tanks or tunnel with green infrastructure and I/I reduction may be the most effective and economical CSO control alternative for the City to incrementally implement in order to reach the required water quality goals. As future conditions change for the City and additional CSO technologies become available or improve, the CSO control alternatives in this Alternatives Report may be revisited in order to suit the City's best interests and needs to meet the water quality goals.

1 INTRODUCTION

1.1 Regulatory Background and Project History

The New Jersey Pollutant Discharge Elimination System (NJPDES) Permit Number NJ0108766 for Combined Sewer Management (CSM) regulates the activities of various components of the City of Hackensack (City) combined sewer system (CSS) and authorizes/controls the release of combined sewer overflow (CSO) discharges into receiving waters. The Final Surface Water Renewal Permit Action (Permit) was issued by the New Jersey Department of Environmental Protection (NJDEP) to the City with an effective date of July 1, 2015 and an expiration date of June 30, 2020.

The City has complied with all report deliverables required per the NJPDES permit to date. The major report deliverables submitted by the City per the NJPDES Permit include the following:

Table 1-1. Summary of Major Report Deliverables for the NJPDES Permit

Permit Condition	Report	59 Month LTCP Due Date	Submitted Date	NJDEP Approval Date
Part IV.D.3.b.i	System Characterization Work Plan (By Arcadis)	January 1, 2016	December 24, 2015 Re. June 21, 2016	July 27, 2016
Part IV.D.3.c	Baseline Compliance Monitoring Program Work Plan (By PVSC)	January 1, 2016	December 29, 2015 Re. February 19, 2016	February 24, 2016
Part IV.D.3.b.ii	System Characterization Report (By Arcadis)	July 1, 2018	June 28, 2018 Re. March 15, 2019	March 19, 2019
Part IV.D.3.b.iii	Public Participation Process Report (By Arcadis)	July 1, 2018	June 28, 2018 Re. January 4, 2019 Re. May 21, 2019	Pending
Part IV.D.3.d	Compliance Monitoring Program Report (By PVSC)	July 1, 2018	June 30, 2018 Re. October 5, 2018	March 1, 2019
Part IV.D.3.b.iv	Consideration of Sensitive Areas Plan (By PVSC)	July 1, 2018	June 28, 2018 Re. October 18, 2018 Re. March 29, 2019	April 8, 2019

Part IV of the Permit establishes the requirements for development of a Long Term Control Plan (LTCP) aimed at minimizing the impacts of CSOs on their respective receiving water bodies. This Development

and Evaluation of Alternatives Report (Alternatives Report) is a subset of a regional Development and Evaluation of Alternatives Report that was prepared and coordinated by the Bergen County Utilities Authority (BCUA). The BCUA regional report, including this subset, City of Hackensack Alternatives Report, was prepared as a cooperative effort among BCUA and the three hydraulically connected CSO Permittees: the City of Hackensack, the Borough of Fort Lee, and the Village of Ridgefield Park.

The objectives of the City's Alternatives Report include the following:

- Screen a broad list of alternatives into a short list of alternatives that are determined to be most suitable for the City's specific site conditions and other evaluation factors, and that are determined to provide feasible solutions to address the Permit requirements and the City's needs;
- Present the development and evaluation of the CSO abatement alternatives as they pertain to the specific site conditions and other influencing factors within the City, including existing and future conditions;
- Develop, through an evaluation process that includes performing model simulations of the various alternatives, the preliminary sizes and locations of those technologies;
- Evaluate the performance of the alternatives. This includes estimating the City's CSO percent volume captured, the reduction in number of overflows, and the reduction of overall CSO volume discharge as a result of implementation of the alternatives; and
- Evaluate the alternatives and various combinations of the alternatives in the factors of siting, institutional issues, public acceptance, performance, implementability, and life cycle costs.

1.2 Summary of System Characterization and Sensitive Area Reports

1.2.1 System Characterization Report Summary

Pursuant to the City's NJPDES Permit, the City submitted a Sewer System Characterization Report (Characterization Report) prepared by Arcadis U.S., Inc. to characterize the City's CSS based on records review, monitoring, and modeling of the system. The City executed a characterization study in accordance with the Hackensack Sewer System Characterization Work Plan (Work Plan) for the LTCP, which was prepared by Arcadis U.S., Inc., was submitted to the NJDEP on June 21, 2016, and was approved by the NJDEP in a letter dated July 27, 2016. After the Work Plan study was completed, the City submitted the Sewer System Characterization Report on June 28, 2018 and it was later approved by the NJDEP in a letter dated March 19, 2019.

The findings from the approved Characterization Report are summarized below:

- The City's CSS consists of two subdrainage areas, which are referred to as Anderson and Court Street. The Anderson Street subdrainage area serves an area of approximately 470 acres and conveys sanitary and combined sewage to a regulator facility at Anderson Street. The Court Street subdrainage area serves an area of approximately 440 acres and conveys sewage to a regulator facility at Court Street. The regulators at Court Street and Anderson Street both overflow to the Hackensack River when

the sewers have reached wet weather capacities. The dry weather flow from the City's CSS is conveyed to the BCUA Water Pollution Control Facility (WPCF) in Little Ferry, NJ.

- The City undertook a monitoring program in 2006 that included rainfall, combined sewer flow, and water quality monitoring. The monitoring program was performed over a three-month data collection period. The results of the rainfall monitoring portion were used to evaluate a "typical" rainfall event and to recommend specific rainfall events for use in development of a CSS computer model. The CSO flow monitoring program provided information on the frequency and duration of overflows in the system relative to the amount of rainfall in a wet weather event; it also provided data for calibration of a computer model of the CSS. Water quality sampling provided information on pollutant wasteloads resulting from the CSO activity.
- A computer model of the City's CSS was developed in 2007 using XP-SWMM software and updated and recalibrated in 2017 using PCSWMM software. The model was based on a 1970 set of survey drawings of the City's sewers and used the 2006 monitoring program data to calibrate and verify the model (in both 2007 and 2017). It was determined that the 2006 monitoring program data was sufficient for calibration in 2017 because there was minimal population change and minimal increase in impervious areas in the City since 2006. The updated model incorporated new condition assessment and survey data that were collected in a 2015 condition assessment program.
- The Characterization Report presented graphs comparing the model results in the updated, recalibrated PCSWMM model to the original XP-SWMM model and the 2006 monitoring program data. The graphs show that the results generally met the calibration criteria for two calibration events and one verification event, as required by the NJDEP approved 2016 Work Plan. 92% of the volume calibration observations were within the targeted range. 75% of the peak flow calibration observations were within the targeted range of -15% and 25%, and 17% of the peak flow calibration observations were conservatively above the targeted range of 25%. 33% of the stage depth difference calibration observations were within the targeted range of -15% and 30%. Sensitivity analyses were performed on the updated model and indicated that the results are sensitive to the vortex valve rating curves that are used in the model simulation. Further, the results are sensitive to the percent impervious, subcatchment area, and conduit roughness coefficient parameters when these parameters are changed significantly; it is noted that these parameters are unlikely to change in any significant manner.
- The Passaic Valley Sewerage Commission (PVSC) conducted a typical year rainfall analysis to identify a "typical" rainfall year that applies to the region of the members of the NJ CSO Group. Based on analysis of rainfall records for Newark International Airport, the analysis determined that 2004 was a typical rainfall year. The updated, recalibrated model was run using the rainfall data for the 2004 calendar year, and the results were used in coordination with 2006 water quality monitoring data to estimate pollutant wasteloads into the Hackensack River.

Per the System Characterization Report approval letter dated March 19, 2019, the NJDEP states:

> "However, the Department is hereby requesting that Table 8-4 (Anderson Street Typical Year Simulation Results & Wasteloads) and Table 8-5 (Court Street Typical Year Simulation Results and Wasteloads) be supplemented with a column for duration of discharge (in hours) for inclusion within the Development and Evaluation of Alternatives for the LTCP as due on July 1, 2019."

To comply with NJDEP's request, Tables 8-4 and 8-5 from the System Characterization Report have been supplemented with a column for duration of discharge (in hours) below.

System Characterization Report Table 8-4: Anderson Street Typical Year Simulation Results and Wasteloads

	Total (All CSO Events)				Pollutant Loadings						
Month	**Days of Over-flow**	**Duration of Discharge (hours)**	**Vol. (MG)**	**% of Annual Volume**	**Fecal**	**Entero**	**CBOD**	**TKN**	**TP**	**TN**	**TSS**
Jan.	4	23.0	1.65	1.6%	1.21E+14	7.32E+13	1,050.6	745.4	27.4	756.6	1,182.3
Feb.	2	27.0	6.03	5.7%	4.43E+14	2.68E+14	3,848.6	2,730.7	100.2	2,771.6	4,331.1
March	5	28.5	1.42	1.3%	1.04E+14	6.30E+13	903.6	641.1	23.5	650.7	1,016.9
April	7	48.8	9.21	8.7%	6.76E+14	4.09E+14	5,873.3	4,167.2	153.0	4,229.6	6,609.5
May	11	43.8	9.07	8.6%	6.67E+14	4.04E+14	5,788.8	4,107.3	150.8	4,168.8	6,514.5
June	6	26.0	6.93	6.6%	5.09E+14	3.08E+14	4,420.7	3,136.6	115.1	3,183.6	4,974.9
July	9	64.8	23.27	22.1%	1.71E+15	1.03E+15	14,846.0	10,533.6	386.7	10,691.4	16,707.1
Aug.	8	32.5	8.60	8.2%	6.32E+14	3.83E+14	5,489.1	3,894.7	143.0	3,953.0	6,177.2
Sept.	7	42.5	23.48	22.3%	1.72E+15	1.04E+15	14,976.3	10,626.1	390.1	10,785.2	16,853.7
Oct.	3	14	0.79	0.7%	5.79E+13	3.51E+13	503.0	356.9	13.1	362.3	566.1
Nov.	8	39	9.31	8.8%	6.84E+14	4.14E+14	5,938.7	4,213.6	154.7	4,276.7	6,683.1
Dec.	6	40.3	5.69	5.4%	4.18E+14	2.53E+14	3,628.1	2,574.3	94.5	2,612.8	4,082.9
Annual	76	430.0	105.45	1	7.75E+15	4.69E+15	67,266.8	47,727.6	1,752.0	48,442.4	75,699.3

Notes:
1. For pollutant loadings, Fecal and Entero are in MPN units, while all remaining numbers are in pounds.
2. Results provided are modeled results using 2004 rainfall data.

System Characterization Report Table 8-5: Court Street Typical Year Simulation Results and Wasteloads

	Total (All CSO Events)				Pollutant Loadings						
Month	**Days of Over-flow**	**Duration of Discharge (hours)**	**Vol. (MG)**	**% of Annual Volume**	**Fecal**	**Entero**	**CBOD**	**TKN**	**TP**	**TN**	**TSS**
Jan.	3	14.0	1.97	1.3%	1.41E+14	8.31E+13	1,177.2	190.8	32.2	179.1	2,116.9
Feb.	3	24.0	10.22	6.7%	7.33E+14	4.30E+14	6,096.6	987.9	167.0	927.5	10,963.1
March	7	18.8	1.67	1.1%	1.19E+14	7.01E+13	994.0	161.1	27.2	151.2	1,787.4
April	7	40.5	14.57	9.6%	1.04E+15	6.13E+14	8,691.5	1,408.4	238.1	1,322.3	15,629.2
May	10	27.3	11.11	7.3%	7.96E+14	4.68E+14	6,627.6	1,073.9	181.5	1,008.3	11,917.8
June	7	21.0	9.24	6.1%	6.62E+14	3.89E+14	5,509.9	892.8	150.9	838.2	9,907.9
July	10	57.0	34.84	23.0%	2.50E+15	1.47E+15	20,785.0	3,368.0	569.3	3,162.1	37,375.9
Aug.	7	30.3	11.57	7.6%	8.29E+14	4.87E+14	6,902.7	1,118.5	189.1	1,050.1	12,412.5
Sept.	6	35.3	32.28	21.3%	2.31E+15	1.36E+15	19,258.3	3,120.6	527.5	2,929.8	34,630.6
Oct.	3	10.3	0.96	0.6%	6.91E+13	4.06E+13	575.0	93.2	15.7	87.5	1,033.9
Nov.	7	32.8	13.72	9.1%	9.84E+14	5.78E+14	8,185.3	1,326.3	224.2	1,245.2	14,718.8
Dec.	6	32.3	9.31	6.1%	6.67E+14	3.92E+14	5,554.1	900.0	152.1	845.0	9,987.5
Annual	76	343.3	151.47	100.0%	1.09E+16	6.38E+15	90,357.1	14,641.5	2,475.1	13,746.2	162,481.5

Notes:
1. For pollutant loadings, Fecal and Entero are in MPN units, while all remaining numbers are in pounds.
2. Results provided are modeled results using 2004 rainfall data.

1.2.2 Sensitive Areas Report Summary

The City, as part of the NJ CSO Group, contracted Greeley & Hansen and CDM Smith through the PVSC to perform a detailed sensitive area analysis that included evaluation for the Hackensack River, which is the receiving water body of the City's CSOs. To satisfy the Permit requirements for sensitive areas on behalf of the permittees of the NJ CSO Group, PVSC submitted a separate report detailing the results of the sensitive areas analysis, entitled Identification of Sensitive Areas Report: CSO Long Term Control Plan and dated June 2018.

The sensitive area analysis efforts included letters to regulatory agencies, review of online resources, and observation studies to identify within the regional study area any Outstanding National Resource Waters (ONRW), National Marine Sanctuaries (NMS), threatened or endangered species, primary contact recreation activities, drinking water intakes, and shellfish beds. The results determined that there are no sensitive areas in proximity to the City's combined sewer outfalls. The City has reviewed and certified the sensitive areas report. The NJDEP provided a technical comment letter to the PVSC sensitive areas report on September 10, 2018 and PVSC submitted a revised Identification of Sensitive Areas Report to NJDEP on October 18, 2018 that addressed NJDEP comments. An additional revision of the Sensitive Areas Report

was submitted to the NJDEP on March 29, 2019 to address further comments from the NJDEP in a letter dated March 1, 2019. The Sensitive Areas Report was approved by NJDEP in a letter dated April 8, 2019.

1.3 Approach to Development and Evaluation of Alternative Controls

The United States Environmental Protection Agency (EPA) recommends a three-step approach for the development of alternatives:

- Definition of water quality objectives
- Definition of a range of CSO control goals to meet the CSO component of the water quality goals
- Development of alternatives to meet the CSO control goals

The City's Alternatives Report intends to follow this approach to the development and evaluation of alternative controls.

2 GENERAL INFORMATION

2.1 Public Participation Process

In accordance with the requirements of Part IV.G.2 of the Permit, the City has undertaken a public participation process to inform the affected public about CSOs and the LTCP process and to solicit feedback throughout the process. Details of the public participation process are provided in the City of Hackensack Public Participation Process Report (PPP Report), prepared by Arcadis and revised January 2019, and in the Supplemental Letter Response – Review of Public Participation Process Report Required by Part IV.D.3.b.iii – City of Hackensack, NJPDES Permit No. NJ0108766 (Supplemental Letter), prepared by Arcadis and dated May 21, 2019.

As detailed in the PPP Report and Supplemental Letter, the City established the Hackensack Public Participation Group, an internal team dedicated to planning public outreach efforts related to the LTCP. Outreach efforts conducted to date include the following:

- Creating a page on the City's website dedicated to information about CSOs and the LTCP: hackensack.org/CSO. The webpage includes links to various sites containing information about CSOs, a map of the City's CSS, and the City's CSO handout. The City's approved reports related to the LTCP are publicly available on the NJDEP website. The City's webpage will continue to be updated throughout the remaining phases of the LTCP.
- Disseminating information about the City's CSS in the City's seasonal newsletter and by distributing paper handouts at the City's 4th of July Event and public spaces such as the Department of Public Works, City Hall, the public library, and the Health Department.
- Developing a CSO survey to solicit feedback from the public. The CSO survey was posted on the City's CSO webpage and distributed via mass email to residents of the City.
- Including a presentation about the LTCP in the City Council meeting on June 11, 2019.

Further information about the public outreach efforts can be found in the PPP Report and Supplemental Letter.

The City participates in the Supplemental CSO Group established by the BCUA, along with the hydraulically connected systems of the Village of Ridgefield Park and the Borough of Fort Lee. The City and its representatives attend the recurring Supplemental CSO Group meetings, and the City shares updates about its LTCP progress. Additionally, in accordance with Part IV.G.2.c, the City has invited several residents of the City to participate in the Supplemental CSO Group activities.

2.2 Approximated Future Conditions

Future conditions and their potential impacts need to be considered for a complex planning project such as the City's LTCP. The primary future conditions considered for this LTCP are listed below and are discussed in the following sections:

- population growth within the municipal boundaries of the City
- the City's planned CSS projects

- the City's future dry weather flows for the design year 2050

2.2.1 Projections of Population Growth

As stated in the City's Characterization Report, the current populations of the Anderson and Court subdrainage areas are estimated at 9,000 and 10,500 respectively. The 2018 population estimate for the City of Hackensack is 44,522 (United States Census Bureau). The latest North Jersey Transportation Planning Authority (NJTPA) municipal forecasts predict an annualized population increase of 0.4% through 2045 for the City. This rate was used to estimate the City's 2050 population, as shown in the table below.

Table 2-1. Population Estimates for the City of Hackensack in 2050

Location	2018 Population Estimate	2050 Population Estimate
Anderson Subdrainage Area	9,000	10,376
Court Subdrainage Area	10,500	12,105
Total Combined Sewer Area Population	19,500	22,481
Total Municipal Population	44,522	51,329

2.2.2 Planned Projects

The City is undergoing partial sewer separation projects near Main Street. The partial sewer separation projects prevent the overland stormwater runoff flow from entering the combined sewer system. The partial sewer separation projects do not separate the internal building plumbing stormwater flow that may enter the combined sewer system, such as roof leaders that enter the combined sewer system directly.

The first partial sewer separation project is in the Anderson Street subdrainage area on Passaic Street, where the sewer system collects stormwater from small portions of State Street, Berry Street, and East Camden Street. The project includes a new stormwater outfall to the Hackensack River near the intersection of Passaic Street and River Street. This partial sewer separation project was not incorporated into the City's PCSWMM model prior to calibration. There are additional project plans for a partial sewer separation project on Main Street near Atlantic Street. This project has not been completed to date. However, as additional combined sewer projects are completed, the City intends to incorporate the total impact into its LTCP.

In addition to the minor sewer separation projects near Main Street, the City is currently proceeding with a stormwater management study within the Court Street subdrainage area. This study includes conceptual designs for managing the stormwater runoff in the area bounded east-west by Prospect Street Avenue and Railroad Avenue, and north-south by James Street and Essex Street. This project could have impacts to the selection of Alternative Controls because of the potential decrease of stormwater flow to the City's CSS. However, this study is currently in a preliminary phase, so the study is not included in the modelling and alternatives for this Alternatives Report. If there is a planned project at the conclusion of this study, it will be incorporated into the City's LTCP.

2.2.3 Projected Future Wastewater Flow

The City's dry weather flows for 2006, as presented in the Characterization Report, are presented in the table below. Projected flows for the year 2050 were calculated using the NJTPA population projections and a flow factor of 75 gallons per capita daily (GPCD), per the methodology stated in N.J.A.C. 7:15 for urbanized municipalities. It was assumed that there would be no significant increases in commercial or industrial sewage generation. The projected flows are presented in the table below.

Table 2-2. Projected Dry Weather Flows for the City's CSS in 2050

	2006 Dry Weather Flow, MGD	Projected Increase in Flow, MGD	2050 Dry Weather Flow, MGD
Anderson Subdrainage Area	1.82	0.10	1.92
Court Subdrainage Area	4.70	0.12	4.82

3 WATER QUALITY OBJECTIVES

The Permit allows the City to evaluate the ability of various alternatives to meet the applicable water quality standards using the "Presumption" or "Demonstration" approach. The objective of these approaches is to provide an adequate level of control to meet the water quality-based requirements of the Clean Water Act. The applicable water quality standards and these approaches for compliance are described later in this section.

The EPA CSO Control Policy recommends flexibility in allowing a municipality to select controls that are cost-effective and tailored to local conditions. For this reason, the choice between the demonstration approach and presumption approach does not necessarily have to be made before a municipality commences work on its LTCP. In some cases, it might be prudent for a municipality to assess alternatives under both approaches.

3.1 Applicable Water Quality Standards

As presented in Table 1-1, on behalf of the NJ CSO Group, PVSC submitted a Baseline Compliance Monitoring Program (BCMP) Report dated June 30, 2018, as per Part IV Section G.9 of the Permit. The NJDEP provided a technical comment letter to the PVSC BCMP Report on September 7, 2018 and PVSC submitted a revised BCMP Report to NJDEP on October 5, 2018 that addressed NJDEP comments. The BCMP Report was approved by NJDEP on March 1, 2019.

The BCMP Report outlines the applicable water quality standards for the Hackensack River at the City's CSOs. "Attachment 2 – Receiving Water Quality Data" in the BCMP Report states that the water quality standards in the Hackensack River at the City's location are Saline Estuary 1 (SE1) and presents the data from the sampling performed. The sampling data found that the Hackensack River is typically within the water quality standards during dry weather. However, there were multiple exceedances of the standards during the dry weather sampling that do not appear to be CSO driven.

3.2 Existing CSO Control Alternatives

In accordance with Part IV.F.6 of the Permit, "Control of Solids/Floatables in CSOs" (one of the Nine Minimum Control Requirements), the City has CSO regulators and screening facilities at both the Anderson Street (001A) and Court Street (002A) combined sewer outfalls. At each location, during dry weather, sewage enters a diversion chamber, is diverted through a pipe, goes to a vortex chamber, and is routed through a pipe into the BCUA trunk sewer. The vortex valve (also referred to as the regulator) is designed to restrict flow and to allow a limited flow to enter the BCUA trunk sewer.

During wet weather events, the flow process is the same as that in dry weather until the maximum allowable flow through the vortex valve is reached. At that point, sewage depth begins to accumulate upstream of the vortex valve into the diversion chamber. Once the water depth in the diversion chamber exceeds the height of the diversion wall, which functions similarly to a weir, flow enters the screening facility, where solids and floatables are removed. Screened flow then enters the outfall pipe that leads to a concrete structure (the tide gate chamber) containing a tide gate. The structure includes wing walls, a concrete apron, riprap, and bituminous paving in order to stabilize the outfall at the Hackensack River. The screened overflow is discharged into the Hackensack River. Additionally, a small quantity of water is used to remove solids and

floatables in the screening facility and is then routed to a foul sewer, which flows directly into the BCUA trunk sewer.

The City's approved Characterization Report provides additional details and diagrams on the CSO regulators and screening facilities at both the Anderson Street and Court Street outfalls.

3.3 Range of CSO Goals Being Evaluated

The alternatives were evaluated for a system-wide total of 4, 8, 12 and 20 overflows per year, as well as elimination or the capture for treatment of no less than 85% by volume of the combined sewage collected in the CSS during precipitation events on a hydraulically connected system-wide annual average basis for the 2004 typical year.

The number of overflows during the 2004 typical year are counted on a hydraulically connected system-wide annual average basis. The City and the Village of Ridgefield Park share the Hackensack River as a CSO receiving water body. Therefore, the number of overflows between the City and the Village of Ridgefield Park during the 2004 typical year are considered one event if the overflows occur during the same 24-hour period. The results within this Alternatives Report present the number of overflows for the City of Hackensack.

Similarly, the elimination or the capture for treatment of no less than 85% by volume of the combined sewage collected in the CSS during precipitation events is on a hydraulically connected system-wide annual average basis. The percent capture calculation methodology described on Page 11 of the PVSC Evaluation of Alternatives Process Memo dated January 7, 2019 was utilized for the percent capture calculations presented herein.

3.3.1 Presumption Approach

Regarding the Presumption approach, the Permit states:

> "The permittee must demonstrate each of the following below:

- No more than an average of four overflow events (see below) per year from a hydraulically connected system as the result of a precipitation event that does not receive the minimum treatment specified below. The Department may allow up to two additional overflow events per year. For the purpose of this criterion, an 'event' is:
    - In a hydraulically connected system that contains only one CSO outfall, multiple periods of overflow are considered one overflow event if the time between periods of overflow is no more than 24 hours.
    - In a hydraulically connected system that contains more than one CSO outfall, multiple periods of overflow from one or more outfalls are considered one overflow event if the time between periods of overflow is no more than 24 hours without a discharge from any outfall.
- The elimination or the capture for treatment of no less than 85% by volume of the combined sewage collected in the CSS during precipitation events on a hydraulically connected system-wide annual average basis.

- The elimination or removal of no less than the mass of the pollutants, identified as causing water quality impairment through the sewer system characterization, monitoring, and modeling effort, for the volumes that would be eliminated or captured for treatment under Section G.4.f.ii."

3.3.2 Demonstration Approach

Regarding the Demonstration approach, the Permit states:

> "A program that meets any of the criteria listed below will be presumed to provide an adequate level of control to meet the water quality-based requirements of the CWA, provided the Department determines that such presumption is reasonable in light of the data and analysis conducted in the characterization, monitoring, and modeling of the system and the consideration of sensitive areas described above.
>
> A permittee may demonstrate that a selected control program, though not meeting the criteria specified under the Presumption Approach above, is adequate to meet the water quality-based requirements of the CWA.
>
> The permittee must demonstrate each of the following below:

- The planned control program is adequate to meet WQS and protect designated uses, unless WQS or uses cannot be met as a result of natural background conditions or pollution sources other than CSOs.
- The CSO discharges remaining after implementation of the planned control program will not preclude the attainment of WQS or the receiving waters' designated uses or contribute to their impairment.
- The planned control program will provide the maximum pollution reduction benefits reasonably attainable.
- The planned control program is designed to allow cost effective expansion or cost-effective retrofitting if additional controls are subsequently determined to be necessary to meet WQS or designated uses."

4 DEVELOPMENT OF ALTERNATIVES

Part IV.G.4.e of the Permit lists seven control technology alternatives that require evaluation and are considered general control approaches. Accordingly, the following seven alternatives are discussed and evaluated in this section of the Alternatives Report:

- Green infrastructure.
- Increased storage capacity in the collection system.
- Sewage treatment plant (STP) expansion and/or storage at the plant (an evaluation of the capacity of the unit processes must be conducted at the STP resulting in a determination of whether there is any additional treatment and conveyance capacity within the STP). Based upon this information, the permittee shall determine (modeling may be used) the amount of CSO discharge reduction that would be achieved by utilizing this additional treatment capacity while maintaining compliance with all permit limits.
- Inflow and infiltration (I/I) reduction to meet the definition of non-excessive infiltration and non-excessive inflow as defined in N.J.A.C. 7:14A-1.2 in the entire collection system that conveys flows to the treatment works to free up storage capacity or conveyance in the sewer system and/or treatment capacity at the STP, and feasibility of implementing in the entire system or portions thereof.
- Sewer separation.
- Treatment of the CSO discharge.
- CSO related bypass of the secondary treatment portion of the STP in accordance with N.J.A.C. 7:14A-11.12 Appendix C, II C.7.

These alternatives were compared to the City's existing CSO facilities and water quality goals to evaluate if the alternatives can be accomplished within the facilities under the permittee's control to meet or help meet the goals.

4.1 Development and Screening of CSO Control Approaches

This section provides background and descriptions of the CSO control technologies listed above that were considered for the City's CSS service area. This section introduces the unique conditions of the City's CSS that were assessed to identify the effectiveness of the alternatives considered. If an alternative was deemed to be potentially effective for CSO control, it was carried forward to the next tier in the evaluation discussed later in this Alternatives Report; an alternative deemed to be ineffective was screened out and is not further discussed in this Alternatives Report.

4.1.1 Green Infrastructure

As stated in the "Green Infrastructure Feasibility Study for Hackensack" (Rutgers Report) prepared by the Rutgers Cooperative Extension Water Resources Program:

> "Green infrastructure is an approach to stormwater management that is cost-effective, sustainable, and environmentally friendly. Green infrastructure projects capture, filter,

> absorb, and reuse stormwater to maintain or mimic natural systems and to treat runoff as a resource. As a general principle, green infrastructure practices use soil and vegetation to recycle stormwater runoff through infiltration and evapotranspiration. When used as components of a stormwater management system, green infrastructure practices such as bioretention, green roofs, porous pavement, rain gardens, and vegetated swales can produce a variety of environmental benefits. In addition to effectively retaining and infiltrating rainfall, these technologies can simultaneously help filter air pollutants, reduce energy demands, mitigate urban heat islands, and sequester carbon while also providing communities with aesthetic and natural resource benefits (USEPA, 2013)."

A variety of factors were considered to evaluate the implementation of green infrastructure in the City of Hackensack. The selected green infrastructure technology will need to be both visually appealing and effective at retaining at least 1-inch of rain water from the designated treatment area. The green infrastructure technologies that were initially evaluated were roadside rain gardens/bioswales and permeable pavement. These technologies can be effective for both stormwater quantity control and stormwater quality control.

4.1.1.1 Bio Retention

As stated in the Rutgers Report:

> "A rain garden/bioswale, or bioretention system, is a landscaped, shallow depression that captures, filters, and infiltrates stormwater runoff. The rain garden removes nonpoint source pollutants from stormwater runoff while recharging groundwater. A rain garden serves as a functional system to capture, filter, and infiltrate stormwater runoff at the source while being aesthetically pleasing. Rain gardens are an important tool for communities and neighborhoods to create diverse, attractive landscapes while protecting the health of the natural environment. Rain gardens can also be installed in areas that do not infiltrate by incorporating an underdrain system.
>
> Rain gardens can be implemented throughout communities to begin the process of re-establishing the natural function of the land. Rain gardens offer one of the quickest and easiest methods to reduce runoff and help protect water resources. Beyond the aesthetic and ecological benefits, rain gardens encourage environmental stewardship and community pride."

Roadside rain gardens/bioswales are being implemented at large scales in cities such as New York City and Philadelphia. Given the design flexibility and the positive results in other cities, roadside rain gardens/bioswales were chosen for further evaluation as a green infrastructure technology alternative.

4.1.1.2 Permeable Paving

As stated in the Rutgers Report:

> "Permeable paving includes pervious concrete, porous asphalt, interlocking concrete pavers, and grid pavers. Pervious concrete and porous asphalt are the most common of the permeable surfaces. They are similar to regular concrete and asphalt but without the fine materials. This allows water to quickly pass through the material into an underlying

layered system of stone that holds the water, allowing it to infiltrate into the underlying uncompacted soil. They have an underlying stone layer to store stormwater runoff and allow it to slowly seep into the ground.

By installing an underdrain system, these systems can be used in areas where infiltration is limited. The permeable pavement system will still filter pollutants and provide storage but will not infiltrate the runoff."

4.1.2 Increased Storage Capacity

The objective of a storage alternative is to reduce overflows by capturing and storing wet weather flows within the system. Once the wet weather event subsides and increased capacity becomes available in the CSS and WPCF, the captured combined sewage will be conveyed to the WPCF. A storage facility is sized to handle a certain quantity of flow. If a storm exceeds the design capacity of the storage system, the first flush, or the most hazardous combined sewage, will be captured and the remaining portion, which would be primarily stormwater, will overflow to the receiving waterbody. Storage technologies typically have high construction and operations and maintenance (O&M) costs compared to other CSO control technologies, but they are a very reliable means of achieving CSO control goals. Storage tanks, deep tunnels, and increased in-line storage, which are various types of storage technologies, were evaluated for the City.

4.1.2.1 Satellite Storage Tanks

Satellite storage tanks are large storage facilities installed in proximity to existing outfalls to store wet weather flows until capacity becomes available in the BCUA trunk sewer and at the WPCF. The storage tanks typically are covered, underground structures that include odor control facilities. A dewatering pump at each tank conveys the combined sewage through a force main back to the existing BCUA trunk sewer after each wet weather event. To prevent flooding of upstream systems, the storage tanks are equipped with an overflow to discharge combined sewage to the receiving water body if the captured volume of combined sewage exceeds the available storage in the tank.

The use of storage tanks, sized to allow a targeted number of overflows per year, can effectively limit the quantity and frequency of CSOs. This technology can be implemented incrementally, one tank at a time, with prioritization for construction of a storage tank in an area with more significant water quality concerns or flooding issues. Drawbacks of this technology include the relatively large land area requirements, high construction and O&M costs, and potential odor issues.

The April 2007 Cost and Performance Analysis Report (2007 Report) that was prepared for the City by Malcolm Pirnie, Inc. (now Arcadis) evaluated two storage tank scenarios. The first involved two storage tanks: one upstream of the Anderson Street outfall and one upstream of the Court Street outfall. The second involved one regional storage tank for the City's CSS located near the Court Street outfall, which would require diversion of the flow from the Anderson Street subdrainage area to the regional storage tank. The regional storage tank alternative would eliminate the need for the Anderson Street outfall and utilize the Court Street outfall as the City's only outfall. Note that the regional storage tank alternative presented in this Alternatives Report would store combined sewage only from the two subdrainage areas of the City's CSS; the regional storage tank alternative presented does not refer to a regional tank for the hydraulically connected communities of the BCUA, Ridgefield Park, and Fort Lee.

4.1.2.2 Storage Tunnel

A tunnel was evaluated as a storage alternative in the City. Tunnels are advantageous because they do not take up valuable aboveground area in the City, where City-owned land is not always accessible. The tunnel would be bored about 100 feet below ground so it would not disturb any existing infrastructure or utilities. The tunnel would be connected to the Anderson Street and Court Street outfalls by drop-down shafts. Tunnels usually have a high overall cost, but their cost per million gallons of storage is reasonable compared to other storage technologies. It was determined that a tunnel alternative was worth further evaluation due to the relatively low cost per unit storage and minimal conflicts with existing infrastructure.

4.1.2.3 Maximize Existing Storage in the Collection System

In-line storage takes advantage of storage within the existing CSS collection system. The City's CSS occasionally surcharges during certain wet weather events. The City has flood-prone areas due to the limited existing capacity of the existing CSS collection system. The City's PCSWMM model supports that the CSS collection sewers surcharge during wet weather events. The flood sensitive areas are shown in Figure 3-1B of the City's Characterization Report. If the City adjusted its regulators to take advantage of additional CSS in-line storage, street flooding would increase during wet weather events. Therefore, due to the City's limited CSS collection system capacity and the existence of flood-prone areas, this alternative was not further considered.

4.1.3 Sewage Treatment Plant (STP) Expansion

The BCUA is undertaking a study regarding STP expansion. Preliminary investigations show that the trunk sewer and STP are near maximum capacity. CSOs potentially can be reduced by increasing the treatment capacity of the plant. The plant expansion would allow a larger portion of wet weather flows to be directed to the treatment plant instead of being discharged to receiving waterbodies. Increasing the portion of flows that is directed to the treatment plant cannot entirely achieve CSO abatement controls because the existing trunk sewers cannot convey enough wet weather flows to the BCUA to achieve 85% capture or to minimize the amount of CSOs. However, if it is determined that additional wet weather flow can be conveyed to the BCUA trunk sewer by the City, this may reduce the size of other technologies that are being evaluated by the City. This alternative may remain on the alternatives short list to explore further the cost saving impact during the final selection process. Prior to further evaluation, this alternative must be approved by the BCUA. Therefore, this alternative is not further evaluated in this Alternatives Report.

4.1.4 Inflow and Infiltration (I/I) Reduction

Excessive I/I can consume the hydraulic capacity of a collection system and increase overall O&M costs. Inflow comes from sources such as roof drains, manhole covers, cross connections from storm sewers, catch basins, and surface runoff, which enter the CSS by design. Within a CSS, surface drainage is the primary source of inflow. Infiltration refers to groundwater that seeps into the CSS through leaking pipe joints, cracked pipes, manholes, and other similar sources. The flow from infiltration tends to be constant, but at a lower volume than that of inflow.

Identifying I/I sources is labor intensive and requires specialized equipment. Significant I/I reductions can be difficult and expensive to achieve. I/I reduction for combined sewers provides limited gains, since water

tends to find another way into the system. However, the benefit of an I/I control program is that it can save money by extending the life of the system, reducing the need for expansion, and lowering pumping and treatment costs.

As mentioned in the City's approved Characterization Report, a condition assessment was performed in 2015. The condition assessment included observations of potential I/I issues. This information allowed for a more detailed I/I analysis as part of this alternatives screening.

4.1.5 Sewer Separation

Sewer separation refers to conversion of the CSS into separate stormwater and sanitary systems. This can involve construction of a new stormwater conveyance system and utilization of the existing CSS for sanitary only, or vice versa. Sewer separation can eliminate or significantly reduce the occurrence of combined sewage back-ups into streets or basements. In a complete sewer separation scenario, sanitary flows would be conveyed to the treatment plant during both wet weather and dry weather, and stormwater flows during wet weather would discharge directly to receiving waterbodies. Complete sewer separation meets water quality goals by significantly reducing the quantities of fecal coliform and other bacteria that enter receiving waters; complete sewer separation is considered the only technology that can achieve zero combined sewer overflows. However, complete sewer separation is costly and disruptive to the public, especially in highly dense urban areas. It is estimated that City-wide sewer separation could cost upwards of $555 million, as shown in Table 4-1. Other CSO control technologies are more cost effective for the City; therefore, the alternative of complete sewer separation was not further evaluated for the City.

Table 4-1. Capital, O&M, and Present Worth Cost Breakdown for System-wide Sewer Separation

Sewer Separation	Present Worth Cost			
	Total Capital Cost	Land Cost	Annual O&M Cost	Present Worth
System Wide	$ 470,350,000	NA	$ 5,570,000	$ 555,230,000
			Total Present Worth Cost	**$ 555,230,000**

Partial sewer separation in critical areas that are susceptible to flooding can be beneficial and cost effective. The City has partially separated sewers in a few small-scale locations, such as those mentioned in Section 2.2.2 of this Alternatives Report.

4.1.6 Treatment of CSO Discharge

The City evaluated several treatment options for CSO control. The evaluation considered the following abatement technologies, all of which are discussed in greater lengths in the PVSC CSO Long Term Control Plan Updated Technical Guidance Manual, January 2018 (TGM 2018):

- Screenings
  - Netting systems
  - Mechanical Bar Screens
  - Fine Screens

  - Band and Belt Screens
  - Drum Screens
- Pretreatment Technologies
  - Vortex/Swirl Separation Technology
  - Ballasted Flocculation
  - Compressible Media Filtration Process
- Disinfection
  - Chlorine Dioxide
  - Sodium Hypochlorite
  - Peracetic Acid (PAA) Disinfection
  - UV Disinfection

As discussed in Section 3.2, screening facilities already are in place in the City.

It was determined that both a disinfection alternative and a disinfection with pretreatment alternative would be evaluated. Disinfection typically is performed on a total suspended solids (TSS) reduced stream following screening and pretreatment. The effectiveness of the disinfection alternative relies on the TSS concentration of the sewage.

The disinfection chemicals considered were sodium hypochlorite and peracetic acid (PAA). Chlorine dioxide was excluded from further evaluation as it has many drawbacks, including safety issues during transport and storage, stability, and production of toxic byproducts. Sodium hypochlorite is more widely used in practice than PAA, but the use of sodium hypochlorite typically requires the addition of sodium bisulfite for dechlorination, which raises O&M costs. PAA will be evaluated because it has a stronger oxidation potential than chlorine dioxide. PAA is non-toxic and does not produce disinfection byproducts during disinfection. Through pilot studies it has been determined that the effectiveness of PAA disinfection is not inhibited by TSS, NH_3, chemical oxygen demand (COD), dissolved oxygen (DO), or pH. Due to the effectiveness of PAA disinfection in a variety of conditions, it was selected as the primary disinfectant in the disinfection only alternative.

The second treatment alternative evaluated was a combination of PAA disinfection with upstream pretreatment. Using information provided in the TGM 2018, the SanSep treatment unit was chosen for evaluation in Hackensack. SanSep has a simple design with no moving parts. The technology is effective at removing TSS at a variety of loading rates. The pretreatment alternative being evaluated in Hackensack would include two groups of SanSep units, one at each outfall, upstream of PAA disinfection.

Disinfection alone and pretreatment in combination with disinfection would treat the CSO discharge without reducing the number of overflows. The extent of pretreatment that may be required to meet future water quality standards is unknown. Despite the uncertainty associated with the level of pretreatment required, the treatment alternative is further evaluated in this Alternatives Report because it is cost competitive with other alternatives.

4.1.7 CSO Related Bypass at Secondary Treatment

The BCUA is undertaking a study regarding CSO-related bypass at secondary treatment. Preliminary investigations show that the BCUA trunk sewer and STP are near maximum capacity. If it is determined that additional wet weather flow can be conveyed to the BCUA trunk sewer by the City, this may reduce the size of other technologies that are being evaluated by the City. This alternative may remain on the alternatives short list to explore further the cost saving impact during the final selection process. Prior to further evaluation, this alternative must be approved by the BCUA. Therefore, this alternative is not further evaluated in this Alternatives Report.

4.2 Summary of Alternatives Screening

Table 4-2 presents a summary of the alternatives that were screened in Section 4 and indicates whether they will be further evaluated or considered in this Alternatives Report.

Table 4-2. Summary of Alternatives Screening

Alternative Screened		Further Evaluation?	Reason
1	Green infrastructure	Yes	High public acceptance; potentially reduces street flooding in localized scenarios; typically, aesthetically appealing; flexible designs; positive results in other cities; lower capital cost for installation compared to other alternatives.
2a	Increased storage capacity - Satellite storage tank(s)	Yes	Effectively limits the quantity and frequency of CSOs; potentially reduces street flooding and water quality issues; cost effective compared to other alternatives.
2b	Increased storage capacity - Storage tunnel	Yes	Effectively limits the quantity and frequency of CSOs; potentially reduces street flooding and water quality issues; minimal disturbance to existing infrastructure or utilities; limited aboveground land required; cost is reasonable compared to other alternatives.
2c	Increased storage capacity - Maximize storage in the collection system	No	No additional capacity available in the City's CSS.
3	STP expansion and/or storage at the plant	No	City cannot expand or increase storage at the STP; BCUA is undertaking a study regarding STP expansion; preliminary investigations show that the trunk sewer and STP are near maximum capacity.
4	I/I reduction	Yes	Saves money by extending the life of the City's CSS; reduces the need for expansion, lowers treatment costs for I/I flow that is conveyed to BCUA.
5	Sewer separation (system-wide)	No	Significant capital cost to accomplish City-wide.
6	Treatment of the CSO discharge	Yes	Dependent on extent of pretreatment required; potentially low capital costs for installation compared to other alternatives.
7	CSO related bypass of secondary treatment at STP	No	City cannot determine if bypass of secondary treatment at STP is possible; BCUA is undertaking a study regarding CSO related bypass at secondary treatment; preliminary investigations show that the trunk sewer and STP are near maximum capacity.

5 COSTING

The cost and performance analysis presented in Section 7.5.3 of this Alternatives Report was prepared in accordance with the guidelines detailed in PVSC's TGM 2018. All present worth costs include the present-day costs for capital costs, land costs, and O&M costs assuming an interest rate of 2.75% and a 20-year period or life of the project. All capital costs include an additional 25% for contingencies, 20% for engineering costs and 15% for the contractor overhead and profit. All costs have been adjusted for present day worth using the Engineering News Record (ENR) construction cost indices (CCI). The cost estimates in this Alternatives Report are Class 5 Estimates, which have a range of -50 to +100% of the actual cost.

Several assumptions specific to the City of Hackensack were used during evaluation of control alternatives. These assumptions include the following:

- Land cost in the City was estimated to be $2,930,000 per acre based on the averages of several real estate property quotes in the City.
- The amount of land required for all treatment and disinfection systems is equal to the sum of the land required for the tanks, equipment, and buildings plus a twenty-five-foot buffer around the area for access and maintenance.

6 AVAILABLE LAND ANALYSIS

As an urban community, Hackensack is nearly fully developed and has very limited vacant land. Many of the alternatives that will ultimately be implemented to address CSOs will be built on publicly owned land or land obtained from private property owners. Construction of new CSS infrastructure is limited by the location of the existing outfalls at Anderson Street and Court Street. Moving the outfalls or extending the connection to the outfalls is not favored or recommended due to the associated cost and complexity. Johnson Park, which is owned by the City, provides available public space near the Anderson Street outfall. There is limited public land available near the Court Street outfall; therefore, it may be necessary to acquire private land for some alternatives such as off-line storage.

7 ALTERNATIVES EVALUATION

7.1 Introduction

This section of the Alternatives Report presents the detailed evaluation of CSO control technologies identified in Section 4 that are under consideration for the City's CSS. The factors of siting, institutional issues, public acceptance, performance, implementability, and cost (implementability is discussed in Section 7.5.1 and cost is discussed in Section 7.5.3) are considered for each alternative. If modeling was applicable for the evaluation of an alternative, the City's calibrated PCSWMM model was utilized to further evaluate.

7.2 Presumption and Demonstration Approaches

Section 3 of this Alternatives Report describes that the water quality standards for the Hackensack River are occasionally exceeded during dry weather. Due to these water quality exceedances, it will be difficult for the City to undertake the demonstration approach for its LTCP. The alternatives were evaluated with the presumption approach CSO control goals. However, as stated in Section 3, it might be prudent for the City to assess alternatives under both approaches before selecting the final approach to be used in its LTCP.

7.3 Development and Evaluation of Alternatives

7.3.1 Single or Group Overflows

The City's two outfalls are approximately one mile apart. Due to the locations of the City's outfalls, the satellite storage tank and treatment facility alternative controls would be implemented at each overflow point. However, a single regional storage tank alternative near the Court Street outfall was evaluated. The storage tunnel alternative would be a regional solution for both outfalls.

7.3.2 Siting Issues

The following sections present the methodology used to determine the siting considerations for each of the alternatives. Section 6 in this Alternatives Report provides an overview of the available land in the City. As described, the alternatives range from being sited entirely within the public right-of-way of roads to requiring construction in public spaces or easements on private properties. As summarized in Table 7-1, the alternatives were rated relative to each other in the factor of siting. In general, alternatives better suited for siting in the public right-of-way or on publicly owned lands received higher ratings because this eliminates the need for land acquisition or easements. Additionally, alternatives that could be sited to minimize disturbance to the public received higher ratings.

7.3.2.1 Siting Issues for Green Infrastructure

The areas for green infrastructure are based on the Rutgers Report, in which a collection of sites in the City has been identified based on site visibility, feasibility, cost-effectiveness, and potential partnerships. The

Table 7-1 Alternatives Evaluation Matrix												
Alternatives	Siting	Institutional Issues	Public Acceptance			Performance	Implementability				Cost	Overall Score
			Environmental Impacts	Social Benefits	Multiple-use Considerations		Constructability	Reliability	Operability	Adaptability		
Green Infrastructure	3	5	4	5	4	2	5	3	3	5	4	43
I/I Reduction (replacement and rehabilitation)	5	4	3	1	1	1	4	4	5	5	5	38
Collection System and Source Controls	4	4	3	2	3	2	4	4	4	4	3	37
Off-line Storage with Storage Tanks	1	1	4	3	5	5	3	5	2	4	2	35
Off-line Storage with Tunnels	3	1	4	3	3	5	2	5	2	2	3	33
Off-line Storage with Regional Storage Tank	2	1	4	3	5	5	1	5	2	2	2	32
Treatment of CSO Discharge	3	2	2	1	1	3	4	4	2	3	4	29

1 = Poor, 2 = Fair, 3 = Good, 4 = Very good, 5 = Excellent
- See Section 7.3.2 for discussion of siting.
- See Section 7.3.3 for discussion of institutional issues.
- See Section 7.3.4 for discussion of public accceptance.
- See Section 7.3.5 for discussion of performance.
- See Section 7.5.1 for discussion of implementability.
- See Section 7.5.3 for discussion of cost.

Rutgers Cooperative Extension Water Resources Program uses a “look here first” method to identify the most accessible and visible sites. These sites include schools, churches, libraries, municipal buildings, public works, firehouses, post offices, social clubs such as the Elks or Moose lodge, and parks/recreational fields. These sites often have large amounts of impervious cover and typically are relatively easy to engage in implementing green infrastructure practices. Sites are selected based on their feasibility or the ability to get the project in the ground. These criteria are based on property ownership and ability to do maintenance. In addition, potential partnerships related to the site help make a project feasible.

Roadside rain gardens/bioswales are installed in sidewalks along roadways with curb cut-outs to allow street runoff to enter the rain gardens and for excess water to exit when the rain gardens are at capacity. Rain gardens typically are four to five feet wide and ten to twenty feet long. They can be placed on sidewalks if they do not interfere with utilities or the pedestrian right of way. Rain gardens are most effective in areas with at least ten feet of depth to bedrock or ground water.

The green infrastructure areas identified in the Rutgers Report treat 34 acres of the City and comprise 5% of the City’s impervious area; implementation of green infrastructure at these sites was evaluated for this Alternatives Report. Figure 7-1 shows the green infrastructure locations selected. A second green infrastructure alternative was considered in which an additional 5% of impervious area would be managed by roadside rain gardens/bioswales to achieve management of 10% of the City’s impervious area.

7.3.2.2 Siting Issues for Satellite Storage Tank(s)

As discussed in Section 4.1.2.1, satellite, or off-line, storage tanks for CSO control should be placed in proximity to existing outfalls. To maximize use of the City’s limited real estate, only below ground storage options are considered in this Alternative Report. Several additional factors were taken into consideration for selection of sites for storage tanks and new gravity sewers, including land ownership and current land use. Publicly owned land or public spaces were considered preferable locations for construction of storage tanks.

The preliminary locations for the two storage tanks and for the single regional storage tank are shown in Figures 7-2 and 7-3, respectively. As shown in Figure 7-2, the separate storage tanks alternative would include one storage tank located upstream of the Anderson Street outfall in Johnson Park, and one storage tank located upstream of the Court Street outfall in the parking lot of Costco. Johnson Park (Block 404 Lot 1) is owned by the City, so no land easements would be needed, and the area is currently an open, grassy space. Construction of a storage tank in the park would require temporary disturbance and extensive permitting, but since storage would be constructed below ground, the park would be restored after construction. An easement would need to be obtained from Costco to allow for construction of the Court Street storage tank in the parking lot (Block 61 Lot 1). As shown in Figure 7-3, the regional storage tank alternative would include only the storage tank in the Costco parking lot.

7.3.2.3 Siting Issues for Storage Tunnel

The storage tunnel alternative consists of an approximately 5,530-foot tunnel that runs parallel to the Hackensack River. The tunnel would be bored 100 feet deep into bedrock and would intersect both the Anderson Street outfall and the Court Street outfall. Due to the depth of the tunnel, there would be minimal utility conflicts or need for utility relocation. The tunnel would be dewatered and pumped back to

BCUA by a pumping facility located near the existing Court Street screening facility. This pumping facility can be built at the location of the existing facility or in the corner of the Costco parking lot described in Section 7.3.2.2. The tunnel would consist of two drop-shafts at the existing outfalls and three access shafts for bends in the tunnel path that are too extreme for the drill to attain. Each drop-shaft or access shaft should require approximately 3,200 square feet of above ground area to construct. Each drop-shaft or access shaft can be built in open available space such as parks or parking lots. There is availability of open space along the projected tunnel path, as shown in Figure 7-4.

7.3.2.4 Siting Issues for I/I Reduction

During the City's 2015 CSS condition assessment project, a total of 851 pipe segments were inspected to identify defects and to classify the type of defect according to Pipeline Assessment Certification Program (PACP) standards (stain, weeper, dripper, runner, and gusher). The results of these inspections determined that 67 manholes and 102 pipe segments, representing a total length of approximately 16,800 ft, should be replaced or rehabilitated to decrease I/I in the City's CSS collection system. Figure 7-5 presents the pipe segments and manholes that were recommended for replacement or rehabilitation (those segments and manholes are highlighted in green) including a table of the estimated I/I removed from each subcatchment in the model. There are minimal siting issues with this alternative because the replacement or rehabilitation of the pipe segments and manholes would occur in place.

7.3.2.5 Siting Issues for Treatment of CSO Discharge

CSO treatment facilities would ideally be placed on publicly owned land or public spaces. The preliminary locations for CSO treatment facilities would mimic the sites chosen for the two storage tanks shown in Figure 7-2. The Anderson Street treatment facility can be placed below ground in the publicly owned Johnson Park and the Court Street treatment facility can be placed below the Costco parking lot, where an easement would be needed. These sites can be utilized for both the disinfection only and the disinfection and pretreatment alternatives.

7.3.3 Institutional Issues

Institutional issues are factors and influences from various organizational, social, community, or other special interest groups that may have significant impacts on the success or failure of a given project. Proposed CSO abatement projects in the City may meet resistance if various institutional issues are not adequately addressed in advance to meet the given institution's needs or desires. Sometimes giving more to an institution's requests can aid in achieving overall project approval even though the portion of the project that contributes to the given institution's cause may only represent a small portion of the overall project. In the City, the relevant institutional issues and their potential impacts to the alternatives that have been selected for further evaluation are as follows:

- Real estate: Since the City has a successful real estate market this may make alternatives that require land, such as tunnels with access shafts and/or storage tanks, less favorable to those real estate institutions. Alternatives such as collection system controls, sewer separation, and green infrastructure would be in easements or public rights-of-way, which is neutral territory for most real estate institutions. Developers are a real estate institution that can have significant impact on a project. Green infrastructure would seem to be more favorable and can enhance real estate values in certain areas.

- Location equity: The tunnel and storage tank alternatives will generally be located near the Hackensack River waterfront because that is the final discharge location of the CSO outfalls. The area surrounding the Anderson Street outfall has a nearby park where there may be more resistance to these types of structures. Collection system and sewer separation projects may meet temporary discord during construction; however, since they are in the public right-of-way, the impacts would be minor. Green infrastructure likely would be more favorable and pleasing to the public regardless of location. Care should be taken to ensure that implementation of the CSO control technologies is fairly distributed across groups of varying socioeconomic status.
- Government institutions: Parks and NJ Transit are government institutions that will have to be managed depending on the alternative.
- Special Interest Groups: Multiple members from NJ Future and the Hackensack Riverkeeper have attended at least one Supplemental CSO Team meeting held at the BCUA. They have stated that they want more green infrastructure including, but not limited to, bioswales, rain gardens, trees, and rain barrels or cisterns.
- Utility rate payers: Since this LTCP will require rate increases to cover the costs, most City rate payers have an interest in the selection and implementation of alternatives. With respect to the implementation of alternatives to address CSOs, it is important to provide technically sound and cost-effective solutions to mitigate impacts to the rates.

Based upon the facts or probable outcomes described above, the alternatives were rated relative to each other in the factor of institutional issues, as summarized in Table 7-1. Overall ratings for institutional issues are as follows:

- Green infrastructure should rank highest regarding institutional issues.
- Rehabilitation and replacement of pipes for removal of I/I would have a very good rating for institutional issues, since the public outreach to communicate the age of the City's sewers has been significant, and because the flood problems also are well known in the selected areas.
- Disinfection will have a fair rating for institutional issues as it will take up land near the outfalls and continue to discharge overflows to the Hackensack River.
- The separate or regional storage tanks and tunnels would have the greatest difficulty with institutional issues, so they have poor ratings.

7.3.4 Public Acceptance

The City will incorporate public feedback throughout the LTCP process to ensure public acceptance of the selected alternatives. As described in Section 2.1 of this Alternatives Report and in the City's PPP Report, the City will solicit public feedback through various means, including Supplemental CSO Team meetings, public meetings held by the City, and public surveys. In addition to taking the comments of the public into consideration, several factors related to public acceptance, which were selected based on the EPA CSO Guidance for Long-Term Control Plan document, were evaluated to determine which alternatives would be most beneficial to the public. The factors evaluated were environmental impact, social benefit, and multiple-use considerations. These factors are defined below and are discussed for each alternative in the following

sections. As summarized in Table 7-1, the alternatives were rated relative to each other for each of these factors.

- Environmental Impact – Environmental impact includes impacts on nature and people. Effects on nature include water quality, threats to endangered species, wetlands impacts, soil erosion, flooding, and other forms of habitat destruction. Effects on people and residents include noise, traffic, and utilities relocation. Alternatives that involve minimal disruption to the lives of residents and nature are more beneficial. An alternative that is very disruptive is less beneficial.
- Social Benefit – An alternative that adds positive aspects to the lives of City residents would be viewed positively by City residents. An alternative that adds to the physical and or mental well-being of the residents would be more beneficial, and an alternative that has no benefit to the physical or mental well-being of the residents would be less beneficial.
- Multi-use Considerations – An alternative that serves a use to the public would be beneficial in gaining support for its implementation. If an alternative can be designed to include a park or walking path, it would have multiple beneficial uses, and would be more beneficial than an alternative that only operates as a CSO control.

7.3.4.1 Green Infrastructure

The implementation of roadside rain gardens/bioswales and permeable pavements may be viewed favorably by the public.

- Environmental Impacts – Green infrastructure source controls received a very good rating because construction would generate minimal impacts related to traffic, dust, and noise.
- Social Benefits – Green infrastructure source controls received an excellent rating in social benefits due to the green space generated from its implementation. Attractive native plants can be used in green infrastructure installations.
- Multiple-use Considerations – Green infrastructure source controls received a very good rating for multiple-use considerations because green space generated by infrastructure can be used as recreational space or be used to grow beneficial plants that can help the community.

7.3.4.2 Satellite Storage Tank(s)

Factors that affect public acceptance are discussed below for the below ground storage tank alternatives:

- Environmental Impacts ‐ Construction of the tanks should have minimal negative environmental impacts to residential or densely populated areas. However, the construction could impact natural environments and various permits would need to be obtained. Environmental impacts associated with storage tank implementation would have to be addressed on a site-specific basis.
- Social Benefits ‐ The storage tank alternatives received good ratings for social benefits because the storage tanks can be constructed unobtrusively in public spaces.
- Multiple-use Considerations ‐ Upon completion of storage tank construction in the proposed locations, Johnson Park and the Costco parking lot, public use of the spaces would not be impacted or inhibited.

7.3.4.3 Storage Tunnel

The public would not object to the implementation of a tunnel alternative. Tunnels are viewed as a favorable alternative because there will be limited traffic and utilities disruption as well as little impact to above ground real estate. It is recommended that the tunnel alternative be paired with green infrastructure to give the public a visible representation of the City's improvements.

- Environmental Impacts – The storage tunnel alternative received a very good rating because most of the construction will take place beneath the ground and would cause minimal disruption to nature and the residents of Hackensack. Construction should not generate traffic or noise or require the movement of utilities. Natural environments would be minimally impacted.
- Social Benefits – Tunnel storage received a good rating in social benefits because the tunnel could have a positive effect on water quality and generate useful space over drop-shafts.
- Multiple-use Considerations – Tunnel storage received a good rating for multiple-use considerations because drop-shafts could be turned into parks, playgrounds, or parking lots.

7.3.4.4 I/I Reduction

As discussed in Chapter 7.3.2.4, the inflow and infiltration pipes replacement would involve substantial road work and would significantly interrupt transportation. Public acceptance for I/I pipes replacement may not be high. Pipe rehabilitation is less invasive, but the downside is the pipes rehabilitated will stay the same size. Instead, replacement would open the opportunity to build larger capacity pipes to fulfill potential future capacity needs.

- Environmental Impacts – I/I reduction received a good rating for environmental impacts. While pipe replacement may have negative environmental impacts from the construction including traffic disruption, potential utilities relocation, noise, and dust, pipe rehabilitation would not be very disruptive to the public.
- Social Benefits – I/I reduction received a poor rating for social benefits. No additional aesthetic value is achieved during or after the construction.
- Multiple-use Considerations – I/I reduction received a poor rating for multiple-use considerations because it solely serves for CSO and flooding control.

7.3.4.5 Treatment of CSO Discharge

- Environmental Impacts – Since the CSO treatment facilities would be located on the periphery of the City, construction of the facilities should have minimal negative environmental impacts to residential or densely populated areas. Additionally, construction should not impact the natural environment because construction should only take place near the existing screening facilities. However, the number of CSO discharges will not change and therefore disinfected CSOs would continue to occur in the Hackensack River.
- Social Benefits – CSO treatment facilities would not provide any aesthetic benefit to the City.

- Multiple-use Considerations – CSO treatment facilities do not have an additional purpose beyond treating CSO discharges.

7.3.5 Performance

Implementation of the alternatives was simulated for the 2004 typical year using the City's PCSWMM model, and the impacts on overflow volume and percent capture were estimated. The performance results for the evaluated alternatives are presented in Table 7-2. As summarized in Table 7-1, the alternatives were rated relative to each other for performance in their effectiveness at achieving CSO control by reducing overflow volume and increasing percent capture.

7.3.5.1 Green Infrastructure

For the 5% impervious area alternative, the pervious pavement and roadside rain gardens/bioswales will be designed based on the sites proposed in the Rutgers Report. The additional rain gardens/bioswales needed to achieve treatment of 10% of the City's impervious area will be designed to treat rainwater from an impervious area that is five to ten times the area (referred to as a loading rate, ranging from 5:1 to 10:1). Each installation should capture the first inch of rain that falls on the treatment area. Model simulations in PCSWMM indicated that green infrastructure can lower the number of overflows from 56 to 51 and reduce the CSO volume by 33 MG and 38 MG for the 5% and 10% impervious area alternatives, respectively. The results of these alternatives can be found in Table 7-2.

7.3.5.2 Satellite Storage Tank(s)

The performance of the satellite storage tanks alternatives was evaluated for 4, 8, 12, and 20 overflows and for 85% capture in the typical year. For the separate tank alternative, two storage nodes were added to the PCSWMM model at approximately the locations shown in Figure 7-2, one upstream of the Anderson and Court Street outfalls, each. A pump link was added to connect each new storage node to the BCUA trunk sewer to model the pumping back of combined sewage after wet weather events. For the regional tank alternative, one storage node was added upstream of the Court Street outfall. The Anderson Street outfall was closed, and a new gravity sewer conduit was added to divert the Anderson subdrainage area flows to the tank located near Court Street.

The tanks for the separate tanks and regional tank alternatives were sized by iteratively running the PCSWMM models and adjusting the tank diameters and depths to achieve the targeted number of overflows per year. The preliminary tank dimensions, total overflow volumes, and percent capture values for the 4, 8, 12, and 20 overflow and 85% capture alternatives are listed in Table 7-2. It is noted that, for each number of overflows per year, the volume of the regional tank was smaller than the combined volume of the two separate tanks; this is because, for the regional tank alternative, the new gravity sewer connecting the Anderson subdrainage area to the tank at Court Street provides additional storage.

7.3.5.3 Storage Tunnel

The tunnel performance was evaluated based on achieving a certain number of overflows and a minimum percent capture, using PCSWMM modelling software. The targets were 4, 8, 12, and 20 overflows, and the percent capture target was 85%. Modelling showed that the tunnel would be effective in attaining the

Table 7-2: Performance Results Summary						
Sub Drainage Area (SDA), Outfall Name	Anderson Street	Court Street	Total Wet Weather Overflow Volume	Percent of Capture	No. of Overflows	Reduction of Overflow Volume from Baseline (%)
Outfall number	001A	002A				
Name of Alternative	OVERFLOW VOLUME (MG)					
Baseline condItions for 2004	105.3	151.3	256.6	68%	56	N/A
Disinfection	105.3	151.3	256.6	68%	56	N/A
Pretreatment & Disinfection	105.3	151.3	256.6	68%	56	N/A
GI - 5% Impervious Area	91.8	131.6	223.4	70%	51	13.0%
GI - 10% Impervious Area	91.6	127.0	218.6	70%	51	14.8%
Removal of Inflow and Infiltration (I&I)	105.2	151.2	256.4	68%	56	0.1%
Tunnel Storage - 17.8ft Diameter Tunnel (4 Overflows)	22.9	3.7	26.6	96%	4	89.6%
Tunnel Storage - 17ft Diameter Tunnel (8 Overflows)	26.5	6.3	32.8	95%	8	87.2%
Tunnel Storage - 14ft Diameter Tunnel (12 Overflows)	37.4	14.8	52.2	93%	12	79.7%
Tunnel Storage - 10.5 ft Diameter Tunnel (20 Overflows)	68.3	32.2	100.4	86%	20	60.9%
Tunnel Storage - 10.5 ft Diameter Tunnel (85% Capture)	68.3	32.2	100.4	86%	20	60.9%
Satellite Tanks - Two tanks, 115 ft dia., 100 ft deep (4 Overflows)	4.9	13.0	17.9	98%	4	93.0%
Satellite Tanks - Two tanks, 105 ft dia., 100 ft deep (8 Overflows)	6.5	19.8	26.4	96%	8	89.7%
Satellite Tanks - Two tanks, 87 ft dia., 100 ft deep (12 Overflows)	9.8	37.3	47.1	94%	12	81.6%
Satellite Tanks - Two tanks, 73 ft dia., 100 ft deep (20 Overflows)	28.3	56.7	85.0	89%	20	66.9%
Satellite Tanks - Two tanks, 60 ft dia., 100 ft deep (85% Capture)	43.6	77.7	121.3	85%	25	52.7%
Regional Tank - One tank, 120 ft dia., 130 ft deep (4 Overflows)	N/A	21.1	21.1	97%	4	91.8%
Regional Tank - One tank, 120 ft dia., 110 ft deep (8 Overflows)	N/A	31.0	31.0	96%	8	87.9%
Regional Tank - One tank, 100 ft dia., 105 ft deep (12 Overflows)	N/A	58.2	58.2	92%	12	77.3%
Regional Tank - One tank, 80 ft dia., 100 ft deep (20 Overflows)	N/A	93.6	93.6	88%	20	63.5%
Regional Tank - One tank, 65 ft dia., 100 ft deep (85% Capture)	N/A	118.7	118.7	85%	21	53.8%
Control Program Alternative - I&I Removal and GI 5% Impervious Area	91.6	131.5	223.1	70%	50	13.0%
Control Program Alternative - I&I Removal and GI 10% Impervious Area	91.7	127.0	218.7	70%	51	14.8%

overflow goals. Each overflow goal resulted in a percent capture over 80%. The size of the tunnel ranged from 10.5 feet in diameter for 20 overflows to 17.8 feet for 4 overflows. Percent capture ranged from 86% for 20 overflows up to 96% percent capture for 4 overflows. The 20 overflow scenario reduced overflow volumes by 61% and the 4-overflow scenario reduces overflow volumes by 90%. 85 percent capture was attained using the 20-overflow scenario which had a percent capture of 86%. The characteristics of the tunnels for each alternative and their performance results are presented in Table 7-2.

7.3.5.4 I/I Reduction

The alternative with inflow and infiltration reduction through pipe replacement or rehabilitation did not reduce the number of overflows using rainfall data for the typical year of 2004. The system's percent capture with implementation of I/I reduction changed by only approximately 0.1% from the baseline model scenario.

Figure 7-5 presents I/I flows for each sub drainage area with implementation of I/I reduction measures; an estimated total flow rate of 0.041 CFS or 0.027 MGD can be eliminated through inflow and infiltration pipes replacement or rehabilitation. This alternative alone will not satisfy the water quality requirements of the Permit.

7.3.5.5 Treatment of CSO Discharge

The disinfection only alternative was sized to treat 100% of the overflow volume that occurred during the 2004 typical year. The disinfection unit at Anderson Street can treat CSOs up to a flow rate of 133 MGD. The disinfection unit at Court Street can treat CSOs up to 89 MGD. 133 and 89 MGD were the maximum flow rates that occurred at the Anderson and Court Street outfalls, respectively, during the 2004 typical year.

The alternative with pretreatment and disinfection was sized to pretreat and disinfect 100% of the overflow volume at the Court Street outfall. The treatment unit placed near the Court Street outfall was sized to pretreat 96.5% and disinfect 100% of the overflow volume during the 2004 typical year. This alternative is capable of attaining current water quality standards but does not meet the 85% capture or 4 overflow goals.

7.4 Preliminary Control Program Alternative

7.4.1 Collection System and Source Controls

As described in Section 4.1.4, a condition assessment for observations of potential I/I issues was performed in 2015. I/I reduction likely will be the first phase of the LTCP to maintain the existing system, potentially reduce the need for expansion, and lower pumping and treatment costs. The possible next phase of the LTCP is implementing green infrastructure to achieve runoff reduction for 5% to 10% of the City's impervious area. Due to the social benefits, the GI phase may help improve public acceptance of the LTCP. The combined implementation of I/I reduction measures and green infrastructure was evaluated as an additional alternative for the City. The City's calibrated PCSWMM model was used to simulate implementation of these controls using the 2004 rainfall data.

The simulations estimated that the percent capture will improve from 68% for the baseline condition to 70% with implementation of this program, and the number of overflows will be reduced by five overflows.

Regardless, this program will help attain more public acceptance since improvements in the City will be readily seen; if the gray infrastructure projects (for example, off-line storage) need to be implemented for higher performance and percent capture goals or CSO reduction goal then the public may be more supportive than they would without these projects completed.

A summary of the evaluation criteria for this alternative is presented below:

- Very good rating in implementability.
- Fair to good rating in environmental considerations.
- Fair rating for performance because it would not significantly reduce the occurrence of CSOs.
- Good to excellent ratings for the institutional, siting, and cost criteria.

7.5 Preliminary Identification of Alternatives

7.5.1 Evaluation Factors

The alternatives were evaluated based on criteria from the EPA CSO Guidance for Long-Term Control Plan document. As stated in Section 7.1, for the current analysis, the criteria of siting, institutional issues, public acceptance, performance, implementability, and cost were evaluated and rated. As illustrated in Table 7-1, these factors were rated on a 1 to 5 scale (1 = Poor, 2 = Fair, 3 = Good, 4 = Very good, 5 = Excellent). Siting issues, institutional issues, public acceptance, and performance were evaluated for each alternative as discussed in Sections 7.3.2, 7.3.3, 7.3.4, and 7.3.5, respectively. Implementability is discussed for each alternative in the following sections. Cost is discussed in Section 7.5.3.

The definitions of the factors that contribute to implementability are defined below:

- **Constructability** – Constructability refers to the level of challenges associated with activities during the project construction phase. While cost usually is a driving factor behind whether a project is implemented or not, there are other qualitative issues that affect constructability. Projects that take place near the surface and are of a smaller scale would receive an excellent rating in constructability. Projects that require a river crossing or rely on complex machinery or other complicated construction methods would receive a poor rating in constructability.
- **Reliability** – There are many technologies and techniques that have been developed to manage CSOs. The reliability rating is based on the track record of these technologies as well as their complexity. Complexity increases with the amount of moving parts involved with the technology. Technologies that are proven and have been implemented successfully in other locations would receive a higher rating, while an alternative that has not been proven or successfully implemented and has many moving parts would receive a poor rating.
- **Operability** – Consideration of operability includes the requirements for personnel to complete O&M and waste management. An alternative that requires skilled personnel for O&M and that generates a large amount of waste, or waste that has difficult disposal, would receive a poor rating.
- **Adaptability** – The ability for an alternative to be implemented in phases affects the adaptability rating. Phased implementation is beneficial because the capital costs can be distributed over time.

Additionally, implementing an alternative in small parts allows the earlier phases to be used to guide later phases and to determine whether the alternative should be implemented across the City. If an alternative can effectively be implemented in various locations or in phases, it would receive an excellent rating. If the location for implementation is restricted and the project can only be completed in large parts, the alternative would receive a poor rating.

7.5.1.1 Implementability of Green Infrastructure

Green infrastructure is a flexible, low impact technology to implement for CSO control.

- **Constructability** – Green infrastructure source controls received an excellent rating in constructability due to their ability to be constructed with limited specialized equipment outside of the vehicular travel lane.
- **Reliability** - Green infrastructure source controls received a good rating in reliability. They are known to perform well but are prone to several issues such as clogging from debris.
- **Operability** - Green infrastructure source controls received a good rating in operability because operating and maintaining green infrastructure does not take highly trained workers and the solids removed are not considered hazardous. Any solids removed from the green infrastructure feature can be disposed of as litter. The amount of maintenance and disposal required can vary depending on frequency and severity of rain.
- **Adaptability** - Green infrastructure source controls received an excellent rating in adaptability. The City can install any number of green infrastructure installations at any time at any location permitted by the city.

7.5.1.2 Implementability of Satellite Storage Tanks

The storage tank alternatives rank high in adaptability and reliability but low in constructability and operability relative to the other alternatives.

- **Constructability** – The storage tanks would require specialized equipment and highly trained workers for construction. Additionally, the storage tanks would require strong foundations and would ideally be constructed in deep rock formations. The regional storage tank alternative also would require construction of approximately 5,400 feet of new gravity sewer to divert flow from the Anderson Street subdrainage area to the new storage tank near Court Street; this construction would likely have many conflicts with existing utilities and would require extensive disturbance to traffic and pedestrians.
- **Reliability** – Storage tanks are considered a reliable and proven technology because they have been implemented in various locations nationwide.
- **Operability** – Storage tanks would not require frequent O&M but would require highly trained workers and solids removal. The solids that are removed from the tanks would require specialized transport and disposal.
- **Adaptability** – The separate storage tank alternative is adaptable for phased implementation. The storage tank in the area with higher water quality and local flooding concerns could be constructed

first. However, the regional storage tank alternative would not be very adaptable for phased implementation.

7.5.1.3 Implementability of Storage Tunnel

Implementing a storage tunnel would be a large and complex project. The following factors were evaluated to determine the alternative's implementability:

- **Constructability** – Off-line storage with tunnels received a fair rating in constructability. Construction of the tunnel alternative would require the involvement of both highly trained workers and specialized equipment.
- **Reliability** – Off-line storage with tunnels received an excellent rating in reliability. Storage tunnels are a proven technology and a popular solution utilized to manage CSOs. This technology is currently being implemented in other urban cities such as Washington, D.C.
- **Operability** – Similar to the storage tanks, tunnels would require highly skilled labor to conduct regular maintenance and solids removal.
- **Adaptability** – Off-line storage with tunnels received a fair rating in adaptability because there is limited flexibility for phased implementation. Once work begins on a tunnel it usually continues until the tunnel is complete. Phasing tunnel construction by segments leads to increased costs associated with repeated assembly and disassembly of tunnel boring machines.

7.5.1.4 Implementability of I/I Reduction

Pipe replacement/rehabilitation for infiltration and inflow control requires labor intensive work and temporary pumping measures. However, it would result in reduced volume of flow and additional capacity for future growth. It may also decrease the chance of local flooding or water in basements. It can be considered for combination with other technologies to provide effective solutions.

- **Constructability** – This alternative received a very good rating in constructability; lining for rehabilitation is easy to construct, but there are challenges for pipe replacement, such as coordination for re-connecting service lines.
- **Reliability** – This alternative received a very good rating in reliability. It is a widely applied technology, not complex to implement, and has limited moving parts.
- **Operability** – This alternative received an excellent rating in operability. It does not require skilled personnel and generates little waste.
- **Adaptability** – This alternative received an excellent rating in adaptability. It can easily be phased in various locations, over a flexible implementation schedule.

7.5.1.5 Implementability of Treatment of CSO Discharge

- **Constructability** – Treatment received a very good rating because the facilities do not present many challenges to construct. Construction would basically involve the installation of different units.

- **Reliability** – Treatment received a very good rating in reliability because the treatment units are proven technologies and have minimal moving parts.
- **Operability** – Treatment received a fair rating for operability because disinfection would require a significant amount of chemical transportation to replenish PAA after every CSO event. Experienced licenced drivers may be required to transport chemicals through the City.
- **Adaptability** – The treatment alternative received a good rating for adaptability because treatment units can be installed one at a time at each outfall. Installation can easily be phased between the two outfalls.

7.5.2 Regulatory Compliance

Table 7-3 summarizes if the preliminary selected alternatives meet the regulatory compliance of the City's Permit:

Table 7-3. Achievement of Compliance with Permit Requirements

			Regulatory Compliance	
Name of Alternative	**Modeled Percent Capture**	**Modeled No. of Overflows**	**Achieves 4 or Less Overflows**	**Achieves 85% Capture**
Baseline conditions for 2004	68%	56	No	No
GI - 5% Impervious Area	70%	51	No	No
GI - 10% Impervious Area	70%	51	No	No
Removal of Inflow and Infiltration (I&I)	68%	56	No	No
Tunnel Storage - 17.8ft Diameter Tunnel (4 Overflows)	96%	4	Yes	Yes
Tunnel Storage - 17ft Diameter Tunnel (8 Overflows)	95%	8	No	Yes
Tunnel Storage - 14ft Diameter Tunnel (12 Overflows)	93%	12	No	Yes
Tunnel Storage - 10.5 ft Diameter Tunnel (20 Overflows)	86%	20	No	Yes
Tunnel Storage - 10.5 ft Diameter Tunnel (85% Capture)	86%	20	No	Yes
Satellite Tanks - Two tanks, 115 ft dia., 100 ft deep (4 Overflows)	98%	4	Yes	Yes
Satellite Tanks - Two tanks, 105 ft dia., 100 ft deep (8 Overflows)	96%	8	No	Yes
Satellite Tanks - Two tanks, 87 ft dia., 100 ft deep (12 Overflows)	94%	12	No	Yes
Satellite Tanks - Two tanks, 73 ft dia., 100 ft deep (20 Overflows)	89%	20	No	Yes
Satellite Tanks - Two tanks, 60 ft dia., 100 ft deep (85% Capture)	85%	25	No	Yes
Regional Tank - One tank, 120 ft dia., 130 ft deep (4 Overflows)	97%	4	Yes	Yes
Regional Tank - One tank, 120 ft dia., 110 ft deep (8 Overflows)	96%	8	No	Yes
Regional Tank - One tank, 100 ft dia., 105 ft deep (12 Overflows)	92%	12	No	Yes
Regional Tank - One tank, 80 ft dia., 100 ft deep (20 Overflows)	88%	20	No	Yes
Regional Tank - One tank, 65 ft dia., 100 ft deep (85% Capture)	85%	21	No	Yes
Control Program Alternative - I&I Removal and GI 5% Impervious Area	70%	50	No	No
Control Program Alternative - I&I Removal and GI 10% Impervious Area	70%	51	No	No

7.5.3 Identification of Preliminary Alternatives

Based on the evaluations presented in this Alternatives Report, a combination of storage tanks or a tunnel with green infrastructure and I/I reduction may provide the most effective and economical CSO control alternatives for the City to incrementally implement in order to reach the required water quality goals. As future conditions change for the City and additional CSO technologies become available or improve, the other CSO control alternatives in this report may be revisited in order to suit the City's best interests and needs to meet the water quality goals.

Detailed opinions of probably costs for the alternatives are presented in this section. Alternatives that were estimated to cost below $20M received a rating of 5 (excellent), $20M-$50M received a rating of 4 (very good), $50M-$100M received a rating of 3 (good), $100M-$200M received a rating of 2 (fair), and $200M received a rating of 1 (poor).

Table 7-4 presents the cost for green infrastructure with installation of roadside rain gardens/bioswales and permeable pavement. As shown, the total present worth (TPW) cost for the alternative with 5% of impervious area controlled is $32M, while the TPW cost for the alternative with 10% of impervious area controlled is $43M.

Tables 7-5, 7-6, and 7-7 show the capital, O&M, and TPW costs for the storage alternatives including the satellite storage tanks, regional storage tank, and storage tunnel alternatives. These tables present the costs for various levels of performance (4, 8, 12, or 20 overflows or 85% capture). The conclusions are summarized below:

- The satellite storage tanks alternative has estimated TPW costs of $140M, $123M, $96M, $79M, and $66M for 4, 8, 12, and 20 overflows, and 85% capture, respectively.
- The regional storage tank alternative has estimated TPW costs of $119M, $108M, $87M, $71M, and $63M for 4, 8, 12, and 20 overflows, and 85% capture, respectively.
- The tunnel alternative has estimated TPW costs of $97M, $94M, $85M, and $74M for 4, 8, 12, and 20 overflows, respectively. The tunnel size to achieve 85% capture would not change from the size needed to achieve 20 overflows; therefore, the cost for an 85% capture scenario would also be $74M.
- The tunnel alternative has lower estimated costs than the satellite storage tanks alternative for the 4, 8, 12, and 20 overflow scenarios.
- The tunnel alternative has lower estimated costs than the regional storage tank alternative for the 4, 8, and 12 overflow scenarios, but higher costs than the regional tank alternative for the 20 overflow scenario.

As shown in Table 7-8, the I/I reduction through pipe replacement and rehabilitation alternative has an estimated TPW cost of $11M.

As shown in Table 7-9, the alternative involving disinfection, only, has an estimated TPW cost of $16M. Disinfection with pretreatment (SanSep) has an estimated TPW cost of $50M.

The estimated TPW cost of $555M for system-wide sewer separation was provided for reference in Section 4.1.5 to illustrate that it is not a feasible alternative because of the high cost relative to other alternatives.

The detailed costs for each alternative are presented in Appendix A.

Table 7-4. Capital, O&M, and Present Worth Cost Breakdown for Green Infrastructure

GI	Present Worth Cost			
	Total Capital Cost	Land Cost	Annual O&M Cost	Present Worth
5% Impervious Area Controlled	$ 27,790,000	NA	$ 280,000	$ 31,980,000
			Total Present Worth Cost	**$ 31,980,000**
10% Impervious Area Controlled	$ 34,670,000	NA	$ 520,000	$ 42,640,000
			Total Present Worth Cost	**$ 42,640,000**

Table 7-5. Capital, O&M, and Present Worth Cost Breakdown for Satellite Storage Tanks

Overflows /Percent Capture	Tank Location	Storage Tank			Present Worth Cost			
		Diameter (ft)	Tank Volume (MG)	Land Required (acres)	Total Capital Cost	Land Cost	Annual O&M Cost	Present Worth
4	Anderson	115	7.77	0.490	$ 51,210,000	$ 1,440,000	$ 1,140,000	$ 70,000,000
	Court	115	7.77	0.490	$ 51,210,000	$ 1,440,000	$ 1,140,000	$ 70,000,000
							Total Present Worth Cost	**$ 140,000,000**
8	Anderson	105	6.48	0.430	$ 44,680,000	$ 1,270,000	$ 1,020,000	$ 61,530,000
	Court	105	6.48	0.430	$ 44,680,000	$ 1,270,000	$ 1,020,000	$ 61,530,000
							Total Present Worth Cost	**$ 123,060,000**
12	Anderson	87	4.45	0.340	$ 34,420,000	$ 990,000	$ 840,000	$ 48,210,000
	Court	87	4.45	0.340	$ 34,420,000	$ 990,000	$ 840,000	$ 48,210,000
							Total Present Worth Cost	**$ 96,420,000**
20	Anderson	73	3.13	0.270	$ 27,770,000	$ 800,000	$ 720,000	$ 39,570,000
	Court	73	3.13	0.270	$ 27,770,000	$ 800,000	$ 720,000	$ 39,570,000
							Total Present Worth Cost	**$ 79,140,000**
85%	Anderson	60	2.11	0.22	$ 22,630,000	$ 640,000	$ 630,000	$ 32,890,000
	Court	60	2.11	0.22	$ 22,630,000	$ 640,000	$ 630,000	$ 32,890,000
							Total Present Worth Cost	**$ 65,780,000**

Table 7-6. Capital, O&M, and Present Worth Cost Breakdown for Regional Storage Tank

Overflows /Percent Capture	Storage Tank			Present Worth Cost			
	Diameter (ft)	Tank Volume (MG)	Land Required (acres)	Total Capital Cost	Land Cost	Annual O&M Cost	Present Worth
4	120	65.00	0.521	$ 95,300,000	$ 1,530,000	$ 1,430,000	$ 118,600,000
						Total Present Worth Cost	**$ 118,600,000**
8	120	11.00	0.521	$ 86,750,000	$ 1,530,000	$ 1,280,000	$ 107,730,000
						Total Present Worth Cost	**$ 107,730,000**
12	100	6.17	0.406	$ 70,900,000	$ 1,190,000	$ 1,000,000	$ 87,250,000
						Total Present Worth Cost	**$ 87,250,000**
20	80	3.76	0.305	$ 58,720,000	$ 890,000	$ 780,000	$ 71,480,000
						Total Present Worth Cost	**$ 71,480,000**
85%	65	2.48	0.238	$ 52,270,000	$ 700,000	$ 660,000	$ 63,080,000
						Total Present Worth Cost	**$ 63,080,000**

Table 7-7. Capital, O&M, and Present Worth Cost Breakdown for Storage Tunnel

Overflows	Storage Tunnel				Present Worth Cost			
	Length (ft)	Depth (ft)	Tunnel Diameter (ft)	Land Required (Acres)	Total Capital Cost	Land Cost	Annual O&M Cost	Present Worth
4	5,530	100	18	0.37	$78,020,000	$1,080,000	$1,190,000	$97,300,000
							Total Present Worth Cost	**$97,300,000**
8	5,530	100	17	0.37	$75,340,000	$1,080,000	$1,160,000	$94,160,000
							Total Present Worth Cost	**$94,160,000**
12	5,530	100	14	0.37	$67,300,000	$1,080,000	$1,070,000	$84,740,000
							Total Present Worth Cost	**$84,740,000**
20	5,530	100	10.5	0.37	$57,930,000	$1,080,000	$970,000	$73,750,000
							Total Present Worth Cost	**$73,750,000**

Table 7-8. Capital, O&M, and Present Worth Cost Breakdown for Infiltration and Inflow Pipes Replacement and Rehabilitation

Inflow and Infiltration	Present Worth Cost			
	Total Capital Cost	Land Cost	Annual O&M Cost	Present Worth
Replacement and Rehabilitation	$11,310,000	NA	NA	$11,310,000
			Total Present Worth Cost	$11,310,000

Table 7-9. Capital, O&M, and Present Worth Cost Breakdown for CSO Discharge Treatment

CSO Treatment	Tank Location	Present Worth Cost			
		Total Capital Cost	Land Cost	Annual O&M Cost	Present Worth
Disinfection Only	Anderson	$4,725,000	$269,000	$257,000	$8,910,000
	Court	$3,915,000	$202,000	$182,000	$6,890,000
				Total Present Worth Cost	$15,800,000
Disinfection with SanSep	Anderson	$21,935,000	$538,000	$259,900	$26,430,000
	Court	$20,285,000	$404,000	$184,800	$23,500,000
				Total Present Worth Cost	$49,930,000

8 SUMMARY

Preliminary analysis indicates that a combination of storage tanks or tunnel with green infrastructure and I/I reduction is the most socially acceptable, effective, and economical CSO control alternative for the City to incrementally implement in order to reach the required water quality goals at the time of this Alternatives Report. However, it should be noted that these are preliminary alternatives and no conclusions have been made for the City's recommended plan. As future conditions change for the City and additional CSO technologies become available or improve, the other CSO control alternatives in this Report may be revisited in order to suit the City's best interests and needs to meet the water quality goals.

Table 8-1 summarizes the CSO control alternatives that were evaluated. Table 8-1 presents the performance, costs and key constraints of each alternative considered.

Table 8-1: Summary						
Name of Alternative	**Percent of Capture**	**No. of Overflows**	**Reduction of Overflow Volume from Baseline (%)**	**Total Estimated Cost**	**Cost per Gallon of CSO Removed**	**Key Constraints**
Baseline condltions for 2004	68%	56	N/A	-	-	-
Disinfection	68%	56	N/A	$ 15,800,000	N/A	Uncertain if this alternative satifies water quality goals, number of overflows not reduced, no pretreatment.
Pretreatment & Disinfection	68%	56	N/A	$ 49,930,000	N/A	Uncertain if this alternative satifies water quality goals, number of overflows not reduced.
GI - 5% Impervious Area	70%	51	13.0%	$ 31,980,000	$0.96	Does not reach performance & water quality goals, number of overflows not reduced.
GI - 10% Impervious Area	70%	51	14.8%	$ 42,640,000	$1.12	Does not reach performance & water quality goals, number of overflows not reduced.
Removal of Inflow and Infiltration (I&I)	68%	56	0.1%	$ 11,310,000	$59.84	Does not reach performance & water quality goals, number of overflows not reduced.
Tunnel Storage - 17.8ft Diameter Tunnel (4 Overflows)	96%	4	89.6%	$ 97,300,000	$0.43	Constructability of a deep tunnel has risks, high cost.
Tunnel Storage - 17ft Diameter Tunnel (8 Overflows)	95%	8	87.2%	$ 94,160,000	$0.43	Constructability of a deep tunnel has risks, high cost.
Tunnel Storage - 14ft Diameter Tunnel (12 Overflows)	93%	12	79.7%	$ 84,740,000	$0.43	Constructability of a deep tunnel has risks, high cost.
Tunnel Storage - 10.5 ft Diameter Tunnel (20 Overflows)	86%	20	60.9%	$ 73,750,000	$0.49	Constructability of a deep tunnel has risks, high cost.
Tunnel Storage - 10.5 ft Diameter Tunnel (85% Capture)	86%	20	60.9%	$ 73,750,000	$0.49	Constructability of a deep tunnel has risks, high cost.
Satellite Tanks - Two tanks, 115 ft dia., 100 ft deep (4 Overflows)	98%	4	93.0%	$ 140,000,000	$0.59	Siting issues for tank locations, high cost.
Satellite Tanks - Two tanks, 105 ft dia., 100 ft deep (8 Overflows)	96%	8	89.7%	$ 123,060,000	$0.53	Siting issues for tank locations, high cost.
Satellite Tanks - Two tanks, 87 ft dia., 100 ft deep (12 Overflows)	94%	12	81.6%	$ 96,420,000	$0.46	Siting issues for tank locations, high cost.
Satellite Tanks - Two tanks, 73 ft dia., 100 ft deep (20 Overflows)	89%	20	66.9%	$ 79,140,000	$0.46	Siting issues for tank locations, high cost.
Satellite Tanks - Two tanks, 60 ft dia., 100 ft deep (85% Capture)	85%	25	52.7%	$ 65,780,000	$0.49	Siting issues for tank locations, high cost.
Regional Tank - One tank, 120 ft dia., 130 ft deep (4 Overflows)	97%	4	91.8%	$ 118,600,000	$0.50	Siting issues for tank location, large diameter sewer construction, high cost.
Regional Tank - One tank, 120 ft dia., 110 ft deep (8 Overflows)	96%	8	87.9%	$ 107,730,000	$0.48	Siting issues for tank location, large diameter sewer construction, high cost.
Regional Tank - One tank, 100 ft dia., 105 ft deep (12 Overflows)	92%	12	77.3%	$ 87,250,000	$0.44	Siting issues for tank location, large diameter sewer construction, high cost.
Regional Tank - One tank, 80 ft dia., 100 ft deep (20 Overflows)	88%	20	63.5%	$ 71,480,000	$0.44	Siting issues for tank location, large diameter sewer construction, high cost.
Regional Tank - One tank, 65 ft dia., 100 ft deep (85% Capture)	85%	21	53.8%	$ 63,080,000	$0.46	Siting issues for tank location, large diameter sewer construction, high cost.
Control Program Alternative - I&I Removal and GI 5% Impervious Area	70%	50	13.0%	$ 43,290,000	$1.29	Does not reach performance & water quality goals, number of overflows not reduced.
Control Program Alternative - I&I Removal and GI 10% Impervious Area	70%	51	14.8%	$ 53,950,000	$1.43	Does not reach performance & water quality goals, number of overflows not reduced.

FIGURES

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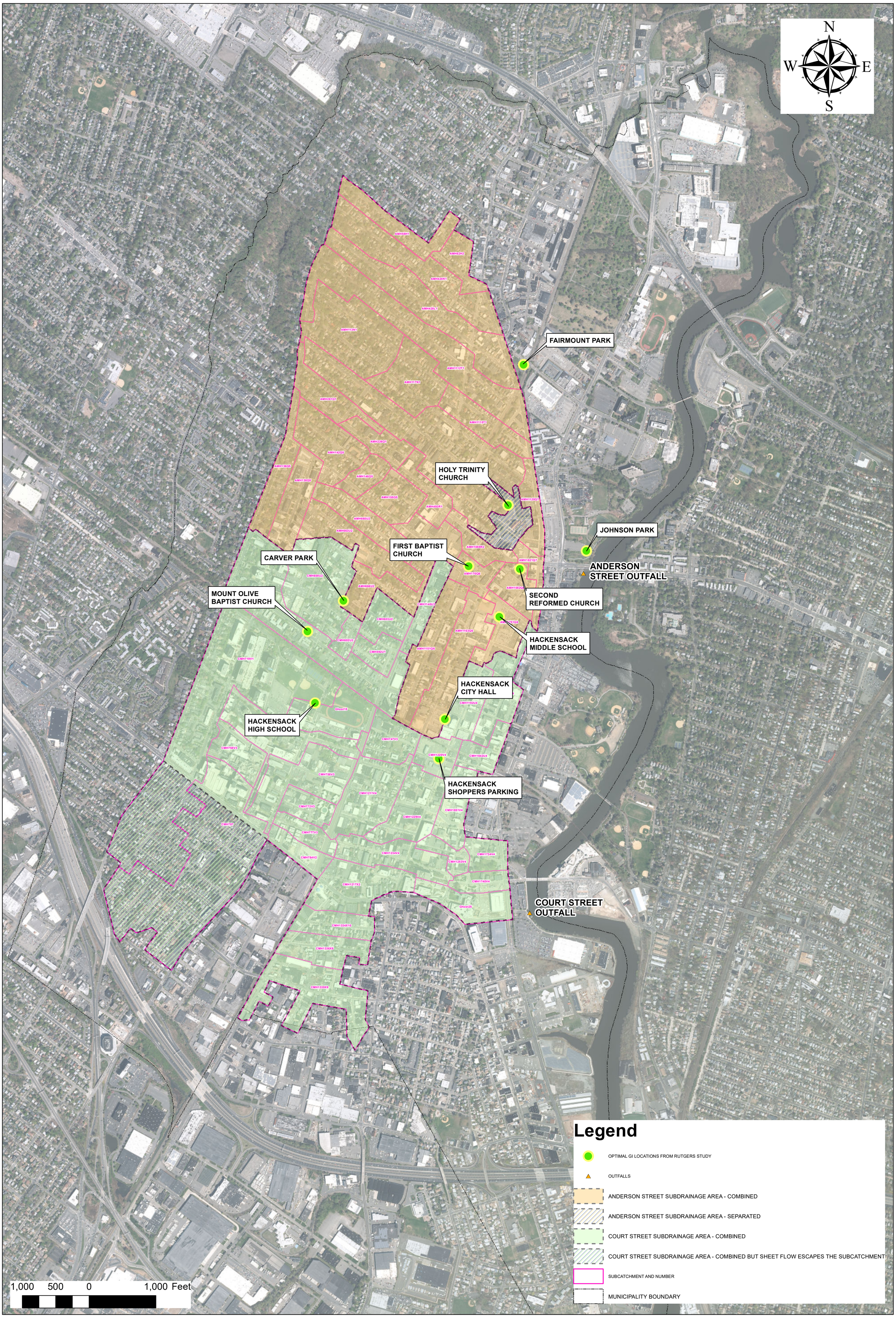

CITY OF HACKENSACK
DEVELOPMENT OF ALTERNATIVES
GREEN INFRASTRUCTURE LOCATIONS

Figure 7-1

ARCADIS Design & Consultancy for natural and built assets

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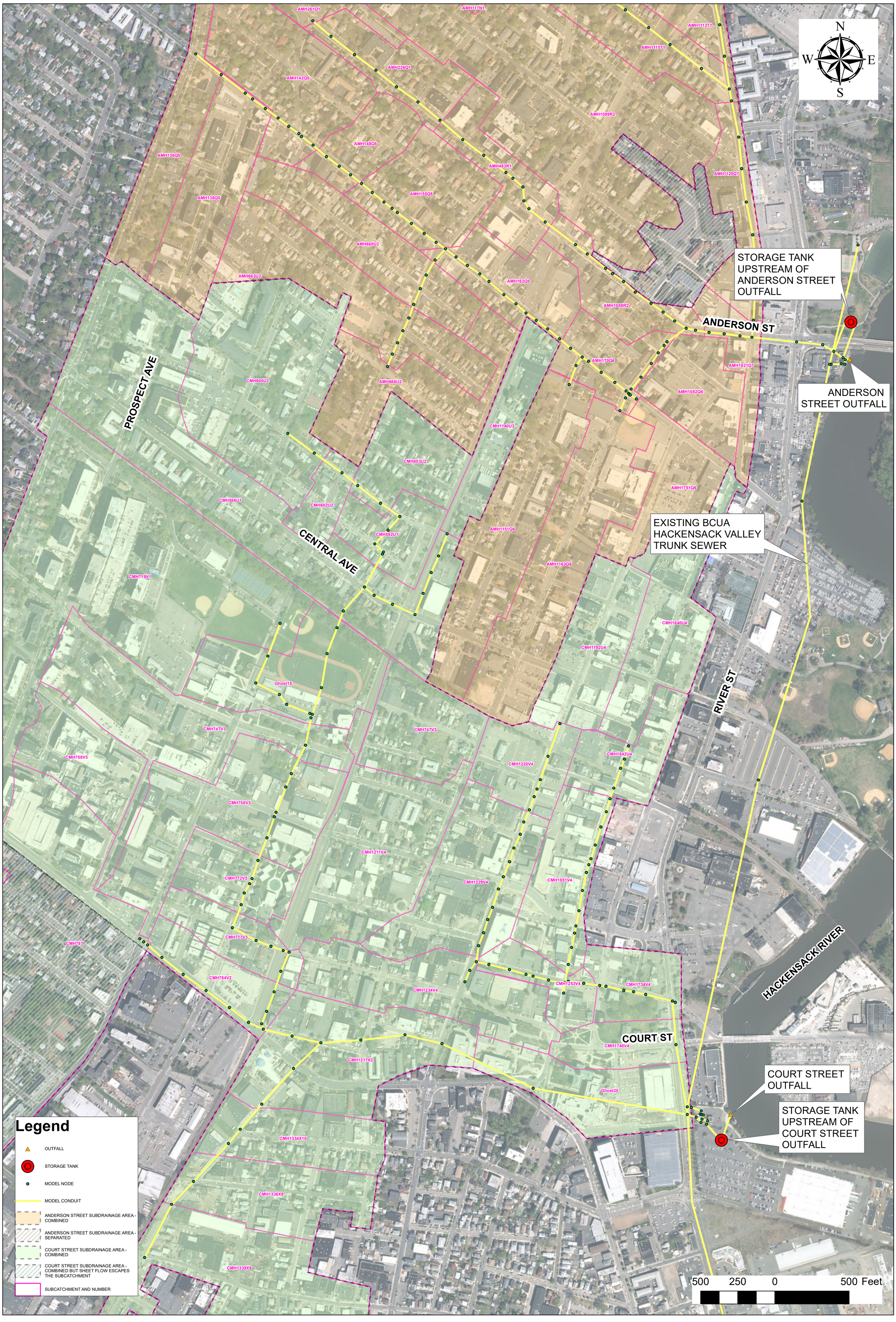

CITY OF HACKENSACK
DEVELOPMENT OF ALTERNATIVES
STORAGE TANK ALTERNATIVE

Figure 7-2

ARCADIS Design & Consultancy for natural and built assets

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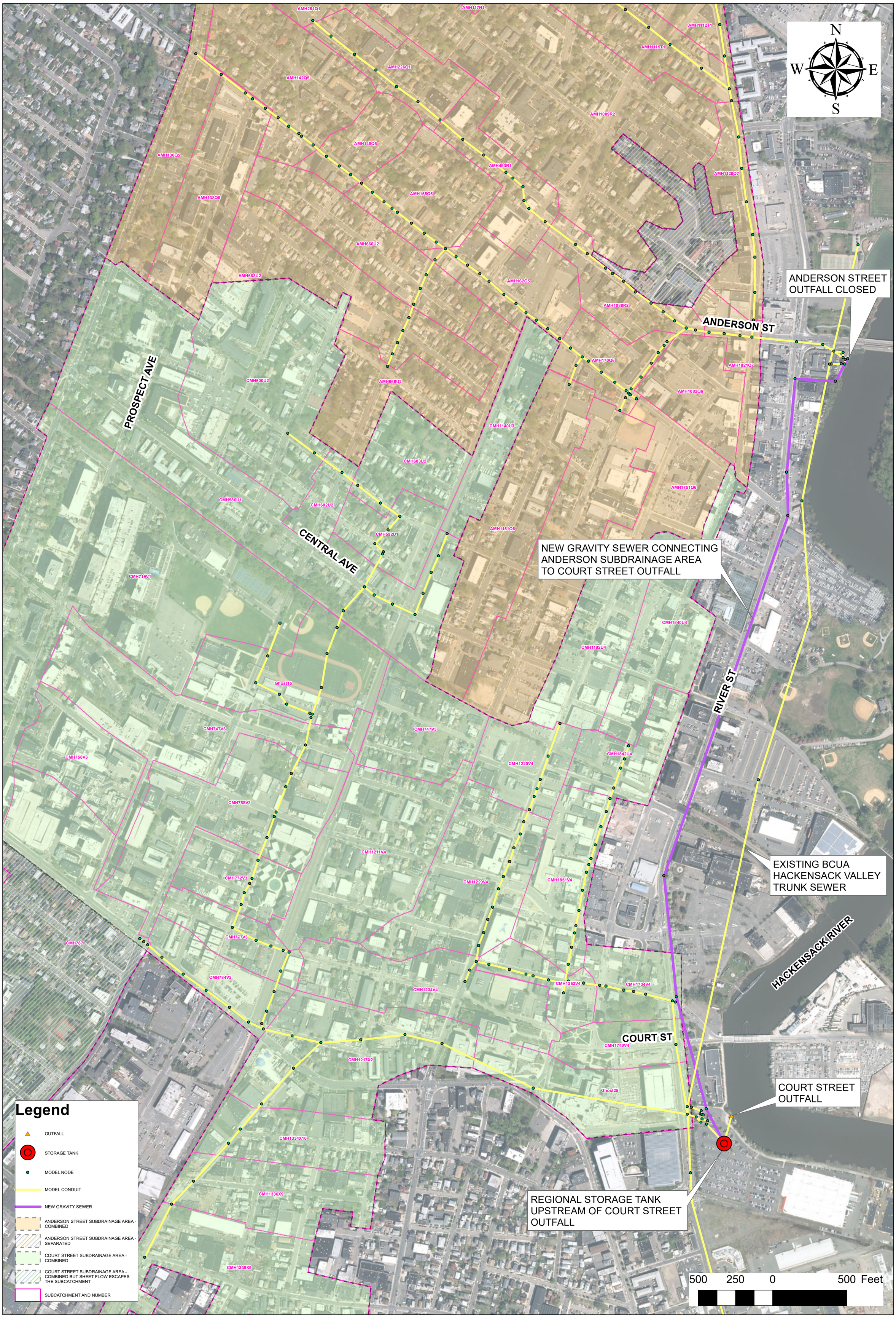

CITY OF HACKENSACK
DEVELOPMENT OF ALTERNATIVES
STORAGE TANK ALTERNATIVE - REGIONAL TANK

Figure 7-3

ARCADIS Design & Consultancy for natural and built assets

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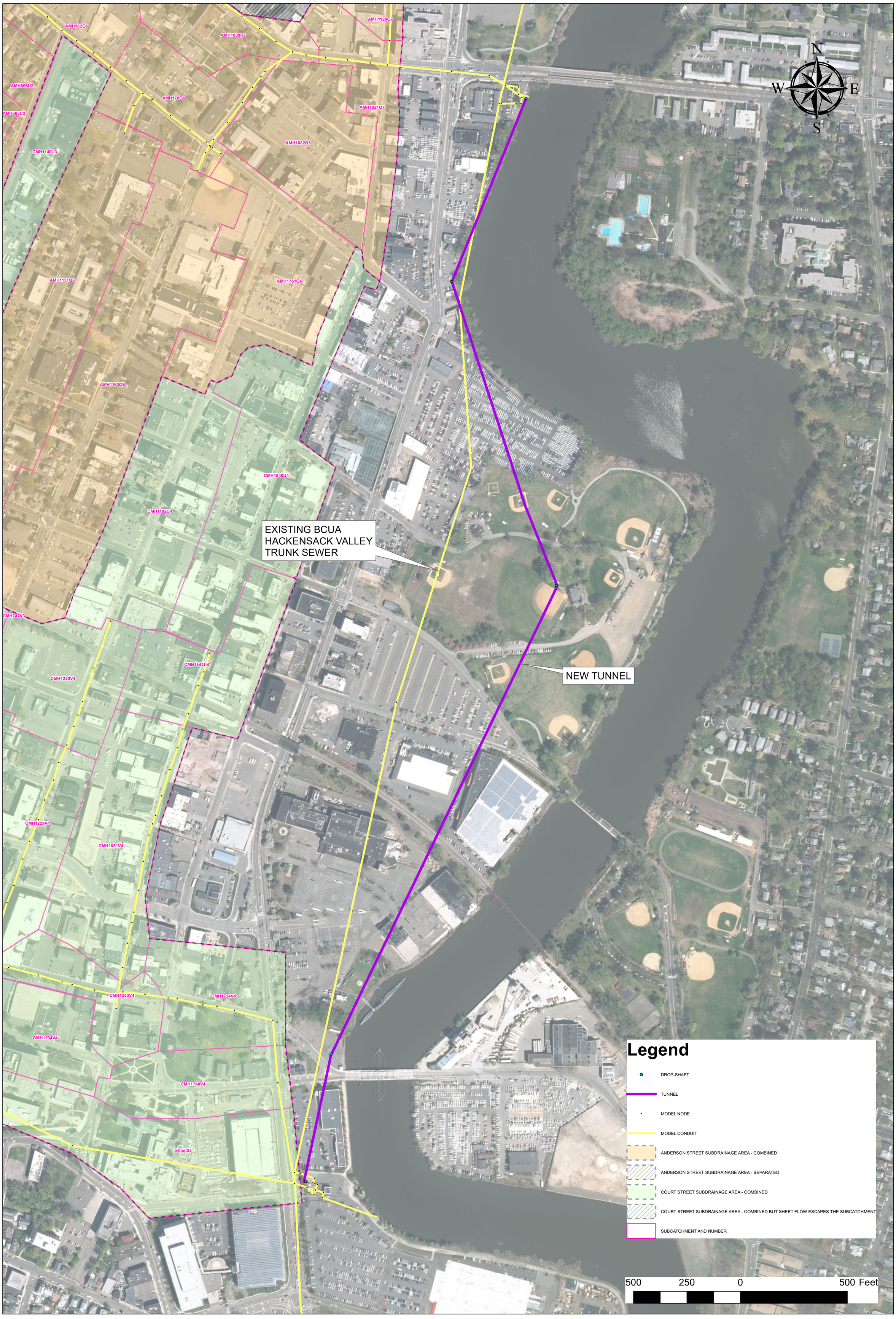

CITY OF HACKENSACK
DEVELOPMENT OF ALTERNATIVES
TUNNEL ALTERNATIVE

Figure 7-4

ARCADIS Design & Consultancy for natural and built assets

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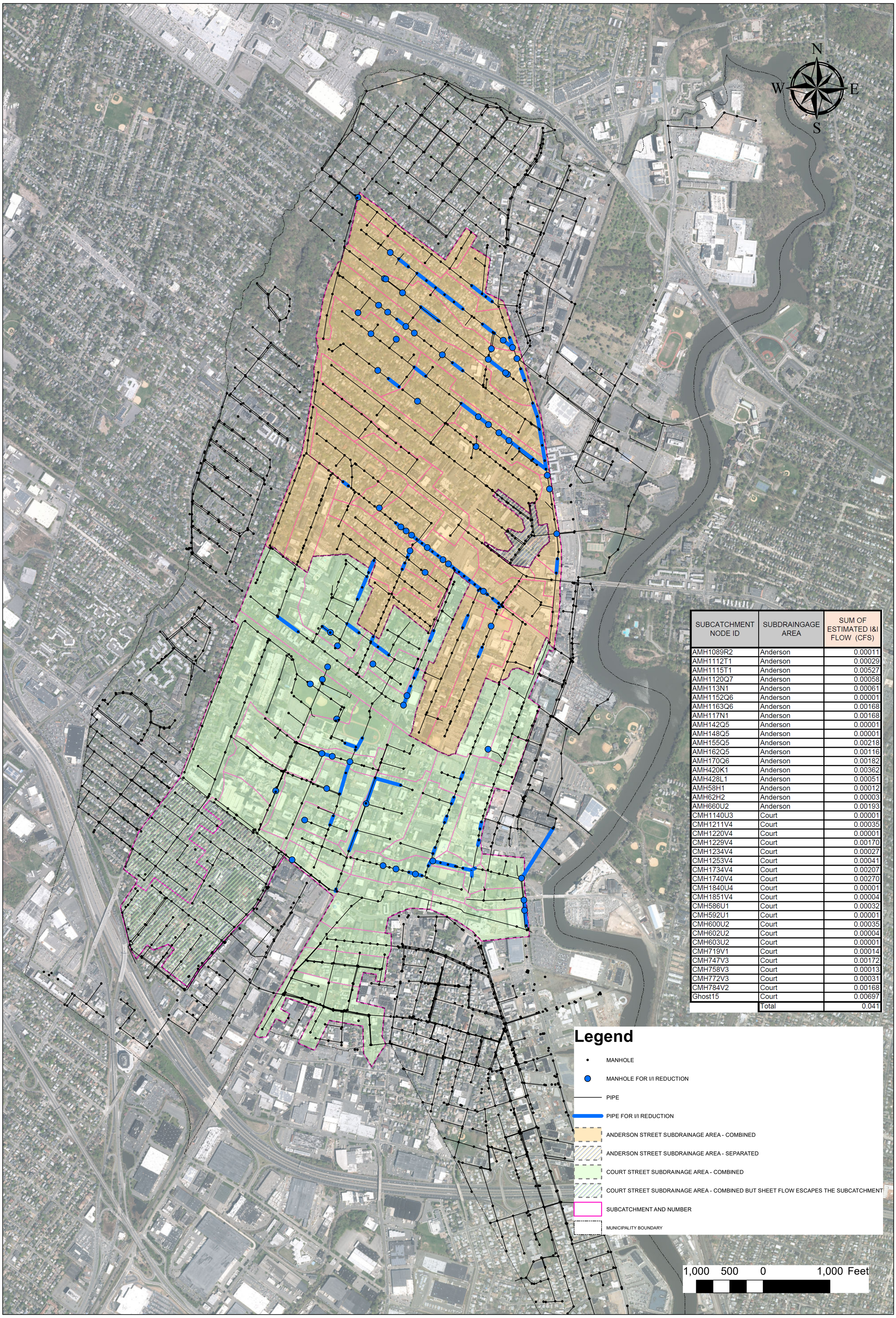

SUBCATCHMENT NODE ID	SUBDRAINGAGE AREA	SUM OF ESTIMATED I&I FLOW (CFS)
AMH1089R2	Anderson	0.00011
AMH1112T1	Anderson	0.00029
AMH1115T1	Anderson	0.00527
AMH1120Q7	Anderson	0.00058
AMH113N1	Anderson	0.00061
AMH1152Q6	Anderson	0.00001
AMH1163Q6	Anderson	0.00168
AMH117N1	Anderson	0.00168
AMH142Q5	Anderson	0.00001
AMH148Q5	Anderson	0.00001
AMH155Q5	Anderson	0.00218
AMH162Q5	Anderson	0.00116
AMH170Q6	Anderson	0.00182
AMH420K1	Anderson	0.00362
AMH428L1	Anderson	0.00051
AMH58H1	Anderson	0.00012
AMH62H2	Anderson	0.00003
AMH660U2	Anderson	0.00193
CMH1140U3	Court	0.00001
CMH1211V4	Court	0.00035
CMH1220V4	Court	0.00001
CMH1229V4	Court	0.00170
CMH1234V4	Court	0.00027
CMH1253V4	Court	0.00041
CMH1734V4	Court	0.00207
CMH1740V4	Court	0.00270
CMH1840U4	Court	0.00001
CMH1851V4	Court	0.00004
CMH586U1	Court	0.00032
CMH592U1	Court	0.00001
CMH600U2	Court	0.00035
CMH602U2	Court	0.00004
CMH603U2	Court	0.00001
CMH719V1	Court	0.00014
CMH747V3	Court	0.00172
CMH758V3	Court	0.00013
CMH772V3	Court	0.00031
CMH784V2	Court	0.00168
Ghost15	Court	0.00697
	Total	0.041

CITY OF HACKENSACK
DEVELOPMENT OF ALTERNATIVES
INFLOW / INFILTRATION ALTERNATIVE

Figure 7-5

ARCADIS Design & Consultancy for natural and built assets

APPENDIX A

Detailed Total Capital, O&M, and Present Worth Costs

Development and Evaluation of Alternatives

Table 1: Green Infrastructure Alternative #1: 5% of Impervious Area Controled

Description	Estimated Quantities	Unit Cost	Units	Total
Green Infrastructure Alternative #1: 5% of Impervious Area Controled				
Bioswale Construction Cost	9.8	$ 157,800	AC.	$ 1,630,000
Peameable Pavement Construction Cost	24.6	$ 573,400	AC.	$ 14,840,000
Green Infrastructure Total Construction Cost				$ 16,470,000
Total Cost with Contingency (25%)				$ 20,590,000
Overhead and Profit (15%)				$ 3,090,000
Legal and Engineering Costs (20%)				$ 4,120,000
Total Capital Cost				**$ 27,790,000**
Maintenance Green Infrastructure				$ 280,000
Annual O & M Cost				$ 280,000
***Present Worth O & M Cost**				**$ 4,190,000**
***Total Present Worth**				**$ 31,980,000**

*20 years life cycle costs for operation and maitenance with an interest rate of 2.75% for present value calculation (P/A = 15.227)

Development and Evaluation of Alternatives

Table 2: Green Infrastructure Alternative #2: 10% of Impervious Area Controled

Description	Estimated Quantities	Unit Cost	Units	Total
Green Infrastructure Alternative #2: 10% of Impervious Area Controled				
Bioswale Construction Cost	40.8	$157,800	AC.	$ 6,440,000
Peameable Pavement Construction Cost	24.6	$573,400	AC.	$ 14,110,000
Green Infrastructure Total Construction Cost				$ 20,550,000
Total Cost with Contingency (25%)				$ 25,680,000
Overhead and Profit (15%)				$ 3,850,000
Legal and Engineering Costs (20%)				$ 5,140,000
Total Capital Cost				**$ 34,670,000**
Maintenance Green Infrastructure				$ 520,000
Annual O & M Cost				$ 520,000
***Present Worth O & M Cost**				**$ 7,970,000**
***Total Present Worth**				**$ 42,640,000**

*20 years life cycle costs for operation and maitenance with an interest rate of 2.75% for present value calculation (P/A = 15.227)

Development and Evaluation of Alternatives
Table 3: Satellite Storage Tanks Alternative (4 Overflows)

Construction Phase	Description	Estimated Quantities	Unit Cost	Units	Total
Satellite Storage Tanks Alternative (4 Overflows)					
TS 1	**Tank #1: Anderson**				
	Tanks including Installation	7.8	$ 3,879,900	MG.	$ 30,140,000
	Force Main including Installation	300	$ 700	L.F.	$ 200,000
	Construction Total Cost				$ 30,350,000
	Total Cost with Contingency (25%)				$ 37,930,000
	Overhead and Profit (15%)				$ 5,690,000
	Legal and Engineering Costs (20%)				$ 7,590,000
	Total Capital Cost				**$ 51,210,000**
	Land Use	**0.49**	**$ 2,934,600**	**AC.**	**$ 1,440,000**
	Operation Storage				$ 240,000
	Maintenance Storage				$ 900,000
	Annual O & M Cost				$ 1,140,000
	***Present Worth O & M Cost**				**$ 17,350,000**
	***Total Present Worth**				**$ 70,000,000**
TS 2	**Tank #2: Court**				
	Tanks including Installation	7.8	$ 3,879,900	MG.	$ 30,140,000
	Force Main including Installation	300	$ 700	L.F.	$ 200,000
	Construction Total Cost				$ 30,350,000
	Total Cost with Contingency (25%)				$ 37,930,000
	Overhead and Profit (15%)				$ 5,690,000
	Legal and Engineering Costs (20%)				$ 7,590,000
	Total Capital Cost				**$ 51,210,000**
	Land Use	**0.49**	**$ 2,934,600**	**AC.**	**$ 1,440,000**
	Operation Storage				$ 240,000
	Maintenance Storage				$ 900,000
	Annual O & M Cost				$ 1,140,000
	***Present Worth O & M Cost**				**$ 17,350,000**
	***Total Present Worth**				**$ 70,000,000**
All Phases	***Total Present Worth**				**$ 140,000,000**

*20 years life cycle costs for operation and maitenance with an interest rate of 2.75% for present value calculation (P/A = 15.227)

Development and Evaluation of Alternatives
Table 4: Satellite Storage Tanks Alternative (8 Overflows)

Construction Phase	Description	Estimated Quantities	Unit Cost	Units	Total
Satellite Storage Tanks Alternative (8 Overflows)					
TS 1	**Tank #1: Anderson**				
	Tanks including Installation	6.5	$ 4,056,500	MG.	$ 26,270,000
	Force Main including Installation	300	$ 700	L.F.	$ 200,000
	Construction Total Cost				$ 26,470,000
	Total Cost with Contingency (25%)				$ 33,090,000
	Overhead and Profit (15%)				$ 4,960,000
	Legal and Engineering Costs (20%)				$ 6,620,000
	Total Capital Cost				**$ 44,680,000**
	Land Use	**0.43**	**$ 2,934,600**	**AC.**	**$ 1,270,000**
	Operation Storage				$ 240,000
	Maintenance Storage				$ 790,000
	Annual O & M Cost				$ 1,020,000
	***Present Worth O & M Cost**				**$ 15,580,000**
	***Total Present Worth**				**$ 61,530,000**
TS 2	**Tank #2: Court**				
	Tanks including Installation	6.5	$ 4,056,500	MG.	$ 26,270,000
	Force Main including Installation	300	$ 700	L.F.	$ 200,000
	Construction Total Cost				$ 26,470,000
	Total Cost with Contingency (25%)				$ 33,090,000
	Overhead and Profit (15%)				$ 4,960,000
	Legal and Engineering Costs (20%)				$ 6,620,000
	Total Capital Cost				**$ 44,680,000**
	Land Use	**0.43**	**$ 2,934,600**	**AC.**	**$ 1,270,000**
	Operation Storage				$ 240,000
	Maintenance Storage				$ 790,000
	Annual O & M Cost				$ 1,020,000
	***Present Worth O & M Cost**				**$ 15,580,000**
	***Total Present Worth**				**$ 61,530,000**
All Phases	***Total Present Worth**				**$ 123,060,000**

*20 years life cycle costs for operation and maitenance with an interest rate of 2.75% for present value calculation (P/A = 15.227)

Development and Evaluation of Alternatives
Table 5: Satellite Storage Tanks Alternative (12 Overflows)

Construction Phase	Description	Estimated Quantities	Unit Cost	Units	Total
Satellite Storage Tanks Alternative (12 Overflows)					
TS 1	**Tank #1: Anderson**				
	Tanks including Installation	4.4	$ 4,541,300	MG.	$ 20,190,000
	Force Main including Installation	300	$ 700	L.F.	$ 200,000
	Construction Total Cost				$ 20,390,000
	Total Cost with Contingency (25%)				$ 25,490,000
	Overhead and Profit (15%)				$ 3,820,000
	Legal and Engineering Costs (20%)				$ 5,100,000
	Total Capital Cost				**$ 34,420,000**
	Land Use	**0.34**	**$ 2,934,600**	**AC.**	**$ 990,000**
	Operation Storage				$ 240,000
	Maintenance Storage				$ 610,000
	Annual O & M Cost				$ 840,000
	***Present Worth O & M Cost**				**$ 12,800,000**
	***Total Present Worth**				**$ 48,210,000**
TS 2	**Tank #2: Court**				
	Tanks including Installation	4.4	$ 4,541,300	MG.	$ 20,190,000
	Force Main including Installation	300	$ 700	L.F.	$ 200,000
	Construction Total Cost				$ 20,390,000
	Total Cost with Contingency (25%)				$ 25,490,000
	Overhead and Profit (15%)				$ 3,820,000
	Legal and Engineering Costs (20%)				$ 5,100,000
	Total Capital Cost				**$ 34,420,000**
	Land Use	**0.34**	**$ 2,934,600**	**AC.**	**$ 990,000**
	Operation Storage				$ 240,000
	Maintenance Storage				$ 610,000
	Annual O & M Cost				$ 840,000
	***Present Worth O & M Cost**				**$ 12,800,000**
	***Total Present Worth**				**$ 48,210,000**
All Phases	***Total Present Worth**				**$ 96,420,000**

*20 years life cycle costs for operation and maitenance with an interest rate of 2.75% for present value calculation (P/A = 15.227)

Development and Evaluation of Alternatives
Table 6: Satellite Storage Tanks Alternative (20 Overflows)

Construction Phase	Description	Estimated Quantities	Unit Cost	Units	Total
Satellite Storage Tanks Alternative (20 Overflows)					
TS 1	**Tank #1: Anderson**				
	Tanks including Installation	3.1	$ 5,191,400	MG.	$ 16,250,000
	Force Main including Installation	300	$ 700	L.F.	$ 200,000
	Construction Total Cost				$ 16,450,000
	Total Cost with Contingency (25%)				$ 20,570,000
	Overhead and Profit (15%)				$ 3,090,000
	Legal and Engineering Costs (20%)				$ 4,110,000
	Total Capital Cost				**$ 27,770,000**
	Land Use	**0.27**	**$ 2,934,600**	**AC.**	**$ 800,000**
	Operation Storage				$ 240,000
	Maintenance Storage				$ 490,000
	Annual O & M Cost				$ 720,000
	***Present Worth O & M Cost**				**$ 11,000,000**
	***Total Present Worth**				**$ 39,570,000**
TS 2	**Tank #2: Court**				
	Tanks including Installation	3.1	$ 5,191,400	MG.	$ 16,250,000
	Force Main including Installation	300	$ 700	L.F.	$ 200,000
	Construction Total Cost				$ 16,450,000
	Total Cost with Contingency (25%)				$ 20,570,000
	Overhead and Profit (15%)				$ 3,090,000
	Legal and Engineering Costs (20%)				$ 4,110,000
	Total Capital Cost				**$ 27,770,000**
	Land Use	**0.27**	**$ 2,934,600**	**AC.**	**$ 800,000**
	Operation Storage				$ 240,000
	Maintenance Storage				$ 490,000
	Annual O & M Cost				$ 720,000
	***Present Worth O & M Cost**				**$ 11,000,000**
	***Total Present Worth**				**$ 39,570,000**
All Phases	***Total Present Worth**				**$ 79,140,000**

*20 years life cycle costs for operation and maitenance with an interest rate of 2.75% for present value calculation (P/A = 15.227)

Development and Evaluation of Alternatives
Table 7: Satellite Storage Tanks Alternative (85% Capture)

Construction Phase	Description	Estimated Quantities	Unit Cost	Units	Total
Satellite Storage Tanks Alternative (85% Capture)					
TS 1	**Tank #1: Anderson**				
	Tanks including Installation	2.1	$ 6,246,300	MG.	$ 13,210,000
	Force Main including Installation	300	$ 700	L.F.	$ 200,000
	Construction Total Cost				$ 13,410,000
	Total Cost with Contingency (25%)				$ 16,760,000
	Overhead and Profit (15%)				$ 2,510,000
	Legal and Engineering Costs (20%)				$ 3,350,000
	Total Capital Cost				**$ 22,630,000**
	Land Use	**0.22**	**$ 2,934,600**	**AC.**	**$ 640,000**
	Operation Storage				$ 240,000
	Maintenance Storage				$ 400,000
	Annual O & M Cost				$ 630,000
	***Present Worth O & M Cost**				**$ 9,610,000**
	***Total Present Worth**				**$ 32,890,000**
TS 2	**Tank #2: Court**				
	Tanks including Installation	2.1	$ 6,246,300	MG.	$ 13,210,000
	Force Main including Installation	300	$ 700	L.F.	$ 200,000
	Construction Total Cost				$ 13,410,000
	Total Cost with Contingency (25%)				$ 16,760,000
	Overhead and Profit (15%)				$ 2,510,000
	Legal and Engineering Costs (20%)				$ 3,350,000
	Total Capital Cost				**$ 22,630,000**
	Land Use	**0.22**	**$ 2,934,600**	**AC.**	**$ 640,000**
	Operation Storage				$ 240,000
	Maintenance Storage				$ 400,000
	Annual O & M Cost				$ 630,000
	***Present Worth O & M Cost**				**$ 9,610,000**
	***Total Present Worth**				**$ 32,890,000**
All Phases	***Total Present Worth**				**$ 65,780,000**

*20 years life cycle costs for operation and maitenance with an interest rate of 2.75% for present value calculation (P/A = 15.227)

Development and Evaluation of Alternatives
Table 8: Regional Storage Tank Alternative (4 Overflows)

Description	Estimated Quantities	Unit Cost	Units	Total
Regional Storage Tank Alternative (4 Overflows)				
Tanks including Installation	11	$ 3,620,100	MG.	$ 39,810,000
Force Main including Installation	300	$ 700	L.F.	$ 200,000
Water Main including Installation	5400	$ 3,000	L.F.	$ 16,460,000
Construction Total Cost				$ 56,480,000
Total Cost with Contingency (25%)				$ 70,590,000
Overhead and Profit (15%)				$ 10,590,000
Legal and Engineering Costs (20%)				$ 14,120,000
Total Capital Cost				**$ 95,300,000**
Land Use	**0.52**	**$ 2,934,600**	**AC.**	**$ 1,530,000**
Operation Storage				$ 240,000
Maintenance				$ 1,190,000
Annual O & M Cost				$ 1,430,000
***Present Worth O & M Cost**				**$ 21,760,000**
***Total Present Worth**				**$ 118,600,000**

*20 years life cycle costs for operation and maitenance with an interest rate of 2.75% for present value calculation (P/A = 15.227)

Development and Evaluation of Alternatives
Table 9: Regional Storage Tank Alternative (8 Overflows)

Description	Estimated Quantities	Unit Cost	Units	Total
Regional Storage Tank Alternative (8 Overflows)				
Tanks including Installation	9.3	$ 3,733,800	MG.	$ 34,750,000
Force Main including Installation	300	$ 700	L.F.	$ 200,000
Water Main including Installation	5400	$ 3,000	L.F.	$ 16,460,000
Construction Total Cost				$ 51,410,000
Total Cost with Contingency (25%)				$ 64,260,000
Overhead and Profit (15%)				$ 9,640,000
Legal and Engineering Costs (20%)				$ 12,850,000
Total Capital Cost				**$ 86,750,000**
Land Use	**0.52**	**$ 2,934,600**	**AC.**	**$ 1,530,000**
Operation Storage				$ 240,000
Maintenance Storage				$ 1,040,000
Annual O & M Cost				$ 1,280,000
***Present Worth O & M Cost**				**$ 19,450,000**
***Total Present Worth**				**$ 107,730,000**

*20 years life cycle costs for operation and maitenance with an interest rate of 2.75% for present value calculation (P/A = 15.227)

Development and Evaluation of Alternatives
Table 10: Regional Storage Tank Alternative (12 Overflows)

Description	Estimated Quantities	Unit Cost	Units	Total
Regional Storage Tank Alternative (12 Overflows)				
Tanks including Installation	6.2	$ 4,109,600	MG.	$ 25,350,000
Force Main including Installation	300	$ 700	L.F.	$ 200,000
Water Main including Installation	5400	$ 3,000	L.F.	$ 16,460,000
Construction Total Cost				$ 42,010,000
Total Cost with Contingency (25%)				$ 52,520,000
Overhead and Profit (15%)				$ 7,880,000
Legal and Engineering Costs (20%)				$ 10,500,000
Total Capital Cost				**$ 70,900,000**
Land Use	**0.41**	**$ 2,934,600**	**AC.**	**$ 1,190,000**
Operation Storage				$ 240,000
Maintenance Storage				$ 760,000
Annual O & M Cost				$ 1,000,000
***Present Worth O & M Cost**				**$ 15,160,000**
***Total Present Worth**				**$ 87,250,000**

*20 years life cycle costs for operation and maitenance with an interest rate of 2.75% for present value calculation (P/A = 15.227)

Development and Evaluation of Alternatives
Table 11: Regional Storage Tank Alternative (20 Overflows)

Description	Estimated Quantities	Unit Cost	Units	Total
Regional Storage Tank Alternative (20 Overflows)				
Tanks including Installation	3.8	$ 4,823,800	MG.	$ 18,140,000
Force Main including Installation	300	$ 700	L.F.	$ 200,000
Water Main including Installation	5400	$ 3,000	L.F.	$ 16,460,000
Construction Total Cost				$ 34,800,000
Total Cost with Contingency (25%)				$ 43,500,000
Overhead and Profit (15%)				$ 6,520,000
Legal and Engineering Costs (20%)				$ 8,700,000
Total Capital Cost				**$ 58,720,000**
Land Use	**0.3**	**$ 2,934,600**	**AC.**	**$ 890,000**
Operation Storage				$ 240,000
Maintenance Storage				$ 540,000
Annual O & M Cost				$ 780,000
***Present Worth O & M Cost**				**$ 11,860,000**
***Total Present Worth**				**$ 71,480,000**

*20 years life cycle costs for operation and maitenance with an interest rate of 2.75% for present value calculation (P/A = 15.227)

Development and Evaluation of Alternatives
Table 12: Regional Storage Tank Alternative (85% Capture)

Description	Estimated Quantities	Unit Cost	Units	Total
Regional Storage Tank Alternative (85% Capture)				
Tanks including Installation	2.5	$ 5,765,300	MG.	$ 14,310,000
Force Main including Installation	300	$ 700	L.F.	$ 200,000
Water Main including Installation	5400	$ 3,000	L.F.	$ 16,460,000
Construction Total Cost				$ 30,970,000
Total Cost with Contingency (25%)				$ 38,720,000
Overhead and Profit (15%)				$ 5,810,000
Legal and Engineering Costs (20%)				$ 7,740,000
Total Capital Cost				**$ 52,270,000**
Land Use	**0.24**	**$ 2,934,600**	**AC.**	**$ 700,000**
Operation Storage				$ 240,000
Maintenance Storage				$ 430,000
Annual O & M Cost				$ 660,000
***Present Worth O & M Cost**				**$ 10,120,000**
***Total Present Worth**				**$ 63,080,000**

*20 years life cycle costs for operation and maitenance with an interest rate of 2.75% for present value calculation (P/A = 15.227)

Development and Evaluation of Alternatives
Table 13: Tunnel Alternative (4 Overflows)

Description	Estimated Quantities	Unit Price	Units	Total
Tunnel Alternative (4 Overflows)				
Tunnel including Installation	5,530	$ 6,000	L.F.	$33,260,000
Tunnel Drop Shaft including Installation	2	$ 2,209,600	EA.	$4,420,000
Tunnel Drop Shaft (Access Only) including Installation	3	$ 1,262,600	EA.	$3,790,000
Force Main Installation	300	$ 500	L.F.	$150,000
Pumps for Storage Tanks including Installation	1	$ 4,620,300	EA.	$4,620,000
Construction Total Cost				$46,240,000
Total Cost with Contingency (25%)				$57,790,000
Overhead and Profit (15%)				$8,670,000
Legal and Engineering Costs (20%)				$11,560,000
Total Capital Cost				**$78,020,000**
Land Use	**0.37**	**$2,934,600**	**AC.**	**$1,080,000**
Operation Tunnel				$470,000
Maintenance Tunnel				$630,000
Maintenance Pump Station				$90,000
Annual O & M Cost				**$1,190,000**
***Present Worth O & M Cost**				**$18,190,000**
***Total Present Worth**				**$97,300,000**

*20 years life cycle costs for operation and maitenance with an interest rate of 2.75% for present value calculation (P/A = 15.227)

Development and Evaluation of Alternatives
Table 14: Tunnel Alternative (8 Overflows)

Description	Estimated Quantities	Unit Price	Units	Total
Tunnel Alternative (8 Overflows)				
Tunnel including Installation	5,530	$ 5,700	L.F.	$31,670,000
Tunnel Drop Shaft including Installation	2	$ 2,209,600	EA.	$4,420,000
Tunnel Drop Shaft (Access Only) including Installation	3	$ 1,262,600	EA.	$3,790,000
Force Main Installation	300	$ 500	L.F.	$150,000
Pumps for Storage Tanks including Installation	1	$ 4,620,300	EA.	$4,620,000
Construction Total Cost				$44,650,000
Total Cost with Contingency (25%)				$55,810,000
Overhead and Profit (15%)				$8,370,000
Legal and Engineering Costs (20%)				$11,160,000
Total Capital Cost				**$75,340,000**
Land Use	**0.37**	**$2,934,600**	**AC.**	**$1,080,000**
Operation Tunnel				$470,000
Maintenance Tunnel				$600,000
Maintenance Pump Station				$90,000
Annual O & M Cost				**$1,160,000**
***Present Worth O & M Cost**				**$17,730,000**
***Total Present Worth**				**$94,160,000**

*20 years life cycle costs for operation and maitenance with an interest rate of 2.75% for present value calculation (P/A = 15.227)

Development and Evaluation of Alternatives
Table 15: Tunnel Alternative (12 Overflows)

Description	Estimated Quantities	Unit Price	Units	Total
Tunnel Alternative (12 Overflows)				
Tunnel including Installation	5,530	$ 4,900	L.F.	$26,910,000
Tunnel Drop Shaft including Installation	2	$ 2,209,600	EA.	$4,420,000
Tunnel Drop Shaft (Access Only) including Installation	3	$ 1,262,600	EA.	$3,790,000
Force Main Installation	300	$ 500	L.F.	$150,000
Pumps for Storage Tanks including Installation	1	$ 4,620,300	EA.	$4,620,000
Construction Total Cost				$39,880,000
Total Cost with Contingency (25%)				$49,860,000
Overhead and Profit (15%)				$7,480,000
Legal and Engineering Costs (20%)				$9,970,000
Total Capital Cost				**$67,300,000**
Land Use	**0.37**	**$2,934,600**	**AC.**	**$1,080,000**
Operation Tunnel				$470,000
Maintenance Tunnel				$510,000
Maintenance Pump Station				$90,000
Annual O & M Cost				**$1,070,000**
***Present Worth O & M Cost**				**$16,350,000**
***Total Present Worth**				**$84,740,000**

*20 years life cycle costs for operation and maitenance with an interest rate of 2.75% for present value calculation (P/A = 15.227)

Development and Evaluation of Alternatives
Table 16: Tunnel Alternative (20 Overflows)

Description	Estimated Quantities	Unit Price	Units	Total
Tunnel Alternative (20 Overflows)				
Tunnel including Installation	5,530	$ 3,900	L.F.	$21,350,000
Tunnel Drop Shaft including Installation	2	$ 2,209,600	EA.	$4,420,000
Tunnel Drop Shaft (Access Only) including Installation	3	$ 1,262,600	EA.	$3,790,000
Force Main Installation	300	$ 500	L.F.	$150,000
Pumps for Storage Tanks including Installation	1	$ 4,620,300	EA.	$4,620,000
Construction Total Cost				$34,330,000
Total Cost with Contingency (25%)				$42,910,000
Overhead and Profit (15%)				$6,440,000
Legal and Engineering Costs (20%)				$8,580,000
Total Capital Cost				**$57,930,000**
Land Use	**0.37**	**$2,934,600**	**AC.**	**$1,080,000**
Operation Tunnel				$470,000
Maintenance Tunnel				$410,000
Maintenance Pump Station				$90,000
Annual O & M Cost				**$970,000**
***Present Worth O & M Cost**				**$14,740,000**
***Total Present Worth**				**$73,750,000**

*20 years life cycle costs for operation and maitenance with an interest rate of 2.75% for present value calculation (P/A = 15.227)

Development and Evaluation of Alternatives

Table 17: Inflow and Infiltration (I/I) Source Control Pipes Replacement and Rehabilitation Costs

Description	Estimated Quantities	Unit Cost	Units	Total
Replacement of Existing Sewer with 8 IN DI Sewer Pipes	420	$ 281	L.F.	$ 120,000
Replacement of Existing Sewer with 10 IN DI Sewer Pipes	380	$ 322	L.F.	$ 120,000
Replacement of Existing Sewer with 18 IN DI Sewer Pipes	170	$ 484	L.F.	$ 80,000
Replacement of Existing Sewer with 24 IN DI Sewer Pipes	100	$ 606	L.F.	$ 60,000
Replacement of Existing Sewer with 36 IN DI Sewer Pipes	410	$ 850	L.F.	$ 350,000
Replacement of Existing Sewer with 42 IN DI Sewer Pipes	880	$ 972	L.F.	$ 860,000
Replacement of Existing Sewer with 48 IN DI Sewer Pipes	1,320	$ 1,094	L.F.	$ 1,440,000
Replacement of Existing Sewer with 60 IN DI Sewer Pipes	210	$ 1,337	L.F.	$ 280,000
Rehibitation of Existing Sewer with 8 IN DI Sewer Pipes	1,870	$ 37	L.F.	$ 70,000
Rehibitation of Existing Sewer with 12 IN DI Sewer Pipes	320	$ 51	L.F.	$ 20,000
Rehibitation of Existing Sewer with 15 IN DI Sewer Pipes	150	$ 63	L.F.	$ 10,000
Rehibitation of Existing Sewer with 18 IN DI Sewer Pipes	1,140	$ 77	L.F.	$ 90,000
Rehibitation of Existing Sewer with 24 IN DI Sewer Pipes	2,480	$ 111	L.F.	$ 280,000
Rehibitation of Existing Sewer with 30 IN DI Sewer Pipes	1,550	$ 153	L.F.	$ 240,000
Rehibitation of Existing Sewer with 33 IN DI Sewer Pipes	120	$ 177	L.F.	$ 20,000
Rehibitation of Existing Sewer with 36 IN DI Sewer Pipes	1,230	$ 203	L.F.	$ 250,000
Rehibitation of Existing Sewer with 38 IN DI Sewer Pipes	100	$ 221	L.F.	$ 20,000
Rehibitation of Existing Sewer with 42 IN DI Sewer Pipes	1,700	$ 260	L.F.	$ 440,000
Rehibitation of Existing Sewer with 48 IN DI Sewer Pipes	1,230	$ 325	L.F.	$ 400,000
Rehibitation of Existing Sewer with 52 IN DI Sewer Pipes	880	$ 372	L.F.	$ 330,000
Rehibitation of Existing Sewer with 60 IN DI Sewer Pipes	130	$ 478	L.F.	$ 60,000
Demolition of Existing Sewer	1,230	$ 120	L.F.	$ 150,000
Sheeting and shoring at 10 foot average depth	1,830	$ 120	L.F.	$ 220,000
Manholes Replacement with frames and covers	67	$ 120	Ea.	$ 50,000
Pipe Bedding 10 foot depth	920	$ 120	CY	$ 110,000
DGA 10 foot depth	920	$ 120	CY	$ 110,000
Base Course Bituminous Pavement	420	$ 120	S.Y.	$ 50,000
Surface Course Bituminous Pavement	500	$ 120	S.Y.	$ 60,000
Testing	250	$ 120	L.F.	$ 30,000
Load, Haul and Dispose of ID-27 Material	3,250	$ 120	Ton	$ 390,000
Construction Total Cost				$ 6,700,000
Total Cost with Contingency (25%)				$ 8,380,000
Overhead and Profit (15%)				$ 1,260,000
Legal and Engineering Costs (20%)				$ 1,680,000
Total Capital Cost				**$ 11,310,000**
Sewer Maintenance **(Not Applicable to Replacement of Existing System)****				
Annual O & M Cost				
Total Present Worth O & M Cost				
***Total Present Worth**				**$ 11,310,000**

*20 years life cycle costs for operation and maintenance with an interest rate of 2.75% for present value calculation (P/A = 15.227)

**Sewer maintenance is current annual cost that the JCMUA pays not and not a new project cost

Development and Evaluation of Alternatives

Table 18: Disinfection Costs with PAA

Treatment	Peak flow (MGD)	Construction Cost	Contingency (25%)	Overhead & Profit (15%)	Legal & Engineering (20%)	Total Capital Cost	Land Use	Annual Worth O&M Costs	Total Present Worth
Anderson	133	$2,800,000	$700,000	$525,000	$700,000	$4,725,000	$269,000	$257,000	$8,910,000
Court	89	$2,300,000	$600,000	$435,000	$580,000	$3,915,000	$202,000	$182,000	$6,890,000
								Total Present Worth Cost	**$15,800,000**

Development and Evaluation of Alternatives

Table 19: Pretreatment and Disinfection Costs with PAA

Treatment	Peak flow (MGD)	Construction Cost	Contingency (25%)	Overhead & Profit (15%)	Legal & Engineering (20%)	Total Capital Cost	Land Use	Annual O&M Costs	Total Present Worth
Anderson	133	$13,000,000	$3,250,000	$2,435,000	$3,250,000	$21,935,000	$538,000	$259,900	$26,430,000
Court	89	$12,000,000	$3,025,000	$2,255,000	$3,005,000	$20,285,000	$404,000	$184,800	$23,500,000
									$49,930,000

Development and Evaluation of Alternatives
Table 20: System-wide Separation (0 overflow)

Description	Estimated Quantities	Unit Cost	Units	Total
System-wide Separation (0 overflow)				
Sewer Separation Total Construction Cost	130,000	$ 2,100	L.F.	$ 278,730,000
Total Cost with Contingency (25%)				$ 348,410,000
Overhead and Profit (15%)				$ 52,260,000
Legal and Engineering Costs (20%)				$ 69,680,000
Total Capital Cost				**$ 470,350,000**
Annual O & M Cost				$ 5,570,000
***Present Worth O & M Cost**				**$ 84,880,000**
***Total Present Worth**				**$ 555,230,000**

*20 years life cycle costs for operation and maitenance with an interest rate of 2.75% for present value calculation (P/A = 15.227)

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15 Village of Ridgefield Park Development and Evaluation of Alternatives Report (Attached)

MOTT MACDONALD

Development and Evaluation of Alternatives

Village of Ridgefield Park
CSO Long Term Control Plan
NJPDES #: NJ0109118

July 1, 2019

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Development and Evaluation of Alternatives

Village of Ridgefield Park
CSO Long Term Control Plan
NJPDES #: NJ0109118

July 1, 2019

Issue and revision record

Revision	Date	Originator	Checker	Approver	Description
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Information class: Standard

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List of Abbreviations and Acronyms

BCMP – Baseline Compliance Monitoring Program
BCUA – Bergen County Utilities Authority
BMP – Best Management Practices
BOD – Biochemical Oxygen Demand
CCI – Construction Cost Index
CMP – Compliance Monitoring Program
COP – Continuous Operating Post
CP – Control Program
CSO – Combined Sewer Overflow
CSS – Combined Sewer Systems
CWA – Clean Water Act
CWA – Clean Water Act
DEAR – Development and Evaluation of Alternatives Report
DEFM – Demographic and Employment Forecast Model
DPW – Department of Public Works
DWF – Dry Weather Flow
EDP – Effective Date of Permit
ENR – Engineering News Record
FOG – Fats Oil and Grease
GI – Green Infrastructure
GP – General Permit
HSG – Hydrologic Soil Group
I/I – Infiltration/Inflow
IDDE – Illicit Discharge Detection and Elimination
LTCP – Long Term Control Plan
MG – Million Gallons
MGD – Million Gallons per Day
MHW – Mean High Water
MS4 – Municipal Separate Storm Sewer System
N.J.A.C – New Jersey Administrative Code
N.J.S.A. – Jew Jersey Statutes Annotated
NJDEP – New Jersey Department of Environmental Protection
NJPDES – New Jersey Pollutant Discharge Elimination System
NJTPA – North jersey Transportation Authority
NMC – Nine Minimum Controls
NOAA – National Oceanic and Atmospheric Administration
PAA – Peracetic Acid
PVSC – Passaic Valley Sewerage Commission
PWQM – Pathogen Water Quality Model
QAPP – Quality Assurance Project Plan
ROSI – Recreation and Open Space Inventory
RTC- Real Time Control
SCD – Soil Conservation District
SCSO – Supplemental CSO
SIU – Significant Indirect Users
TAZ – Traffic Analysis Zone
TBM – Tunnel Boring Machine
TGM – Technical Guidance Manual
TMDL – Total Maximum Daily Loads
TSS – Total Suspended Solids
TWA – Treatment Works Approval
TY – Typical Year
USACE – United States Army Corps of Engineers
USCG – United States Coast Guard

USEPA – United States Environmental Protection Agency
USFWS – United States Fish and Wildlife Service
UV – Ultra Violet
WQS – Water Quality Standards
WRRF – Water Resource Recovery Facility

Certification(s)

Certification

Title: **Ridgefield Park Development and Evaluation of Alternatives Report**

NJPDES Number NJ0109118

"I certify under penalty of law that this document and all attachments were prepared under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons who manage the system, or those persons directly responsible for gathering the information, the information submitted is, to the best of my knowledge and belief, true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for purposely, knowingly, recklessly, or negligently submitting false information."

Permitee: Alan O'Grady 6/19/19

Alan O'Grady Date

Superintendent, Ridgefield Park Village

Certification

Title: **Ridgefield Park Development and Evaluation of Alternatives Report**

NJPDES Number NJ0109118

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Permitee: John Dening 6/25/2019

John Dening, NJ P.E. #45517 Date

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Executive summary

This Development and Evaluation of Alternatives Report (DEAR) has been prepared in partial fulfilment of the Village of Ridgefield Park's obligations under their individual Combined Sewer Management Permit, permit number NJ0109118, issued March 12, 2015, with an effective date of July 1, 2015, and minor revisions issued October 9, 2015. Specifically, this report addresses the requirement of Part IV.D.3.b.v, as per the detail provided in Part IV.G.4. This report is being developed cooperatively with the Bergen County Utilities Authority (BCUA) Combined Sewer Overflow (CSO) Group. Reference are also made to prior reports submitted by Ridgefield Park or submitted by the NJ CSO Group on behalf of Ridgefield Park including:

- Combined Sewer System (CSS) Characterization Report
- Public Participation Process Report
- Sensitive Areas Report
- Baseline Compliance Monitoring Program (BCMP) Report

These documents have been approved by the New Jersey Department of Environmental Protection (NJDEP).

Permittee Background

The Village of Ridgefield Park (Village), located in south central Bergen County New Jersey, is bounded by the Overpeck Creek on the east and south, the Hackensack River on the west and Interstate Highway 80 on the north. The Village has a population of about 12,980 people within its municipal boundaries, which encompasses approximately 2 square miles.

The Village has a total of ten (10) Combined Sewer Overflow (CSO) regulating facilities, three of which are owned by BCUA. The regulators are connected to six (6) CSO discharge points, four of these discharge points flow into the Hackensack River, while the remaining two discharge to the estuary portion of Overpeck Creek.

Public Outreach

Public outreach and input are an important component of the LTCP progress, and the project team has endeavored to provide opportunities for public education and awareness, as well as to gain feedback on the CSO control alternatives. Efforts by the overall BCUA CSO Group, of which the Village is a part, are documented in the BCUA Report. This section only covers activities since the Public Participation Process report was last submitted on January 07, 2019, prior activities are documented in that report. Below id a summary of activies specific to The Village, undertaken since the approves Public Participation Progress Report.

- Village of Ridgefield Park Supplemental CSO (SCSO) Team Meetings
- Village Caucus Meeting
- Earth Day Public Outreach Event
- Additional Outreach Efforts

Sensitive Areas

Consistent with the requirements of the National CSO Control Policy, the NJPDES CSO Permits stipulates that the highest priority must be given to controlling overflows to sensitive areas. A thorough assessment was conducted as described in the Identification of Sensitive Areas Report, dated June 2018, submitted by PVSC to the NJDEP, and approval was received on April 8, 2019. The approved report concluded

there were no sensitive areas within Ridgefield Park. Accordingly, the Village's outfalls will be added uniformly with consideration to the overall reductions in systemwide volume and frequency of overflows.

Future Baseline Condition

The Permit requires the permittee to simulate "conditions as they are expected to exist after construction and operation of the chosen alternative(s)" (Part IV.G.4.e). The intent is to reduce the risk that foreseeable changes in the community and sewer system will reduce the effectiveness of the proposed LTCP facilities. To address this, an evaluation of anticipated changes to the Village's population and potential changes to sewer flows was undertaken. Discussions were also held with the Village to document planned changes to the sewer system. It has been assumed that the alternatives that are selected through the LTCP process will be constructed and implemented over a 30-year period. As such, the year 2050 has been selected as the future baseline condition. A comparison of the 2015 and 2050 baselines is provided below in Table 1, there was no meaningful chnage to overflow volume or frequency.

Table 1: Comparison of 2015 Baseline and 2050 Future Baseline

Outfall	Baseline 2015			Baseline 2050			Change		
	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)
001A	19	6.3	74	19	6.4	74	0	0.0	0
002A	13	0.4	11	13	0.4	11	0	0.0	0
003A	45	15.3	300	45	15.3	300	0	0.0	0
004A	53	24.1	614	53	24.1	614	0	0.0	0
005A	25	3.6	121	25	3.6	121	0	0.0	0
006A	12	0.5	39	12	0.5	39	0	0.0	0
Total	53	50.3	--	53	50.3	--	0	0.0	--

Screening of CSO Technologies

A two-tiered approach was applied to the development of alternatives, starting with a screening analysis followed by an evaluation of the remaining CSO control alternatives. The intent is to give adequate attention to the breadth of alternatives available, but to limit the list of alternatives evaluated to a reasonable amount. This is consistent with Chapter 3 of EPA's Guidance for Long Term Control Plans.

The first step of the screening process was to identify the breadth of alternatives which can then narrowed down to alternatives appropriate for the evaluation process. If necessary, a representative technology to apply to the evaluation is identified. A comprehensive list of CSO control alternatives was prepared by the NJCSO Group.

The screening took place on several levels. In some cases, a general category was screened in or out based on its applicability to the Village. If the general category of technologies was applicable as were many sub-categories, the screening reduced the sub-categories to a reasonable number of representative sub-categories.

The screening was based on the requirements to "evaluate the practical and technical feasibility of the proposed CSO control alternative(s)" (Part IV.G.4.e) to determine if the alternative will proceed to a more detailed evaluation in Section D. This above requirement introduced three concepts that were addressed for each technology:

- Evaluate
- Practical
- Technical Feasibility

Siting of CSO Facilities

Preliminary siting issues is listed in USEPA's Combined Sewer Overflow – Guidance for Long Term Control plans (EPA 832-B-95-002 September 1995) as a screening mechanism and recommends the evaluation of the following:

- Availability of sufficient space for the facility on the site
- Distance of the site from CSO regulator(s) or outfall(s) that will eb controlled
- Environmental, political, or institutional issues related to locating the facility on the site

The Village of Ridgefield Park was first analyzed using the following publicly available GIS information:

- Aerial photography
- Land Use / Land Cover
- Parcel data, including vacant land, land ownership, and property value information
- Open Space / Green Acres
- Soil Type
- Topography
- Known Contaminated Sites
- Brownfields

Potential sites were identified as were the constraints on each site. Some sites were eliminated from consideration due to the suitability for siting CSO control facilities.

Performance Objectives

The magnitude of the facilities in terms of CSO volume managed is the primary driver of both its cost and effectiveness. Accordingly, a procedure was developed to achieve the desired control objectives, in this case limiting the overflows to 0, 4, 8, 12 or 20 during the Typical Year. Since the permit requires the levels of control to be established based on the hydraulically connected system it is not adequate merely to achieve the desired number of overflows at each individual outfall, or within Ridgefield Park. Prior to the evaluation it was necessary to determine for the BCUA system what storm events must be controlled for each level of control. Since the LTCP may incorporate a mix of volume-based controls (storage) as well as peak flow-based control (treatment) the same sets of storms were established for either control methodology.

Control Programs

Six (6) Control Programs were developed, each is discussed in greater detail in Section 7.4.

- Control Program 1 – Elimination of CSO 006A
- Control Program 2 – Consolidated Tank Storage
- Control Program 3 – Consolidated Tunnel Storage
- Control Program 4 – Consolidated End of Pipe Treatment
- Control Program 5 – Sewer Separation
- Control Program 6 – Green Infrastructure

Evaluation

Each alternative was implemented in the approved InfoWorks ICM 2050 baseline model and the modeled facilities scaled to achieve each of the performance objectives for the Typical Year rainfall. The exception was green infrastructure which was implemented to address 2.5%, 5%, 7.5%, 10% and 15% of the modeled directly connected impervious areas. 20-year net present work costs were generated for each alternative using capital costs and operations and maintenance costs. For comparison purposes each alternative was normalized by the cost to remove on gallon of CSO during the Typical Year. Results are summarized below in Table 2 through Table 7.

Table 2: 20-Year net present worth for all control programs

Control Program	NPW Summary - Overflows per Year ($M)				
Level of Control	0	4	8	12	20
1) Eliminate Outfall 006	NA	NA	NA	NA	NA
2) Storage (Consolidated)	$84	$54	$52	$47	$34
3) Tunnel	$118	$99	$99	$92	$86
4) Treatment (Consolidated)	$87	$77	$77	$77	$60
5) Sewer Separation	$193	NA	NA	NA	NA
	NPW Summary - % of Impervious Area Managed ($M)				
	2.50%	5%	7.50%	10%	
6) Green Infrastructure	$2.7	$6	$9	$12	

Table 3: Summary of CSO Volumes for Typical Year

Control Program	2015 Baseline (MG)	Level of Control - Overflows during Typical Year (MG)				
		0	4	8	12	20
1. Eliminate CSO-006A	50.3	NA	NA	NA	NA	NA
2. Consolidated Tank Storage	50.3	0.0	5.7	5.8	9.7	21.5
3. Tunnel	50.3	0.0	4.7	4.7	7.9	11.4
4. Consoldiated End of Pipe Treatment	50.3	0.0	0.2	0.2	0.2	3.0
5. Sewer Separation	50.3	0.0	NA	NA	NA	NA
% Impervious to GI		2.5%	5%	7.5%	10%	
6. Green Infrastructure	50.3	49.9	49.4	48.9	48.3	

Table 4: Summary of CSO Volume Reductions for Typical Year

Control Program	2015 Baseline (MG)	Level of Control - Overflows during Typical Year (MG)				
		0	4	8	12	20
1. Eliminate CSO-006A	50.3	NA	NA	NA	NA	NA
2. Consolidated Tank Storage	50.3	50.3	44.5	44.4	40.6	28.7
3. Tunnel	50.3	50.3	45.6	45.6	42.4	38.8
4. Consoldiated End of Pipe Treatment	50.3	50.3	50.0	50.0	50.0	47.2
5. Sewer Separation	50.3	50.3	NA	NA	NA	NA
% Impervious to GI		2.5%	5%	7.5%	10%	
6. Green Infrastructure	50.3	0.3	0.8	1.4	2.0	

Table 5: Summary of Frequency of Overflows for Typical Year

Control Program	2015 Baseline	Level of Control - Overflows during Typical Year				
		0	4	8	12	20
1. Eliminate CSO-006A	53	NA	NA	NA	NA	NA
2. Consolidated Tank Storage	53	0	4	4	10	20
3. Tunnel	53	0	4	4	7	10
4. Consoldiated End of Pipe Treatment	53	0	1	1	2	10
5. Sewer Separation	53	0	NA	NA	NA	NA
% Impervious to GI		2.5%	5%	7.5%	10%	
6. Green Infrastructure	53	53	53	53	53	

Table 6: Summary of Percent Capture Achieved by Each Control Program

Control Program	2015 Baseline	Level of Control - Overflows during Typical Year				
		0	4	8	12	20
1. Eliminate CSO-006A	69.5%	NA	NA	NA	NA	NA
2. Consolidated Tank Storage	69.5%	100.0%	96.5%	96.5%	94.1%	86.9%
3. Tunnel	69.5%	100.0%	97.2%	97.2%	95.2%	93.1%
4. Consoldiated End of Pipe Treatment	69.5%	100.0%	99.9%	99.9%	99.9%	98.2%
5. Sewer Separation	69.5%	100.0%	NA	NA	NA	NA
% Impervious to GI		2.5%	5%	7.5%	10%	
6. Green Infrastructure	69.5%	69.7%	70.0%	70.3%	70.7%	

Table 7: Net present worth costs normalized by gallon of CSO reduction

Control Program	Cost per Gallon Volume CSO Reduction ($/gal)				
Level of Control	0	4	8	12	20
1) Eliminate Outfall 006	NA	NA	NA	NA	NA
2) Storage (Consolidated)	$1.7	$1.2	$1.2	$1.1	$1.2
3) Tunnel	$2.4	$2.2	$2.2	$2.2	$2.2
4) Treatment (Consolidated)	$1.7	$1.5	$1.5	$1.5	$1.3
5) Sewer Separation	$3.8	NA	NA	NA	NA
	Volume Reduction for Impervious Area Managed (MG)				
	2.50%	5%	7.50%	10%	
6) Green Infrastructure	$9.1	$7.2	$6.3	$5.8	

Each alternative was ranked on the six categories below, and the results are summarized in Table 8:

- Cost
- CSO Reduction
- CSO Frequency Reduction
- Institutional Issues
- Implementability
- Public Acceptance

Table 8: Summary of Control Program Rankings

Control Program	Cost	CSO Volume Reduction	CSO Frequency Reduction	Institutional Issues	Implement-ability	Public Acceptance	Weighted Score
1. Eliminate CSO-006A	NA	NA	NA	NA	NA	NA	NA
2. Consolidated Tank Storage	4	5	5	4	3	3	4.0
3. Tunnel	3	5	5	4	2	2	3.5
4. Consoldiated End of Pipe Treatment	4	5	5	2	3	2	3.6
5. Sewer Separation	2	5	5	3	2	2	3.1
6. Green Infrastructure	1	1	1	5	4	5	2.7
Weighting	25%	15%	15%	15%	15%	15%	100%

1 Introduction

The Village of Ridgefield Park (Village), located in south central Bergen County New Jersey, is bounded by the Overpeck Creek on the east and south, the Hackensack River on the west and Interstate Highway 80 on the north, see Figure 1.1. The Village was incorporated in 1892 having previously been the northwestern portion of Ridgefield Township. The Village developed primarily as a residential municipality, with one and two-family homes, built on relatively small lots. By 1920 the population had increased to 8,600 residents with continued growth until 1970 when the population peaked at approximately 14,000. As of the 2010 census the population was 12,729 and according to the 2015 mid-census estimate, the population is 12,976.

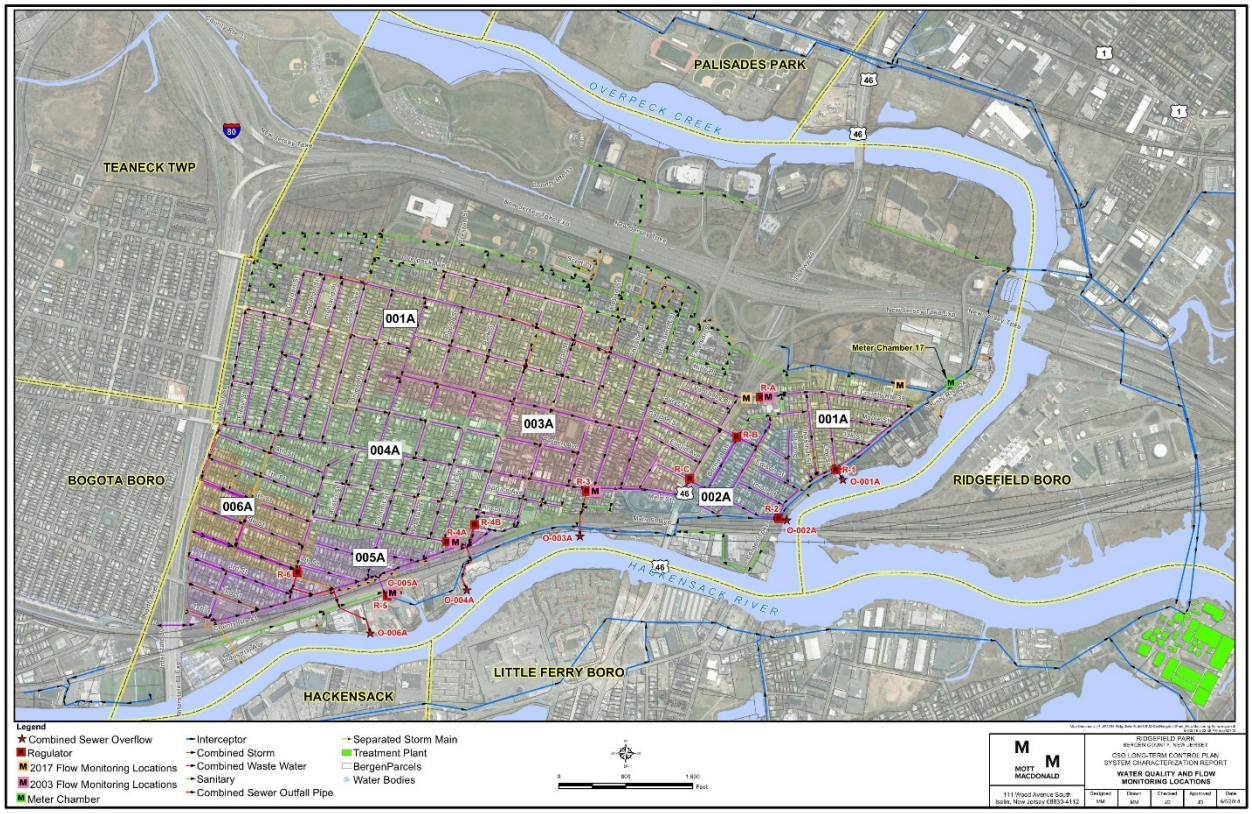


Figure 1-1: Ridgefield Park Location Map

The Village of Ridgefield Park owns and operates the combined sewer system within the Village limits. Wastewater from the Village is transported through the BCUA's Ridgefield Park Intercepting Sewers, under Overpeck Creek before discharging into the BCUA Overpeck Interceptor/Relief Sewer, which flows to the BCUA Water Resource Recovery Facility (WRRF) in Little Ferry, approximately two miles away, see Figure 1-1. The Village has a total of ten (10) Combined Sewer Overflow (CSO) regulating facilities, three of which are owned by BCUA. The regulators are connected to six (6) CSO discharge points, four of these discharge points flow into the Hackensack River, while the remaining two discharge to the estuary portion of Overpeck Creek. Each outfall is equipped with a solids and floatables control facility.

This Development and Evaluation of Alternatives Report has been prepared on behalf of the Village of Ridgefield Park in coordination with the Bergen County Utilities Authority CSO Group (the BCUA CSO Group consists of the BCUA, Village of Ridgefield Park, Fort Lee Borough and City of Hackensack) to meet certain conditions (Part IV D.3.b.v) of the New Jersey Pollutant Discharge Elimination System

(NJPDES) individual permit actions issued by the New Jersey Department of Environmental Protection (NJDEP) for Combined Sewer Overflow control, with an effective date of July 1, 2015 (referred herein as the NJPDES CSO Permit or the Permit). As permittees of a hydraulically connected system, the Village and the BCUA CSO Group are cooperating and collaborating on the development of a Long Term Control Plan (LTCP) for CSO control per the permit conditions. The Village and the BCUA CSO Group are communicating regularly, sharing information, and exchanging hydraulic models for appropriate characterization, monitoring, and integrated modeling of the hydraulically connected system to effectively develop and evaluate the alternatives presented in this report.

1.1 Regulatory Background and Project History

1.1.1 Permittee Background

The Village of Ridgefield Park provides wastewater and stormwater collection and conveyance services to about 12,980 people within its municipal boundaries, which encompasses approximately 2 square miles. Their collection and conveyance systems include an extensive network of sewers, manholes, catch basins, overflow control facilities, and drainage conduits, totaling around 105,000 LF, or approximately 20 miles of pipe. The Village of Ridgefield Park does not own or operate any wastewater treatment plant facilities outside of the solids/floatables control facilities on the outfalls; wastewater flows are conveyed by gravity via the Ridgefield Park Branch Trunk Collector Sewer, then to the Overpeck Creek Trunk/Relief Sewer, which connects to the BCUA Water Resource Recovery Facility (WRRF) located in Little Ferry, NJ. The Village owned sewer system assets are operated and maintained through the Public Works Department.

Much of the Village is served by a combined sewer system (CSS) that collects and conveys both sanitary and stormwater flows in the same conduit. The combined sewers are prevalent throughout the northern, western, and southern sections of the Village, coinciding with its historical residential, industrial, and commercial development. The existing combined system includes regulators and diversion structures, solids and floatables control facilities, interceptor connections, and outfalls at various locations. Stormwater has been partially separated from the combined sewer system, especially along Teaneck Avenue. The area east of the NJ Turnpike, and immediately west of it, are separately sewered.

All dry weather sewage from the Village owned sewer system is conveyed to and treated at the BCUA WRRF. BCUA is a regional wastewater conveyance and treatment public entity established by the Bergen County Board of Chosen Freeholders in 1947 and currently serves forty-seven municipalities and approximately 565,000 residents in eastern Bergen County. The treatment plant is permitted for an average daily flow of 94 million gallons per day (MGD), but can process additional quantities of flow during wet weather events.

In 2015 the New Jersey Department of Environmental Protection revoked prior authorizations related to combined sewer overflows under NJPDES Master General Permit No. NJ0105023 and issued individual permits to individual municipalities and commissions/authorities that own or operate facilities that control, transport, or treat wastewater flows from combined sewer systems. CSO Discharges from the Village of Ridgefield Park's 6 designated CSO outfalls are authorized and regulated by NJPDES Permit No. NJ0109118. While the BCUA does not own or operate CSO outfalls, the downstream portion of the BCUA trunk sewer system receives and conveys combined sewage from the Village, the Borough of Fort Lee, and the City of Hackensack, whose systems are hydraulically connected. As such, the NJDEP revoked and reissued the BCUA's individual Category "A" Permit No. NJ0020028 to incorporate and update CSO NJPDES permit requirements as part of the recent permit actions.

In the current NJPDES Individual Permits, the NJDEP has mandated that the permittees undertake steps as needed for the development of a CSO Long Term Control Plan incorporating permit conditions that closely reflect the control standards and goals of the CSO Control Policy established by United States Environmental Protection Agency (EPA). The individual permits also encourage permittees to cooperate in the development of a plan, that incorporates all permittees, within a hydraulically connected system, into a single, Regional CSO LTCP. The Village of Ridgefield Park and the rest of the BCUA CSO Group have joined together in a cooperative manner to develop a regional plan.

A CSO LTCP involves a comprehensive study of the hydraulically connected sewer system and the evaluation of alternatives for reducing CSO impacts to receiving waters. It investigates the hydrologic and hydraulic relationships between precipitation, conveyance, treatment capacity, and overflows and includes a feasibility study to evaluate the scope, costs, and performance of possible control alternatives for reducing the frequency and volume of CSO discharges.

The EPA CSO Control Policy and the individual NJPDES CSO Permits describe nine elements or requirements for the development of a CSO Long Term Control Plan:

1. Characterization, monitoring, and modeling of the combined sewer systems to provide a thorough understanding of the hydraulically connected system, its response to various precipitation events, the characteristics of the overflows, and the water quality impacts that result from the CSOs;
2. A public participation process that actively involves the affected public in the decision-making process to select long term CSO controls;
3. Consideration of sensitive areas in identifying the highest priority for controlling overflows;
4. Evaluation of alternatives that considers a reasonable range of CSO control options that provide a level of control, presumed (per the criteria given in the Policy and Permit) or demonstrated to meet the water quality-based requirements of the Clean Water Act (CWA);
5. Cost/performance considerations to demonstrate the relationships among a comprehensive set of reasonable control alternatives;
6. An Operational plan that incorporates revisions to the operation and maintenance program necessary after approval of the LTCP, to incorporate its associated CSO controls;
7. Maximizing treatment at the existing publicly owned treatment works (POTW) treatment plant during and after each precipitation event so that such flows receive treatment to the greatest extent practicable, including utilizing existing tankage for storage, while still meeting permit limits;
8. An implementation schedule addressing the construction and financing of proposed CSO controls; and
9. A post-construction compliance monitoring program adequate to verify compliance with water quality-based CWA requirements and designated uses, as well as to ascertain the effectiveness of implemented CSO controls.

The NJPDES Individual Permit divided the above requirements into three sequential steps, providing an orderly progression for the development of the LTCP. The tasks undertaken, and the documents submitted under each step, per the specified schedule, are as follows:

- *Step 1 incorporates the characterization, monitoring, and modeling element and components of the public participation process, consideration of sensitive areas, and compliance monitoring program. It is further divided into the following submittal requirements and schedule:*
  - Permittees were required to submit a System Characterization Work Plan within 6 months from the effective date of the permit (EDP), which corresponded to a due date of January 1, 2016. This document was submitted on time and the approved work plan served as the basis for the subsequent characterization, monitoring, and modeling efforts.

- Permittees were required to submit a System Characterization Report within 36 months of the EDP, which equates to a due date of July 1, 2018. This document was submitted on time and served as the basis for the subsequent development and evaluation of alternatives efforts.
- Permittees were required to submit a Public Participation Process Report within 36 months from the EDP (i.e., July 1, 2018). This documents waso submitted on time and contributed to the development and evaluation of alternatives efforts.
- Permittees were required to submit a Consideration of Sensitive Areas Information document within 36 months from the EDP (i.e., July 1, 2018). This documents waso submitted on time and contributed to the development and evaluation of alternatives efforts.
- Although listed separately from the steps in the permit under the LTCP Submittal Requirements, permittees were also required to submit a baseline Compliance Monitoring Program (CMP) Work Plan by January 1, 2016 and were then required to submit a baseline CMP Report and data within 36 months of the EDP, or again July 1, 2018. The Village and other members of the BCUA CSO Group collaborated with the NJ CSO Group, a coalition of New Jersey CSO permittees, to satisfy these permit conditions through a regional ambient water quality sampling and testing program and pathogen water quality modeling.

- *Under Step 2, permittees are required to submit a Development and Evaluation of Alternatives Report within 48 months from the EDP, or a due date of July 1, 2019. This step involves evaluating a broad range of control alternatives to meet CWA requirements and water quality standards (WQS) using either the "Presumption" Approach or "Demonstration" Approach and the corresponding conditions prescribed in the permit. Maximizing treatment at the existing WRRF treatment plant and cost and performance considerations also need to be addressed in Step 2. This report is being submitted in fulfillment of this permit condition.*

- *Under Step 3, permittees are required to submit a Selection and Implementation of Alternatives Report that incorporates the final plan selection and implementation schedule, for the construction and financing of proposed CSO controls. A proposed operational plan revision schedule and a post-construction compliance monitoring program also should be addressed. This submittal is due within 59 months from the EDP, which corresponds to a due date of June 1, 2020.*

This report has been prepared to fulfill the requirement for the Development and Evaluation of Alternatives Report as outlined in the individual NJPDES Permit No. NJ0109118 for the Village of Ridgefield Park, specifically Part IV.D.3.b.v and as detailed in Part IV G.4.

The NJPDES Individual Permit further stipulates that each permittee must work cooperatively with other permittees and municipalities, in a hydraulically connected system, such that the LTCP is developed consistently based on the same data, characterization, analysis methods, and results. The Village and the BCUA CSO Group have actively collaborated on the previous elements of the LTCP, such as the System Characterization Report, as well as this Development and Evaluation of Alternatives Report.

The Village and the BCUA CSO Group are also involved as members of the regional assembly of NJPDES Individual permittees referred to as the NJ CSO Group. Members of this group consist of municipalities, sewerage authorities, and other entities that have been issued a NJPDES Individual permit and have agreed to work jointly in addressing common and overlapping areas for permit compliance. On behalf of the member permittees, the NJ CSO Group, with the Passaic Valley Sewerage Commission (PVSC) serving as the managing member, is providing shared services for these common elements of the permits to avoid costly and inefficient duplication of efforts. These shared services encompass certain water quality monitoring and modeling work, such as ambient in-stream sampling and testing, baseline compliance monitoring reporting, and pathogen receiving water quality modeling; production of CSO outfall identification signs; setup and operation of a CSO notification system website; and coordination

with regulatory agencies on common issues for permit compliance. This regional partnership also assists in sharing information and gathering feedback on techniques for public involvement and CSO controls.

1.2 Summary of System Characterization and Sensitive Area Reports

In the CSO LTCP process, an extensive understanding of the combined sewer system is critical to the development of a computer model capable of accurately modeling the collection system and characterizing its response to precipitation events. As such, a thorough record review and updated sewer system description, inventory, assessment, and hydrologic and hydraulic model were developed and compiled in previous steps of the CSO long term control planning process. The following provides a summary of the final Combined Sewer System (CSS) Characterization report that was re-submitted to NJDEP on February 18, 2019, and approved on March 11, 2019.

1.2.1 Sewer System Description

The Village's wastewater and stormwater collection and conveyance system consists of a complex network of intercepting sewers, sewer mains, manholes, catch basins, overflow control facilities, outfalls, and stormwater sewers. Much of the Village has combined sewers that collect and convey sanitary and stormwater flows in the same conduit, but in certain areas stormwater runoff is conveyed by separate storm sewers. The breakdown of sewers with the Village is noted in Table 1-1.

Table 1-1: Sewer System Summary

Drainage Basin	Combined sewers	Storm (to CSS)	Separate Sanitary Sewers	Separate Storm Sewers
1	85.10%	6.20%	0.00%	8.70%
2	90.90%	6.70%	0.00%	2.40%
3	91.00%	9.00%	0.00%	0.00%
4	91.50%	5.80%	0.00%	2.70%
5	66.50%	13.40%	0.00%	20.10%
6	72.40%	2.20%	0.00%	25.40%
Average	85.37%	6.90%	0.00%	7.73%

*Average weighted by sewer length

The combined sewer system makes up a little over 85% of the total sewer length (excluding building laterals) within the Village, while approximately 6.9% are storm sewers that connect to the CSS and 7.7% are classified as separate storm sewers. There are no pumping stations owned or operated by the Village, nor are there any sewers tributary to the Village's CSS from other municipalities.

1.2.2 Land Use

The Village of Ridgefield Park encompasses a total area of 1,230 acres, of which 490 acres are serviced by the combined sewer system and 740 acres are serviced by separate storm sewers. Land use within the Village is primarily residential, with limited commercial, industrial and open space areas, combined and separately sewered areas. The Village also has a disproportionate amount of transportation facilities as it traversed by three major highways and several rail lines. The land uses within the Village for 2012 are depicted below. Current land use is provided in Figure 1-2 and Impervious Cover in Figure 1-3.

There has been less than 1% change in the land use and impervious cover between 2007 to 2012, nor has there been any significant development from 2012 to the present. There has been no significant change in land use or impervious areas within the Village for over 10 years.

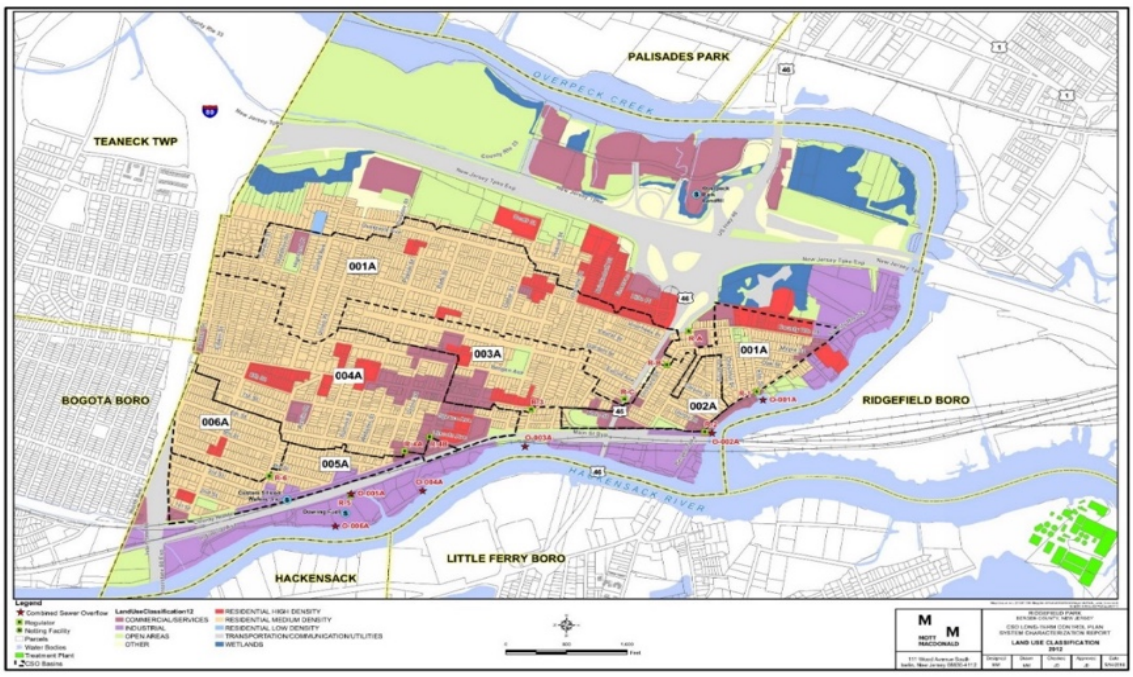


Figure 1-2: Land Use Map

The population of Ridgefield Park was 12,873 under the 2000 Census and 12,729 in the 2010 Census. Population estimates, made for 2015, estimated that the population of the Village increased to 12,976. Population growth in the Village has been essentially static with a less than 1% change in net population between 2000 and 2015.

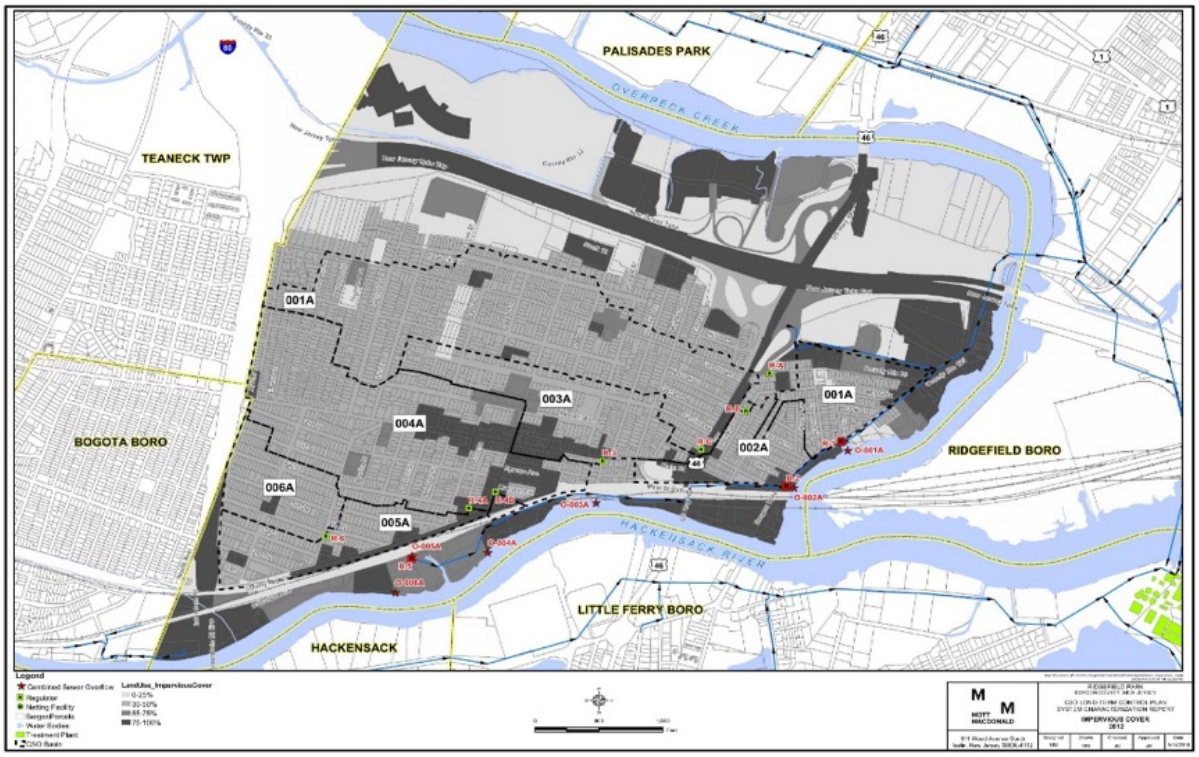


Figure 1-3: Impervious Cover Map

1.2.3 Significant Indirect Users

During the characterization three significant indirect users (SIU) were reported within Ridgefield Park, which are regulated by the BCUA:

- Overpeck Park Landfill; Facility #1074, 50 Challenger Road, Landfill Leachate – This site reportedly discharges approximately 9,000 gpd by pumping station to the separate sanitary sewer in Leonia and thus while located in Ridgefield Park is not tributary to its sewer system.
- Custom Silicon Wafers Inc; Facility #1112, 80 Rail Road Avenue, Manufacture of polished silicon wafers for use in electronics – This site discharges approximately 250 gpd to the combined sewer system.
- Dowling Fuel, Facility #1118, 100 Industrial Avenue, Groundwater, dewatering and remediation – This site discharges approximately 500 gpd to the combined sewer system. – This was a temporary discharge that was discontinued in December of 2017

The locations of significant indirect users (SIUs) relative to the CSO Basins are indicated in Figure 1-4. The only remaining SIU tributary to the combined sewer system is Custom Silicon Wafers, Inc. The potential impacts to water quality from this SIU are discussed in the BCUA report. Sampling data indicates its potential impact to water quality is minimal, and that it does not merit additional requirements to prevent discharges during wet weather.

Figure 1-4: Significant Indirect Users

1.2.3.1 Water Quality and Hydraulic Monitoring

The Village of Ridgefield Park developed an InfoWorksICM (hydrologic and hydraulic) model of its combined sewer system, as a predictive tool to evaluate system improvements. To generate adequate data on the actual physical characteristics of the Village's combined sewer, the Village performed a GPS survey, in addition to a precipitation and sewer flow monitoring program appropriate for the extent and complexity of the combined sewer system. Most system monitoring was conducted in 2003 and 2004, but additional monitoring was conducted in 2016. Additional information on the hydrologic and hydraulic monitoring conducted can be found in "Village of Ridgefield Park Sewer System Characterization Report", revised February 11, 2019 as approved by the NJDEP in their March 11, 2019 letter.

1.2.3.2 Sewer System Modeling

The sewer system model was developed following the procedures outlined in the System Characterization Quality Assurance Project Plan (QAPP) using the InfoWorksICM computer program. InfoWorksICM is a widely used software package capable of modeling overland rainfall and runoff responses and accurately calculating hydraulic pipe flows through collection system conveyance links. The software allows an extensive portion of the physical system to be represented, along with the complex network of interconnections and flow conditions occurring within it.

The existing conditions model incorporates all sewers 18" and larger in diameter, and a substantial number of smaller sewers. All trunk sewers, overflow control structures, and outfall pipes have been included in the model, along with various sewage and stormwater facilities. This broad model geometry facilitates simulating and routing of dry weather and wet weather flow components throughout the

combined sewer collection system. The Village of Ridgefield Park model was incorporated into the InfoWorksICM model for the BCUA district-wide model that includes the entire BCUA transport system plus the other combined sewer systems in Hackensack and Fort Lee. Additional information on the model development can be found in "Village of Ridgefield Park Sewer System Characterization Report", revised February 11, 2019, as approved by the NJDEP in their March 11, 2019, letter.

1.2.3.3 Receiving Water Quality Monitoring and Modeling

As part of the NJ CSO Group collaboration, a field sampling and analytical testing program for existing ambient pathogen water quality conditions in the participating CSO permittees receiving waters was implemented. The NJ CSO Group Compliance Monitoring Program Report (CMP Report), dated June 2018, was submitted to the NJDEP by PVSC, as the lead organization of the NJ CSO Group. The CMP Report provided a full narrative of the implemented Baseline Compliance Monitoring Program (BCMP), including the program description; the field sampling and the field and laboratory analytical methods used; the data quality objectives; an evaluation of data completeness, precision, and representativeness; and presentations and discussion of the program results. The intent of the program was to generate adequate data to establish existing ambient water quality conditions for pathogens in the CSO receiving waters and to update, calibrate and validate a pathogen water quality model of the receiving waterbodies. The BCMP report was submitted to the NJDEP and was approved on March 1, 2019.

The BCMP involved baseline, source, and wet weather event sampling of waterbodies throughout the region, with three sampling locations situated at or near the Village of Ridgefield Park, including sampling stations maintained by the New Jersey Harbor Discharges Group (NJHDG). Baseline sampling was performed twice a month in May and June; weekly in July, August, and September; and monthly from October through April. Source sampling occurred at the same time as baseline sampling, but targeted major influent streams within the study area to establish non-CSO loadings. Event sampling was timed to coincided with rainfall, to capture 3 discrete wet-weather events, and was limited to select sampling locations.

The CMP Report organizes the baseline, source, and event sampling data by waterbody grouping, station number, and specific waterbody. A total of 23 baseline and source events were completed from April 2016 through March 2017, while the wet weather event sampling was completed across 4 sampling dates in June 2016, January 2017, and April 2017. All samples collected were analyzed for fecal coliform and enterococcus and samples from freshwater locations were also analyzed for E. coli. During field sampling, field measurements were also made for: temperature, salinity, dissolved oxygen, light penetration (secchi depth), and turbidity. Depending on the sampling location, samples were collected at either 1 or 2 depths. For event sampling, locations were typically sampled twice per day for 3 days.

Using information collected as part of the baseline monitoring, the NJ CSO Group developed a Pathogen Water Quality Model (PWQM) for consideration in the CSO LTCP alternatives selection and Compliance Monitoring Program. The PWQM was created to supplement ambient in-stream pathogen monitoring under the baseline CMP and facilitate the evaluation of CSO control approaches and alternatives. Water quality data collected under the baseline CMP and collection system characterization work was being used to calibrate and validate the PWQM. It is a computational model of the receiving waters that may be used to simulate bacteria concentrations in the waters of the New York-New Jersey Harbor complex under existing conditions and to assess the attainment of applicable water quality standards under potential future conditions. The PWQM will function as a validated tool that reasonably replicates the observed timing, duration, and magnitude of ambient pathogen data in the study area. It is capable of simulating pathogen water quality conditions for varying CSO strategies to assist in assessing the CSO impacts on ambient pathogen water quality.

The sampling and modeling results were documented in the Baseline Compliance Monitoring Program Report, submitted by PVSC on behalf of the NJ CSO Group and was approved by NJDEP on March 1, 2019. It is anticipated that the model will be used to evaluate the overall effectiveness of the Regional CSO LTCP.

1.2.3.4 Characterization of System Performance

The EPA National CSO Control Policy indicates that CSO control alternatives be assessed on a "system-wide, annual average basis". This can be accomplished by selecting a typical hydrologic year and using the precipitation records for the period in a continuous model simulation of the hydraulically connected system. The selection of a typical hydrologic record serves to provide a representative and unbiased prediction of average design rainfall conditions that incorporates the variability observed in the historical records. In conjunction with the NJ CSO Group, local historical rainfall data and storm patterns were analyzed and calendar year 2004 was selected as the Typical Year (TY) hydrologic dataset for the LTCP efforts by the NJ CSO Group permittees. With the submission of the Typical Hydrologic Year Report by PVSC on behalf of the NJ CSO Group, the NJDEP responded in May 2018 that the submitted report addressed all its questions and comments to its satisfaction.

As previously noted, the InfoWorksICM model can be used as a predictive tool to assess the existing sewer system performance under varying rainfall conditions. Accordingly, a model simulation was conducted for the Village of Ridgefield Park combined sewer system using the historic precipitation data (5-minute time series) for 2004 from Newark Airport. The Typical Year model simulation for the current sewer system provides detailed information on the response of each drainage basin within the system to individual rainfall volumes and intensities, and results in an estimated total annual overflow volume from all CSO outfalls of 50.3 million gallons (MG) for the Typical Year. The results from the analysis, including predictions of the number of overflow events, overflow volume, and duration, and the peak flow occurrence by outfall location are summarized in the table below. The durations noted are the predicted total cumulative time of CSO discharge through the year for the indicated outfall location. The system-wide event counts, durations, and peak flows are the predicted maximum values observed across all outfalls for the year.

Since the characterization report, additional field data was collected on the orifice plates in the regulators. The results of these updates are reflected in the summary below in Table 1-2.

Table 1-2: Typical Year Existing Conditions Annual Total CSO Characterization by Outfall

Outfall No.	Outfall Name	Annual Total			Maximum
		No. Overflow Events	Overflow Volume (MG)	Duration (hours)	Peak Flow (MGD)
001A	Bergen Turnpike	19	6.3	74	19.4
002A	Main Street and Bergen Turnpike	13	0.4	11	6.2
003A	Christie Street	45	15.3	300	31.9
004A	Mount Vernon Street	53	24.1	614	56.3
005A	Industrial Avenue	25	3.6	121	5.0
006A	Hackensack Avenue	12	0.5	39	2.6
System-wide Total		**not appl.**	**50.3**	**not appl.**	**not appl.**
System-wide Maximum		**53**	**24.3**	**614**	**56.3**

1.2.4 Sensitive Areas Report Summary

Consistent with the requirements of the National CSO Control Policy, the NJPDES CSO Permits stipulates that the highest priority must be given to controlling overflows to sensitive areas. The permits define sensitive areas as designated Outstanding National Resource Waters; National Marine Sanctuaries; waters with threatened or endangered species and their habitat; waters used for primary contact recreation (including but not limited to bathing beaches); public drinking water intakes or their designated protection areas; and shellfish beds. If a CSO outfall discharges to a sensitive area, the CSO outfall is to be eliminated or relocated wherever physically possible and economically achievable, and where elimination or relocation is not feasible, treatment of the overflow deemed necessary to meet water quality standards must be provided. The implementation schedule for the LTCP must also place the highest priority to controlling CSOs to sensitive areas.

A thorough assessment of the potential need for a higher prioritization of any specific CSO discharge location in the Village of Ridgefield Park due to the presence of sensitive areas was conducted. This work included a detailed investigation of the subject waterbodies performed by the NJ CSO Group on behalf of the participating permittees, as described in the Identification of Sensitive Areas Report, dated June 2018, submitted by PVSC to the NJDEP, and approval was received on April 8, 2019. The approved report concluded there were no sensitive areas within Ridgefield Park.

Additional consideration of the Village of Ridgefield Park setting, incorporating discussions and input from the Supplemental CSO Team, was also made. Based on these examinations it was determined that canoeing is listed as a secondary contact and thus, while the Wanda Canoe Club utilizes Riverside Park in Ridgefield Park (behind the Department of Public Works (DPW) building) it does not constitute a sensitive area under the permit. A review of the proximity of the launch area to surrounding outfalls (006A, 005A, and 004A) shows that each of these outfalls is located around 600 – 800 feet away from the launch area and thus not in the immediate area (within 100 feet) of the launch. In addition, this segment of the Hackensack River is tidal and thus the impact of these outfalls impact both upstream and downstream areas. The Village will keep in mind the public interest in this area in its planning, however the Village consistent with the NJDEP's determination, has determined that no sensitive areas exist within its receiving waters and community and no elevated or accelerated CSO controls for sensitive areas are required.

1.3 Approach to Development and Evaluation of Alternative Controls

The development and evaluation of alternative controls is the initial step in the planning process needed to develop the site-specific projects proposed for implementation under the LTCP. There are a large variety of control methods that could be utilized to reduce or eliminate discharges from the combined sewer system and this report represents the screening process and evaluative methods used to reduce the number of control alternative that will be ultimately evaluated for implementation. It should be noted that the information contained within this report is preliminary and does not represent the Village's CSO Long Term Control Plan. Rather the information contained herein will be used over the next eleven months to develop a more detailed evaluation of the recommended alternatives to choose those that will be ultimately implemented.

1.3.1 Report Goals and Objectives

The development and evaluation of CSO control programs is an essential step in the planning process for identifying specific projects proposed for implementation under the LTCP. There are numerous control methods that could be utilized to reduce or eliminate discharges from the combined sewer system and this report represents the process used to review these various CSO control technologies and develop different control alternatives. These control strategies are evaluated according to their practical and technical feasibility, and potential water quality benefits relative to the requirements of the CWA.

This Development and Evaluation of Alternatives Report (DEAR) incorporates a comprehensive review and analysis of applicable CSO control strategies based on the information gathered and presented in the System Characterization Reports. The Village has developed a thorough understanding of their wastewater collection system, including the system's responses to precipitation events of varying duration ant intensity, and the capacity of the systems to capture and treat flows from the combined sewer system (CSS).The hydrologic and hydraulic model approved by the NJDEP has been used to simulate the system performance under the baseline conditions as well as the system's response for a broad level of controls in the development of CSO control alternatives.

The program objectives addressed herein are:

- Organize the evaluation of controls with an approach that is understandable and consistent with the NJPDES CSO permits and National CSO Control Policy;
- Present a broad range of CSO control strategies to meet the NJPDES permit requirements;
- Identify and review a variety of CSO control technologies within the general categories of source controls, collection system controls, storage and treatment technologies;
- Evaluate an appropriate range of CSO control programs for control levels, technical merit, ability to be implemented and costs
- To evaluate the full range of alternatives with respect to meeting water quality standards and protecting designated uses.
- Present cost/performance considerations; and,
- To organize the evaluation of controls in a technical framework and approach that is understandable and consistent with the CSO Control Policy and EPA Guidance.
- Provide an update on the public participation process.

The program goal is to develop a range of control programs that are capable of cost-effectively improving water quality within the impacted receiving waters and that can be further evaluated for implementation. The contents of this report collectively relate to each of these goals and objectives and provides the information necessary for the Village to advance the LTCP process to the Selection and Implementation of Alternatives step.

The USEPA recommends a three-step sequence for the development of alternatives:

1. Definition of water quality objectives.
2. Definition of a range of CSO control goals to meet the CSO component of the water quality goals.
3. Development of alternatives to meet the CSO control goals.

The USEPA expects the long-term CSO control plan to evaluate controls that will ultimately result in compliance with water quality standards (WQS) and under the Presumptive Approach suggests no more than four overflows per year or 85% capture by volume of CSO discharges. However, the solution will also depend on the permittees' ability to fund these projects. Accordingly, it is prudent to evaluate various levels of control including, but not limited to achieving 0, 4, 8, 12 and 20 overflow events per year based on an average (typical) year. Alternatively, the long-term plan could evaluate controls that achieve various levels of capture for treatment. The water quality-based requirements of the Clean Water Act can be met using either the Presumption Approach or the Demonstration Approach in the evaluation of alternatives.

The "Presumption" Approach refers to a program that is presumed to meet WQS using the following criteria for combined sewer flows remaining after the minimum treatment of primary clarification, solids and floatables disposal, and disinfection of effluent, if necessary. The Presumption Approach requires the following:

- No more than an average of four overflow events per year occurs from a hydraulically connected system as the result of a precipitation event. The Department may allow up to two additional overflow events per year.
- Elimination or the capture for treatment of no less than 85% by volume of the combined sewage collected in the CSS during precipitation events on a hydraulically connected system-wide annual average basis.
- Elimination or removal of no less than the mass of the pollutants, identified as causing water quality impairment.

The “Demonstration” Approach refers to a program that uses a receiving water model to meet the water quality-based requirements of the Clean Water Act taking into consideration the following:

- The planned control program is adequate to meet WQS and protect designated uses, unless WQS or uses cannot be met because of natural background conditions or pollution sources other than CSOs.
- The CSO discharges remaining after implementation of the planned control program will not preclude the attainment of WQS or the receiving waters' designated uses or contribute to their impairment.
- The planned control program will provide the maximum pollution reduction benefits reasonably attainable.
- The planned control program is designed to allow cost effective expansion or cost-effective retrofitting if additional controls are subsequently determined to be necessary to meet WQS or designated uses.

This Development and Evaluation of Alternatives Report has been created to provide a comprehensive review and analysis of a range of CSO control strategies to meet the CSO component of the water quality goals. Per the requirements of CSO NJPDES Permit No. NJ0109118, the list of alternatives that have been evaluated includes:

- Green infrastructure
- Increased storage capacity in the collection system
- STP expansion and/or storage at the plant. This includes evaluation of the capacity of the unit processes to determine the amount of CSO discharge reduction that would be achieved by utilizing additional treatment capacity while maintaining compliance with all permit limits.
- Inflow/Infiltration (I/I) reduction to free up storage capacity or conveyance in the sewer system and/or treatment capacity at the STP
- Sewer separation
- Treatment of the CSO discharge
- CSO related bypass of the secondary treatment portion of the STP

This report utilizes the information gathered as part of the System Characterization to develop and evaluate the range of CSO control alternatives listed above. The hydrologic and hydraulic model reviewed by the NJDEP has been used to simulate the existing conditions as well as the response of the system under each of the alternatives after construction and operation, including anticipated changes to the CSS usage and configuration. The CSO control alternatives are evaluated according to their practical and technical feasibility, water quality benefits of constructing and implementing various remedial controls, and combination of such controls and activities.

The program goal is to develop a range of control alternative that are capable of cost-effectively improving water quality within the impact receiving waters, and that could be further evaluated for implementation

under the Village's CSO LTCP. The contents of this report collectively relate to each of these goals and objectives and provides the information necessary for the Village and BCUA to advance the LTCP process to the Selection and Implementation of Alternatives step.

2 General Information

2.1 Public Participation Process Update

2.1.1 Public Participation since Approved Public Participation Process Report

Public outreach and input are an important component of the LTCP progress, and the project team has endeavored to provide opportunities for public education and awareness, as well as to gain feedback on the CSO control alternatives. Below is a summary of activities specific to the Village of Ridgefield Park.

Much of the outreach took place through quarterly meetings of the Village of Ridgefield Park Supplemental CSO (SCSO) Team, and through the individual actions of the members.

Mott MacDonald prepared a presentation on the progress of the alternatives evaluation and presented it to the Village's CSO Supplemental Team on January 23, 2019. The presentation provided and update to the Team on the status of the overall project, alternatives to be considered, and where those alternative controls may be located. A follow up presentation providing more specifics on alternative controls and their possible locations and anticipated costs was presented to the Village SCSO Team on May 28, 2019. To advertise the event the Village posted an announcement on the Village's website and copies of the event flier were distributed to the Village Council, posted in Village Hall and the public library, and distributed at the Village's Earth Day Celebration. Any members of the public that attended were able to hear an overview of the potential CSO control alternatives and presentations by the other communities. The presentation slides for Ridgefield Park were included in the meeting minutes, as were the sign-in sheets, which were posted to the Village website.

To further advance the public involvement a presentation was made at the Village's April 4, 2019 caucus meeting. The meeting included both village official and members of the public. Mott MacDonald presented an overview of combined sewers, the regulatory history and prior work leading up to the current permit. The Village's obligations under the permit were reviewed and the current progress discussed. The anticipated work and overall project schedule were presented and the Village informed of the upcoming decisions they will be required to make.

In addition to the quarterly meetings, the Village also has a Earth Day public event each year at Riverside Park. This year's Earth Day was held on May 11, 2019 at which the Village made arrangements to have information on the combined sewer system and the current project available for interested citizens. A demonstration rain barrel was provided by the BCUA along with additional literature. The event was also used to advertise the May 28, 2019 CSO Supplemental Team Meeting that was used to discuss the results of the development and evaluation of alternatives.

2.1.2 Anticipated Future Public Participation Efforts

In a letter to NJDEP dated May 22, 2019 the Village indicated that the following public outreach activities were anticipated through the submittal of Selection and Implementation of Alternatives Report due June 1, 2020.

The following is a list of activities the Village intends to undertake during the time-period specified.

- Continuation of Supplemental CSO Team meeting on approximately a quarterly basis.
- Continue to seek additional members for the Supplemental CSO Team.
- Present the progress of the project to the Village Council at two (2) meetings.
- Hold two (2) meetings in the evening, open to the public, and in a public building such as the Municipal Building or a school.

- Include an additional article relating to CSOs in the Village's newsletter.
- Present the project to meetings of two (2) local community groups, using suggestions from the supplemental CSO team.
- Publicize the project at the Earth Day celebration in 2020.

It is noted that the above list of activities would represent approximately monthly contact with the public. The above describes the intent of the Village and is not to the exclusion of other outreach activities, nor is a guarantee to conduct every activity on the list, particularly activities that may rely on factors outside of the Village's control.

2.2 Approximated Future Conditions

The model was developed in 2018 and needs to be applicable during the entire planning period. Future conditions for flow projections were based on a thirty-year implementation period and thus should reflect anticipated conditions in 2050.

2.2.1 Projections of Population Growth

The Permit requires the permittee to simulate "conditions as they are expected to exist after construction and operation of the chosen alternative(s)" (Part IV.G.4.e). The intent is to reduce the risk that foreseeable changes in the community and sewer system will reduce the effectiveness of the proposed LTCP facilities. To address this, an evaluation of anticipated changes to the Village's population and potential changes to sewer flows was undertaken. Discussions were also held with the Village to document planned changes to the sewer system. It has been assumed that the alternatives that are selected through the LTCP process will be constructed and implemented over a 30-year period. As such, the year 2050 has been selected as the future baseline condition.

It is acknowledged that sea levels have been rising and are expected to continue to rise over the life of the project and beyond, however, the rate of change is uncertain. To overflow, the water level in the combined sewer must exceed the tide elevation. The rate of discharge is also related to the relative elevation difference between the water level in the combined sewer and the receiving water. Thus, increased sea levels would tend to reduce the volume of combined sewage overflow. Existing tide levels were used to provide a conservative estimation of the alternatives' performance.

There have been discussions of changes in rainfall patterns. Unfortunately, there are no reliable predictions that can be applied to create a Typical Year for planning purposes. It is noted that, through the development of the Typical Year, that for the top 10 contenders, there were years ranging from 1973 to 2014 with every decade in between represented, and initially the top two ranked years were from the 1980s. This seems to indicate that the rainfall pattern as they relate to Typical Year analysis have been relatively static. Accordingly, lacking a reasonable method for predicting future weather conditions, it is reasonable to assume the 2004 rainfall is suitable for use in the future baseline condition.

It is noted that there is a great deal of uncertainty in future projections and that as the planning horizon increases the uncertainty increases dramatically. This is evidenced below in cases where a variety of reputable sources produce widely differing projections. The goal was to select future conditions that would be a reasonable, yet conservative, estimate of likely future conditions. It is noted that actual future conditions could vary substantially due to demographic trends, economic conditions, changes in technology, climate impacts and a myriad of other influences beyond the control of the Permittee.

The residential portion of the Village west of the NJ Turnpike is substantially developed with very few areas that are undeveloped. It is anticipated that most of the future residential development will take place in the separate sewered areas east of the NJ Turnpike. Any proposed development in the rest of the

Village will likely continue as it has in the past as subdivision and infill development, and such growth can reasonably be expected to be negligible, as further discussed in this section.

Several population projections were sourced to select a reasonable projection for the future baseline. These are summarized as follows:

2.2.1.1 U.S. Census Bureau

The United State Census Bureau is considered an authoritative source for population data. Data is available from the 1970, 1980, 1990, 2000 and 2010 censuses, as well as population projections through 2015, see Table 2-1.

Table 2-1: US Census Population Data

Year	Population
1970	13,990
1980	12,738
1990	12,522
2000 (US Census)	12,873
2010 (US Census)	12,729
2017 (US Census 7-Year Estimate)	13,154

Source: https://factfinder.census.gov/faces/tableservices/jsf/pages/productview.xhtml?src=CF

As can be seen from the data, the population in the Village dropped between 1970 and 1980, but has stayed relatively constant, within a few hundred people, since then. It is possible to extrapolate a population for 2050 using this data, however the estimates from the historic long-term trends (1990 to 2015) would produce a number that is likely too low. Even using the short-term trends (2010-2015) may produce an unrealistically low estimate since the population changed less than 2% during that five-year period. The overall conclusion from the historic Census data, which pertains primarily to the combined area, is that the population has historically been static, and it is reasonable to assume it will remain relatively static. Thus, for future baseline purposes all future growth can be assumed to take place in the separately sewered areas.

2.2.1.2 North Jersey Transportation Authority (NJTPA)

The NJTPA is a metropolitan planning organization with federal authorization. It is responsible for the 13 northern counties in New Jersey and is responsible for overseeing certain transportation related projects and studies. The NJTPA updates its regional forecasts for population, households and employment every four years.

In 2017, NJTPA completed the latest set of forecasts. Final forecasts were approved by the NJTPA Board on November 13, 2017 and extend to 2045. The NJTPA employs the Demographic and Employment Forecast Model (DEFM). According to their website:

> The DEFM uses regional and county level forecasts of employment, population and households produced from a regional econometric modeling effort and allocates these forecasts to a localized Traffic Analysis Zone (TAZ) level. It also aggregates the TAZ level information to the municipal level. The DEFM uses data elements that influence location behavior to perform this allocation analysis including:
>
> - Current land use data (residential, commercial, industrial and vacant land);

- Composite zoning estimates for density;
- Highway and transit accessibility;
- Historical growth; and
- Known project developments.

The NJTPA forecasts strong growth in the population of Ridgefield Park, but as noted above the NJTPA forecasts only extend to 2045 and fall short of the 2050 planning period. The forecast, which was extended to 2050 anticipating full buildout after 2045, is summarized in Table 2-2 below:

Table 2-2: NJTPA population projections

County	Municipality Code	Municipality Name	2015 Population	2045 Population	Annualized % Population Change 2015-2045	2050 Population Extrapolation
Bergen	3400362940	Ridgefield Park, village	12,946	17,960	1.1%	18,970

Source: https://www.njtpa.org/data-maps/demographics/forecasts.aspx

It can be seen from the above table, that based on the historical growth rate, the NJTPA forecast produces an unrealistically high estimate for 2050, which is most like an indication that the 1.1% annual growth cannot be continuously sustained. Likely, redevelopment which is driving the overall population growth will reach a saturation point at which time the population growth would stagnate. Thus, the average growth rate of 1.1% from 2015 through 2045 is driven by redevelopment earlier in the period and would likely not persist beyond 2045. The NJTPA population projection does indicate a population growth in Ridgefield Park to 17,960 people in 2045, which will be used as a comparison to other population projection methods.

2.2.1.3 New Jersey Department of Labor

Population and labor force projections on a county-wide basis have been developed by the New Jersey Department of Labor extending to 2034. To obtain an estimated population for 2050, we assumed that Ridgefield Park will grow at the same rate as the county over the entire period. Accordingly, since Village made up 1.4% of the county population in 2010 it would be expected to make of 1.4% of the county population in 2050. The projected county population was extrapolated to 2050, which yields the following estimates shown in Table 2-3.

Table 2-3: New Jersey Department of Labor population estimates for Bergen County

	Census	Projections to July 1		Projected for LTCP
County	4/1/2010	2017	2034	2050
Bergen	905,143	936,692	1,032,770	1,117,854
Ridgefield Park	12,729	13,172	14,523	15,720

It is noted that the Village of Ridgefield Park may undergo development in the future, and this method yields a population in 2050 that is less than some other projections of the current estimated population. The redevelopment and demographic trends of Ridgefield Park may not be mirrored by the overall county. As such, projected trends in population growth at the county level are not suitable for projecting the

population of Ridgefield Park and Department of Labor population estimates were not used exclusively for this analysis.

2.2.1.4 Undeveloped Properties - Sky Mark Property Projections

The Village of Ridgefield Park has limited open space for redevelopment except for the property east of the NJ Turnpike and south of Route 46, currently under planning for a mixed-use development. While it is unknown as to whether this planned development will move forward, it is anticipated that this property will be developed over the next thirty years in some manner.

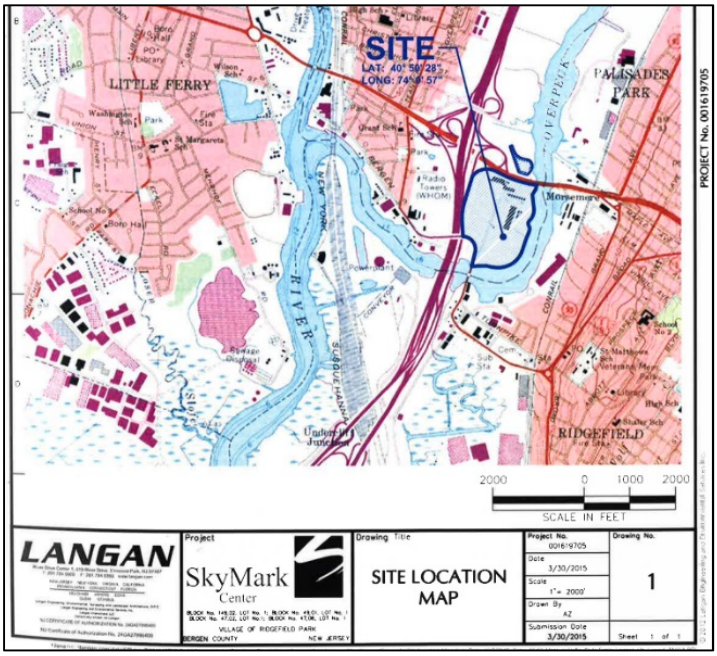


Figure 2-1: Skymark Site Location Map by Langan

The Sky Mark Center (See Figure 2-1) was proposed for development in the 2015 as a mixed-use project that was to include over 340,000 sf of retail, a hotel (300 rooms) and 30,000sf conference center, and approximately 2,000 residential units that were 90% one bedroom and 10% two bedrooms. Nevertheless, for planning purposes it was decided that a more conservative estimate would be dividing the units between 50% one bedroom, 45% two-bedroom units and 5% three-bedroom units. The project is proposed to connect directly to the BCUA Branch Intercepting Sewer near the Overpeck Creek. Based on assumed split of dwelling unit sizes see Table 2-4, it is estimated that the population growth for each unit constructed will be 1.87 people. Based on this projection it is estimated that the 2,000 new units would generate an additional population of 3,740 persons, which would bring the total population of the Village to around 16,470.

Table 2-4: Anticipated household size distribution.

Housing Type/Size	Total Persons per Unit (including school children)
Studio/1 BR (50%)	1.526
2BR (45%)	2.106
3BR (5%)	3.109
Weighted Average	1.87

Source: Multiplier source: “Who Lives in New Jersey Housing?” Table II-A-1 Total Persons, Prepared by David Listokin

2.2.1.5 BCUA Wastewater Management Plan Projections

The Bergen County Utilities Authority provides wastewater service to approximately half of Bergen County including the Village of Ridgefield Park. The Authority updated their Wastewater Management Plan (WMP) in 2014 to plan for anticipated changes to their service area and population over the next 20 years (2013 – 2033). As part of this planning document the BCUA made projections on population increases that may occur within their service area, in addition to new areas that may be serviced. The Authority's WMP projected that the population within the Village of Ridgefield Park would increase from an estimated population of 12,864 people in 2013 to 13,757 people in 2033, which extrapolated to 2050 would result in 14,620 people. More Details can be found in the BCUA report.

2.2.2 Summary of Anticipated Population Growth

Mott MacDonald considered various sources of data on population growth as outlined above to determine a reasonable increase in population within the Village during the thirty-year planning period for the CSO LTCP. Overall, five sources of population growth were used and extrapolated to 2050 to provide a reasonable understanding of the overall potential population growth within the Village. The projections ranged from a low of 14,620, based on extrapolation of recent census data to populations of almost 18,000 based on NJTPA projections. Overall, it is anticipated that projections based on previous population data will not be reliable since there is undeveloped land in the Village that is currently in the planning phase for development. Population increases are closely tied to the overall economy and accordingly very difficult to project. A conservative approach would be to take the three highest population projections as summarized in Table 2-5 and to average those for planning purposes. It should be noted that this is in close agreement with the estimates of population increases made from development of the largest parcel of undeveloped property within the Village and therefore anticipated to be representative of future population growth within the Village. Accordingly, modeling projections for future conditions within the Village of Ridgefield Park will be made using an anticipated Village population of 16,720 people, with most of the population growth located on the Sky Mark property.

Table 2-5: Summary of Population Projections to 2050

Data Source	Projected Population to 2050 – All Sources (People)	Projected Population to 2050 - Conservative (people)
NJTPA	17,960	17,960
US Census Projection	15,910	
NJ Department of Labor	15,720	15,720
Sky Mark Development Analysis	16,470	16,470
BCUA Projections	14,620	
Average	16,100	16,720

2.2.3 Future Baseline Modeling

Future baseline conditions were implemented in the InfoWorksICM model by adding a sanitary sewer sub-catchment located on the site of the proposed Skymark development. The sub-catchment was assigned a population equal to the projected population increase, and assigned typical per capita flows and diurnal patterns from other areas in the Village. As expected when the model was run there were no meaningful changes in overflow volumes. The sub-catchment was assigned a population equal to the projected Increased populations, no change as expected in separate sewered areas. See Table 2-6.

Table 2-6: Comparison of 2015 Baseline and 2050 Future Baseline

Outfall	Baseline 2015			Baseline 2050			Change		
	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)
001A	19	6.3	74	19	6.4	74	0	0.0	0
002A	13	0.4	11	13	0.4	11	0	0.0	0
003A	45	15.3	300	45	15.3	300	0	0.0	0
004A	53	24.1	614	53	24.1	614	0	0.0	0
005A	25	3.6	121	25	3.6	121	0	0.0	0
006A	12	0.5	39	12	0.5	39	0	0.0	0
Total	53	50.3	--	53	50.3	--	0	0.0	--

3 Water Quality Objectives

3.1 Applicable Water Quality Standards

The NJDEP has established water quality standards for each of the receiving waters within the State of New Jersey based on their intended use and whether they are freshwater or saline waters. The standards are based on both bacterial and physical/chemical standards such as levels of dissolved oxygen, turbidity, nutrients, pH, etc. Discharges from combined sewer overflows contribute pathogens, and thus the parameter of interest for CSOs is the bacterial standards. Bacterial standards are typically set with monthly mean and single sample maximums set at levels to protect the watercourse's primary or intended use. The following outlines the bacterial standards and protected uses for each water quality classification:

Table 3-1: Applicable water quality standards

Class	Description	Bacterial Standards	Monthly Mean	Single Sample Max	Protected Uses
SC	Saline Ocean	Entero	35	104	Primary Contact, Shellfishing
SE1	Saline Estuary	Entero	35	104	Primary Contact
SE2	Saline Estuary	Fecal	770	NA	Secondary Contact
SE3	Saline Estuary	Fecal	1500	NA	Secondary Contact
FW2	Fresh Water	E. coli	126	235	Primary Contact and Public Water Supply

Ridgefield Park is part of the Hackensack River Watershed (HUC-11 02030103180 - Hackensack River below and including Hirshfeld Brook), which is in State Watershed Management Area 5. The Village is situated on a ridge that divides two sub-watersheds, to the east of the ridge is in the Overpeck Creek Watershed and to the west is the Hackensack River watershed, Fort Lee Road to Oradell gage. The combined sewers discharge to two watercourses, for 003A-006A the Hackensack River which has a surface water quality classification of Saline Estuary one (SE-1) and the Overpeck Creek which has a surface water classification of Freshwater 2, Non-Trout and Saline Estuary 2 (FW2-NT/SE2) (See Figure 3-1). There is a tide gate structure on the Overpeck Creek at the New Jersey Turnpike (Route 95) and both 001A and 002A combine sewer discharges are downstream of this point in the tidal (saline estuary) portion of the creek.

The saline regions of the Hackensack River appear on the NJDEP's 303(d) List of Water Quality Limited Waters primarily for chemical and pesticide contamination including, but limited to: Dioxin, Heptachlor epoxide, PCB in Fish Tissue, DDT and its metabolites in Fish Tissue, Mercury in Fish Tissue, Chlordane in Fish Tissue, and Dieldrin issues extending the full length of the Estuary from its confluence with Newark Bay up to the USGS river gage at the Oradell dam. These contaminants impact the designated use of fish consumption for SE1, 2, and 3 classified waters. In addition, low dissolve oxygen levels that impact aquatic life have been detected primarily in the SE-3 designated region, and high enterococcus levels in the SE1 region of the river, which includes the reach along Ridgefield Park.

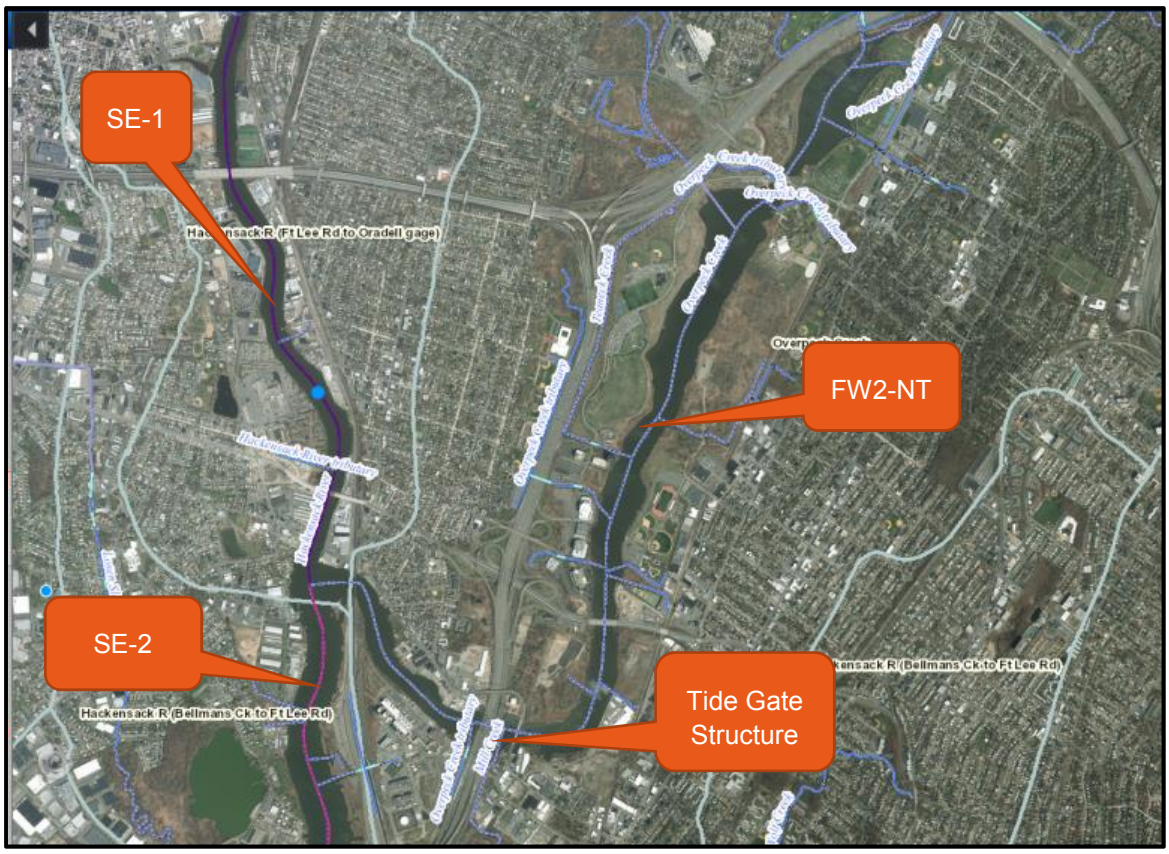


Figure 3-1: WQS Around Ridgefield Park

Overpeck Creek is also listed on the NJDEP 303(d) List of Water Quality Limited Waters for chemical and pesticide contamination including Chlordane in Fish Tissue, DDT and its metabolites in Fish Tissue, Dioxin, and PCB in Fish Tissue, all of which impact the designated use of fish consumption for SE2 and FW2 waters. In addition, high levels of Escherichia coli (E-coli) were also detected in the FW-2 section of the Creek, which impacts recreation within the region. It should be noted that there are no CSO discharge points within the freshwater segment of Overpeck Creek.

3.2 Existing CSO Controls

The Village of Ridgefield Park is currently in compliance with all NJPDES Permit requirements. Solids/Floatables Control facilities were constructed in the late 1990's and have been well maintained and operated since then. In the early 2000's the regulator gate valves at Regulators 003, 004a, and 004b were replaced with vortex valves. There have been no additional modifications to the Ridgefield Park combined sewer system other than those previously noted.

3.3 Range of CSO Goals Being Evaluated

The EPA CSO Control Policy and the Village's NJPDES Permit requires that the CSO Long Term Control Plan be developed using either the Presumptive or Demonstrative approach as outlined below.

3.3.1 Presumptive Approach

The "Presumption" Approach refers to a program that, once fully implemented, is presumed to meet WQS. The Presumption Approach requires the following:

- No more than an average of four overflow events per year occurs from a hydraulically connected system as the result of a precipitation event. The Department may allow up to two additional overflow events per year.
- Elimination or the capture for treatment of no less than 85% by volume of the combined sewage collected in the CSS during precipitation events on a hydraulically connected system-wide annual average basis.
- Elimination or removal of no less than the mass of the pollutants, identified as causing water quality impairment.

3.3.2 Demonstration Approach

The "Demonstration" Approach refers to a program that uses a receiving water model to meet the water quality-based requirements of the Clean Water Act taking into consideration the following:

- The planned control program is adequate to meet WQS and protect designated uses, unless WQS or uses cannot be met because of natural background conditions or pollution sources other than CSOs.
- The CSO discharges remaining after implementation of the planned control program will not preclude the attainment of WQS or the receiving waters' designated uses or contribute to their impairment.
- The planned control program will provide the maximum pollution reduction benefits reasonably attainable.
- The planned control program is designed to allow cost effective expansion or cost-effective retrofitting if additional controls are subsequently determined to be necessary to meet WQS or designated uses.

This Development and Evaluation of Alternatives Report has been created to provide a comprehensive review and analysis of a range of CSO control strategies to meet the CSO component of the water quality goals.

The Permit requires a selection of presumptive or demonstration during the alternatives analysis, whereas the EPA's CSO Guidance for Long Term Control Plans states:

> *"The CSO Control Policy recommends flexibility in allowing a municipality to select controls that are cost-effective and tailored to local conditions. For this reason, the choice between the demonstration approach and presumption approach does not necessarily have to be made before a municipality commences work on its LTCP. In some cases, it might be prudent for a municipality to assess alternatives under both approaches. In addition, if a municipality has CSOs that occur to two different water bodies, a control plan that includes the demonstration approach for one receiving water and the presumption approach for the other may be appropriate."*

The Baseline Compliance Monitoring Program and Model Development undertaken by the NJ CSO Group (includes Ridgefield Park) indicates that complete elimination of all CSO discharges within the Gateway Region (Passaic River, Hackensack River, Newark Bay, and Overpeck Creek will not bring the Hackensack River above Overpeck Creek into compliance with pathogens due to other pollutant loads. Accordingly, the Village of Ridgefield Park will be undertaking a Demonstrative Approach in the development of their LTCP to evaluate the maximum pollutant reduction benefits reasonably attainable. However, in consultation with the Department, as additional information becomes available and as progress is made the Village may revise this selection.

4 Development and Screening of CSO Control Alternatives

4.1 Introduction

A two-tiered approach was applied to the development of alternatives, starting with a screening analysis followed by an evaluation of the remaining CSO control alternatives. The intent is to give adequate attention to the breadth of alternatives available, but to limit the list of alternatives evaluated to a reasonable amount. This is consistent with Chapter 3 of EPA's Guidance for Long Term Control Plans which states:

> *"Ultimately, the NPDES permitting authority should be satisfied that the municipality has studied all reasonable options in developing a list of final alternatives for evaluation and that the evaluation process incorporates all identified concerns"*
>
> *"As noted previously, in the initial alternative development steps, the number of alternatives necessary to cover the range of control levels for each CSO can be very large. Judgment is necessary to develop a manageable array of alternatives. It is important to remember that the iterative screening of alternatives is flexible and not a rigid process."*
>
> *"Given the number of specific control measures within each of these categories and the range of sizing options for specific measures, initially it might be practical to consider general categories, such as storage or treatment, rather than specific storage or treatment technologies."*
>
> *"Alternatively, it might be appropriate to identify "representative" technologies, with the understanding that specific technologies would be considered as part of more detailed evaluations."*
>
> *"Once a range of CSO control goals has been developed and approaches to structuring CSO control alternatives have been identified, the next step is to develop specific alternatives to achieve the various CSO control goals."*

The first step of the screening process was to identify the breadth of alternatives which can then narrowed down to alternatives appropriate for the evaluation process. If necessary, a representative technology to apply to the evaluation is identified. A comprehensive list of CSO control alternatives was prepared by the NJCSO Group and this list is screened below.

The evaluation of seven (7) CSO control alternatives is mandated in the Permit in Part IV G 4e. This list is not intended to be limiting, but rather sets general categories of control alternatives that must be considered. The list of control alternatives provided in the Permit is broad enough that most of the control alternatives explored in the subsequent sections fall within the list. The seven (7) control alternatives listed in the Permit, and the corresponding sections of this report in which they are addressed are:

i. Green infrastructure [Refer to Section 4.3]
ii. Increased storage capacity in the collection system. [Refer to Section 4.5 and Section 4.6]
iii. STP expansion and/or storage at the plant (an evaluation of the capacity of the unit processes must be conducted at the STP resulting in a determination of whether there is any additional treatment and conveyance capacity within the STP). Based upon this information, the permittee shall determine (modeling may be used) the amount of CSO discharge reduction that would be achieved by utilizing this additional treatment capacity while maintaining compliance with all permit limits. [This is BCUA's responsibility, refer to Section 4.7.1]

iv. If applicable, the evaluation of dry and wet weather flows that entering the combined sewer system from separately sewered municipalities, and in all cases Inflow/Infiltration (I/I) reduction in the entire collection system that conveys flows to the treatment works to free up storage capacity or conveyance in the sewer system to inform the evaluation of I/I reduction as an alternative in the LTCP. [This is BCUA's responsibility, refer to Section 4.4.1]

v. Sewer separation. [Refer to Section 4.8]

vi. Treatment of the CSO discharge. [Refer to Section 4.9]

vii. CSO related bypass of the secondary treatment at the STP portion of the STP in accordance with N.J.A.C. 7:14A-11.12 Appendix C, II C.7. [This is BCUA's responsibility, refer to Section 4.7.2]

The screening took place on several levels. In some cases, a general category may be screened in or out based on its applicability to the Village. In other instances, while the general category may be applicable, only certain specific sub-categories of the control may be applicable. If the general category is applicable as are many sub-categories, the screening will reduce the sub-categories to a reasonable number of representative sub-categories. This is allowable under Part IV G 4a, which calls for the Permittee to "evaluate a reasonable range of CSO control alternatives".

The screening was based on the requirement to "evaluate the practical and technical feasibility of the proposed CSO control alternative(s)" (Part IV G 4e) to determine if the alternative will proceed to a more detailed evaluation. The above requirement introduces three concepts that may be addressed for each alternative.

- Evaluate – As per the requirements of Part IV G 4a and b the alternatives must contribute to the "water quality-based requirements". This means that while an alternative may be beneficial as a matter of good practice, if the benefit cannot be quantified in terms of water-quality benefits resulting from CSO reduction, it cannot be evaluated. Many such practices are already in place as requirements under the Permit or the Village's MS4 permit. These practices would be considered part of the Baseline and their continuation part of the Future Baseline and would not be part of the LTCP.

- Practical – The facilities and measures ultimately implemented under the LTCP must be practical for the Village to implement. For example, a residential neighborhood could not be displaced to make room for a storage or treatment facility. Nor could the Village's entire municipal budget be devoted to CSO controls. Accordingly, alternatives that clearly have excessive community/societal impacts or alternatives the provide marginal CSO controls at high costs will be removed from consideration.

- Technical feasibility – Technology is continually advancing and what is not technically feasible today, may be in the future. However, there are no guarantees of such advancement. There are certain general limits, for example, maximum tunnel diameter and depth of open cut pipe installation, that will be observed for cost and safety reasons. Accordingly, technical feasibility is limited to the current state of the practice. Future advancements, should they occur, will need to be addressed in future permit iterations.

The screening will focus on applying the above criteria to the alternatives described in the following sub-sections. A summary matrix of the screening alternatives is included in Section4.10.

4.2 Source Control

The EPA defines source controls as those that impact the quality or quantity of runoff entering the combined sewer system. Source control measures can reduce volumes, peak flows, or pollutant discharges that may decrease the need for more capital-intensive technologies downstream in the CSS.

However, source controls typically require a high level of effort to implement on a scale that can achieve a measurable impact. Source controls are discussed in more detail in this section.

The Village is already performing many of the quantity and quality source control measures described herein as best management practices (BMPs) for stormwater management and pollution prevention. These BMPs are often system-wide controls that are complemented by general public housekeeping efforts (i.e., litter control, household hazardous waste collection, and illegal dumping ordinances). These current management practices will be continued as applicable to help optimize system operations and minimize CSO discharges and impacts to receiving water bodies.

4.2.1 Stormwater Management

Stormwater management consists of measures designed to capture, treat, or delay stormwater prior to entering the CSS.

4.2.1.1 Street/Parking Lot Storage (Catch Basin Control)

Street and parking lot storage can be accomplished by modifying catch basins to restrict the rate of stormwater runoff that enters the CSS. A portion of the stormwater runoff that would otherwise immediately enter the CSS is allowed to pond on streets or parking lots for a period before entering the CSS. However, such intentional surface detention is associated with various risks and public safety concerns, including hydroplaning, ice formation during winter months, and flood damage due to malfunctioning control devices.

Providing surface storage by intentionally ponding runoff in streets and parking lots risks property damage and potential injury, and is not considered a practical component of a LTCP. Accordingly, street/parking lot storage will be removed from consideration.

4.2.1.2 Catch Basin Modification (Floatables Control)

Catch basin modifications consist of various devices that can help prevent floatables from entering the CSS. Inlet grates can reduce the amount of street litter and debris that enters the catch basin. Other modifications such as hoods, submerged outlets and vortex valves alter the outlet pipe hydraulics and keep floatables from exiting the catch basin and continuing downstream.

In Ridgefield Park, outfalls are equipped with netting facilities which already provide control of coarse solids and floatables. Catch basin modification to incorporate additional floatables controls would not provide a significant or quantifiable contribution to the LTCP and will be removed from consideration for the alternative evaluation.

4.2.1.3 Catch Basin Modifications (Leaching)

Catch basin modifications for leaching consist of catch basin base and riser sections that permit infiltration of stormwater into the ground. Leaching catch basins are generally installed in a geotextile and crushed stone lined excavation. Leaching catch basin installations are limited to highly permeable soils and should not be installed in series with other drainage structures. Leaching catch basins can be installed with or without an outflow pipe. Basins without an outflow pipe can overflow into streets and parking lots and then freeze under excessive storm events or if soils decrease permeability over time. These control measures function much like an infiltration basin without an emergency overflow pipe. To avoid this adverse feature, an outflow pipe should be included in all leaching modified catch basins unless there is minimal flow to the basin, and a low overflow damage risk to the surrounding area.

To function leaching catch basins must be able to infiltrate the water collected in their sumps. This requires two things; soils capable of infiltration and separation from groundwater. In general, the soils in the Village are mixed and groundwater is relatively deep except for the areas near receiving waters. The infiltration rate is related to the surface area exposed and thus modification to the catch basin would

normally require removal of the catch basin bottom and filling the bottom with stone. This is labor intensive and may be comparable in cost to replacing the entire catch basin. In addition, stormwater runoff contains grit, sand, and other fines that would eventually clog the soil surface and reduce or eliminate the ability to infiltrate, due to the high ratio of impervious are to infiltration area. Accordingly, catch basin modifications for leaching will be removed from consideration for the alternative evaluation.

4.2.2 Public Outreach Program

Public education and outreach are non-structural control measures aimed at limiting the negative effects of certain human behavior on the CSS. Promoting certain human actions and discouraging others can impact the quality and quantity of water discharged to the CSS. Existing stormwater management and CSO permits have several requirements for public education and outreach and these ongoing programs incorporate many of the practices described herein.

The Village of Ridgefield Park already has stormwater management programs and ordinance as part of their MS4 Program. While public outreach programs are beneficial, they are generally not evaluated as part of the LTCP. This does not mean the LTCP will not include public outreach, but rather that it is not a quantifiable component of the plan generally because these programs rely on human behavior which cannot be predicted. Nevertheless, the specific public outreach alternatives identified are summarized below for completeness.

It is anticipated that public outreach will continue under future iterations of the Permit and should be considered a key component of the LTCP. However, the impact of public outreach cannot be quantified, thus public outreach is removed from future consideration for alternative evaluation.

4.2.2.1 Water Conservation

Water conservation in CSS areas can reduce the volume of direct discharges to the system. Water conservation measures include the installation of low-flow fixtures, education to reduce water waste, leak detection and correction, and other programs.

Although water conservation has merits in reducing water demand, and can reduce dry weather flows in the sewer system, it has minimal impact on peak wet weather flows. It does not change the total pollutant load but results in less flow with a higher concentration. It is also difficult to enforce long term, as residents can change plumbing fixtures. Accordingly, while the Village should continue its current programs and code enforcement to conserve water, it is not practical to make it a component of the alternatives evaluation. The impact of water conservation measures on dry weather flows may be apparent in future CSS monitoring programs.

4.2.2.2 Catch Basin Stenciling

Stenciling consists of marking catch basins with symbols and text such as," Drains to the River" or "Only Rain Down the Storm Drain". This measure can help increase public awareness of the sewer system and discourage the public from dumping trash into the CSS, which can cause blockages and lead to CSOs. Catch basin stenciling is only as effective as the public's understanding and acceptance of the program.

Catch basin stenciling is already required under the New Jersey stormwater management permits, and the Village is complying with the applicable requirements. Any benefits derived from stenciling would have been seen in the system characterization and may be observed in future combined sewer system characterizations. The performance of stenciling is dependent on human behavior, i.e. the response of the observer to alter their actions due to the presence of the stenciling, which cannot be reliably enforced or predicted. Accordingly, catch basin stenciling will be removed from consideration for the alternatives evaluation.

4.2.2.3 Community Cleanup Programs

Community cleanup programs are an inexpensive and effective way to reduce floatables entering the CSS and provide educational benefits to the community. Cleanup activities can be organized by local businesses, non-profit organizations, and student chapters at all levels. It is a great way to raise the sense of community spirit and environmental awareness.

The Village currently supports and hosts various community cleanup efforts and these existing programs and practices will be continued. As an existing practice, this measure will not be considered further for the alternatives evaluation.

4.2.2.4 FOG Program

Fats, oils and grease (FOG) are not water soluble and will buildup and clog sewer and drainage pipes, resulting in messy, costly sanitary sewer overflows. These overflows are bad for commercial and retail businesses, the environment, and public health. FOG programs often consist of food service establishment inspection, installation of Grease Removal Devices (GRDs) and development of a preferred pumper program for proper maintenance of GRDs.

FOG programs have little effect on the number of bacteria in the collection system and do not provide any flow reductions. Accordingly, this measure is a program enhancement and will not be considered further for the alternatives evaluation.

4.2.2.5 Garbage Disposal Restrictions

Garbage disposals provide a convenient means for residences and businesses to dispose of food waste. However, the use of garbage disposals increases the amount of food scrap entering the sewer system and is known to cause blockages and decrease the flow capacity in the CSS. Restricting garbage disposal usage has the potential to decrease the number of blockages that occur each year. Garbage disposal restrictions require an increased allocation of resources for enforcement and can face considerable public resistance. Furthermore, this practice does very little to reduce wet weather CSO events or decrease bacteria loads.

Accordingly, garbage disposal restrictions will be removed from future consideration for the alternatives evaluation.

4.2.2.6 Pet Waste Management

When pet waste is not properly disposed of, it can be carried away by stormwater runoff and washed into storm drains or nearby streams. Since storm drains do not always connect to treatment facilities, untreated animal feces often end up in waterways, causing significant water pollution. An effective pet waste management program can help increase public awareness and encourage proper waste disposal. This is a low cost, long term program that has the potential to reduce bacteria loads to both the CSS and directly to local streams.

The Village currently enforces a pet waste management ordinance as required under the stormwater management permits. As an existing practice that will be continued, the impact of pet waste management is reflected in the baseline conditions. Accordingly, pet waste management will be removed from future consideration for the alternatives evaluation since it is already in place.

4.2.2.7 Lawn and Garden Maintenance

Failure to apply chemical treatments to lawns or gardens per USEPA guidelines may lead to ineffective treatment and contamination of the waterways through runoff or groundwater. A public outreach program that explains the guidelines and the reasons they exist may help reduce waterway contamination. This

information is currently available to the public on the following USEPA website: https://www.epa.gov/safepestcontrol/lawn-and-garden. Runoff that contains chemical treatments can contribute to decreased water quality downstream of the CSS in the receiving waters.

The permittees will continue the current practices to minimize and control these chemical uses. As an existing practice that will be continued, the measure will be removed from future consideration for the alternatives evaluation.

4.2.2.8 Hazardous Waste Collection

Improperly disposed hazardous waste can find its way into stormwater runoff and into storm drains and waterways. Hazardous waste that ends up in storm drains does not necessarily end up in a treatment facility and can cause significant surface water pollution. To prevent this, household hazardous waste collection events can be scheduled a few times every year to allow the community to properly dispose of any hazardous waste.

The Village participates in a County hazardous waste collection program and anticipates continued participation in the program. As an existing practice that will be continued, the impact of program is reflected in the existing conditions. Accordingly, the measure will be removed from future consideration for the alternatives evaluation since it is already in place.

4.2.3 Ordinance Enforcement

4.2.3.1 Construction Site Erosion and Sediment Control

Construction site erosion and sediment control involves management practices aimed at controlling the transport of sediment and silt by stormwater from disturbed land. Erosion and sediment control have the potential to reduce sediment loads to both the CSS and directly to streams, and can help reduce operation and maintenance (O&M) costs for sewer cleaning.

N.J.S.A. 4:24-39, NJ Soil Erosion and Sediment Control Act, requires all construction activities greater than 5,000 square feet to complete an application for certification of an erosion and sediment control plan for activities during construction. New Jersey's requirements for soil erosion and sediment control are enforced through the Bergen County Soil Conservation District and it is anticipated this enforcement will continue at its current level or with more stringent requirements in the future. Accordingly, construction site erosion and sediment control will be removed from future consideration for the alternatives evaluation since it is already in place.

4.2.3.2 Illegal Dumping Control

Illegal dumping is the disposal of trash or garbage by dumping, burying, scattering, or unloading trash in an unauthorized place, such as public or private property, streets or alleys, or directly into the CSS. When it occurs, illegal dumping contributes a considerable amount of floatables to stormwater runoff, as well as a moderate number of bacteria, settleable solids, and other pollutants.

The Village of Ridgefield Park enforces ordinances to control illegal dumping and intends to continue doing so. Accordingly, illegal dumping control will be removed from future consideration for the alternatives evaluation since it is already in place.

4.2.3.3 Pet Waste Control

As described in the previous section, pet waste can be a significant contributor of bacteria to stormwater. Public education and outreach programs can help raise public awareness and reduce the level of improper waste disposal. Additional gains can be made through enforcement of the pet waste ordinances, which can be an effective tool in achieving public compliance. Significant resources would need to be

devoted to enforcement to achieve similar improvements to Pet Waste Management, which requires very few resources to implement.

The Village currently enforces a pet waste management ordinance as required under its MS4 permit and as a matter of good practice. The impact of pet waste management should be reflected in the characterization. Accordingly, pet waste management will be removed from future consideration for the alternatives evaluation since it is already in place.

4.2.3.4 Litter Control

Litter consists of waste products that have been disposed of improperly in an inappropriate area. Litter is easily washed into the collection system during wet weather events, which increases the amount of floatables in the system. Strict enforcement of the litter control ordinances can help to curb violations and decrease the amount of floatables that make their way into the CSS. Like Pet Waste Control, public outreach and education is a more effective use of resources to achieve similar water quality improvements.

The Village currently enforces a litter control ordinance as required under its MS4 permit and as a matter of good practice. The impact of the litter control ordinance should be reflected in the characterization. Accordingly, litter control will be removed from future consideration for the alternatives evaluation since it is already in place.

4.2.3.5 Illicit Connection Control

An illicit discharge is any discharge to the municipal separate storm sewer system (MS4) that is not composed entirely of stormwater, except for discharges allowed under a NPDES permit or waters used for firefighting operations. Illicit connections can contribute polluted water, solids, and trash to the stormwater system, where it is eventually discharged to the environment without receiving proper treatment. These connections can be reduced through the implementation of an illicit discharge detection and elimination (IDDE) program. Although this measure does not directly target the CSS, it can have significant impacts on local water quality that can help to address Total Maximum Daily Loads (TMDLs). Illicit connection control is not particularly effective at achieving any of the primary goals of the LTCP.

The Village currently controls illicit connections under its MS4 permit and as a matter of good practice. Illicit connection control is applicable only to separately sewered areas since combined sewers are intended to accept sanitary flows. Accordingly, illicit connection control will continue to be monitored by the Village as part of its MS4 Permit, but will be removed from future consideration for the alternatives evaluation since it is already in place.

4.2.4 Good Housekeeping

4.2.4.1 Street Sweeping/Flushing

Municipal street cleaning enhances the aesthetic appearance of streets by periodically removing the surface accumulation of litter, debris, dust and dirt, which prevents these pollutants from entering storm or combined sewers. Common methods of street cleaning are manual, mechanical and vacuum sweepers, and street flushing. However, the total public area accessible to street sweepers is limited, and generally does not include sidewalks and parking lot areas. Although street sweeping/flushing can reduce the concentration of floatables and pollutants in storm runoff that originate from the street, the measure has minimal impact on bacteria or CSO volume reduction.

The Village currently has a program to sweep streets and anticipates that program will continue as required under its MS4 permit. The impact of street sweeping is already reflected in the characterization and its continuation would be reflected in future baseline. Thus, there is no quantifiable additional impact of street sweeping. Accordingly, street sweeping will be removed from future consideration for the alternatives evaluation since it is already in place.

4.2.4.2 Leaf Collection

Leaf collection is an important part of stormwater management because it not only keeps leaves out of the stormwater or combined sewer system to maintain its maximum flow capacity, but also benefits water quality by reducing nutrients such as phosphorous and nitrogen that can originate from the decomposition of leaves. In most municipalities, this long-term stormwater management measure is scheduled based on seasonal patterns, and is an effective tool to maintain capacity in both the separate storm sewer and the CSS

The Village currently has a leaf collection program and anticipates that program will continue as required under its MS4 permit. The impact of leaf collection is already reflected in the characterization and its continuation would be reflected in future baseline. Thus, there is no quantifiable additional impact of street sweeping. Accordingly, leaf collection will be removed from future consideration for the alternatives evaluation since it is already in place.

4.2.4.3 Recycling Program

Recycling programs provide a means for the public to properly dispose of items that may otherwise end up entering the CSS, such as motor oil, anti-freeze, pesticides, animal waste, fertilizers, chemicals, and litter. These programs are usually effective in reducing floatables and toxins.

The Village currently has a recycling program and anticipates the program will continue. The impact of recycling is already reflected in the characterization and its continuation would be reflected in future baseline. Thus, there is no quantifiable additional impact of a recycling program. Accordingly, recycling will be removed from future consideration for the alternatives evaluation since it is already in place.

4.2.4.4 Storage/Loading/Unloading Areas

Industrial and commercial users would be required to designate and use specific areas for loading and unloading operations. This would concentrate the potential for loading and unloading related waste to a few locations on site, making it easier to manage waste. The effectiveness of this technology is limited to the number of industrial users upstream of CSO regulators. NJDEP administers an industrial stormwater permitting program to ensure that significant industrial facilities manage stormwater runoff to minimize contact between pollutants and stormwater, including requirements to implement BMPs for loading and unloading activities.

Implementing and enforcing ordinances regarding storage/loading/unloading areas in Village is unlikely to produce a quantifiable benefit to the CSO program. There are minimal industrial areas in the combined sewer area. These are regulated under industrial stormwater permits which require good housekeeping procedures to be followed. Containment is required for loading and unloading areas for hazardous materials. Any solids and floatables prevented from entering the combined sewer system would likely be captured by the existing netting facilities, minimizing any benefits from enforcing additional ordinances. Accordingly, storage/loading/unloading areas ordinances are removed from future consideration for the alternatives analysis.

4.2.4.5 Industrial Spill Control

Industrial users would be required to utilize spill control technologies like containment berms and absorbent booms to mitigate the risk of contaminants entering the waterway or collection system. Like Storage/Loading/Unloading Areas, the effectiveness of this technology is limited to the number of industrial users upstream of CSO regulators. NJDEP administers an industrial stormwater permitting program to ensure that significant industrial facilities manage stormwater runoff to minimize contact between pollutants and stormwater, including requirements to implement BMPs for spill containment.

Industrial users are required to provide containment for storage and loading/unloading areas for hazardous materials. Additional ordinances are unlikely to produce additional benefits beyond the current enforcement practices, and the benefits of such actions cannot be quantified in terms of reduction to CSO

pollutant loadings. Accordingly, industrial spill control is removed from future consideration for the alternatives analysis.

4.3 Green infrastructure

Green infrastructure (GI) is a source control that reduces runoff volumes, peak flows, and/or pollutant loads. GI utilizes the processes of infiltration, evapotranspiration, and capture for re-use to reduce the amount of runoff volume (USEPA, 2014). It can effective at increasing the time of concentration of remaining runoff and reducing pollutant loads through sedimentation and filtration. This technology can be used alone in a scalable manner, or it can be used in conjunction with gray infrastructure to reduce its size and cost. GI's benefits can extend beyond reducing the flow of water into CSSs during wet weather events. Through mimicking a more naturalized system, GI can deliver a broad range of ecosystem services or benefits to people, some of which include: improved community livability (aesthetics and property values), human health, air quality, water quality, groundwater recharge, wildlife habitats and connectivity, reduced heat island effects, reduced energy use, green jobs, and recreational opportunities (USEPA, 2014). It can also help reduce flooding and is flexible for addressing climate change (droughts or increased precipitation).

As described in Greening CSO Plans: Planning and Modeling Green Infrastructure for Combined Sewer Overflow (CSO) Control (USEPA, 2014), the EPA requires that any incorporation of GI into a LTCP include analysis in two areas:

1. Community and political support for GI.
2. Realistic potential for GI implementation

The permittees will assess the public support from stakeholders in the community and government for the GI alternatives through the implementation of the LTCP Public Participation Process Report. The realistic potential for the implementation has been screened within this section and refined further in the alternatives evaluation.

This evaluation is being conducted as an element of the development and evaluation of alternatives for compliance with the Permit. The permit requires "The permittee shall evaluate … the water quality benefits of constructing various remedial controls …" Therefore, the focus of this report shall be the impact of green infrastructure with respect to reductions in CSO volumes and frequencies, i.e. water quality benefits. Green infrastructure has many other benefits that do not pertain to water quality benefits and these benefits may result in green infrastructure being implemented apart from the CSO LTCP or in a decision to implement it at a greater cost than other alternatives. However, that decision must be made by the governing body and other stakeholders.

The goal is to evaluate the optimal implementation level for green infrastructure as it is applicable to the LTCP. Too little could result in missed potential benefits, while overcommitting may result in higher costs and maintenance efforts that are impractical to accomplish. Overcommitting may also result in a LTCP that cannot be accomplished because enough opportunities to install green infrastructure may not exist or be practical resulting in the permittees failing to meet its permit obligations. The following factors are considered in evaluating green infrastructure for applicability to the LTCP.

1. Green infrastructure must be sited, designed, constructed and maintained to provide a high level of confidence that it will continually perform as expected. To do this, the evaluation and analysis was conducted using guidance from:
   - *NJ Stormwater Best Management Practices Manual*, NJDEP, April 2004 Revised September 2014, February 2016, September 2016, November 2016, September 2017 & November 2018.
   - *Greening CSO Plans: Planning and Modeling Green Infrastructure for Combined Sewer Overflow (CSO) Control,* U.S. Environmental Protection Agency, March 2014.

- *Evaluating Green Infrastructure: A Combined Sewer Overflow Control Alternative for Long Term Control Plans,* NJDEP, January 2018.

2. The green infrastructure must be under the control of the Village to ensure that it remains in place and that maintenance occurs. Practices evaluated will be sited on land owned by the Village including:
   - Municipal owned rights-of-way
   - Public buildings
   - Libraries
   - Parks: At this time, it is uncertain if the Green Acres program will allow widescale use of parks for managing offsite stormwater, so parks will be considered to only manage stormwater generated within the park.
   - Property of affiliated public entities, like municipal schools districts.

3. Publicly available data will be utilized. Given the planning level of this evaluation, these sources of information may or may not be complete and will be subject to professional judgement and experience in their interpretation. Sources of data include:
   - Soil surveys
   - Aerial photography
   - Land use and land cover data sets
   - Property owner data sets
   - Site visits

The requirements for evaluating green infrastructure listed above are rigorous and can greatly increase the cost and limit the opportunities for green infrastructure. However, these requirements only apply to green infrastructure in the context of the LTCP and does not limit implementation of green infrastructure by the permittees or other entities apart from the LTCP. Green infrastructure can be implemented which is not formally incorporated into the LTCP and benefits may manifest themselves in future iterations of the system characterization. It may also be possible to expand the implementation of green infrastructure through public-private cooperation with formal agreement to perpetuate and maintain the green infrastructure, however, the measurable success of such a program involves factors beyond the control of the permittees and factors that cannot be evaluated at this time. As such, opportunities for additional green infrastructure exist, but not necessarily within the scope of the LTCP.

It may be possible to incorporate green infrastructure to reduce the need for gray infrastructure. This evaluation is intended to evaluate different levels of green infrastructure that could be practically implemented under the criteria above for the LTCP.

There are a variety of green infrastructure practices that can be applied to combined sewer areas. Each practice has advantages and disadvantages, which impact its applicability and performance. Considering different levels of implementation as well as combinations of practices, the number of possible alternatives exceeds a reasonable number. As such, the most common urban application of green infrastructure, roadside bioswales, was selected as a representative practice for evaluation. The subsections below explore the applicability of various types of green infrastructure, and Section 7.4.6 discusses how the reasonable extents of the practices were determined and how the overall implementation of green infrastructure was evaluated through the equivalent implementation of bioswales.

4.3.1 Green Roofs

Green roofs have bioretention media that collect runoff to promote evapotranspiration and achieve water quality standards through soil media filtration. They are typically shallow in depth (4-8”) based on the ability of the building to support the weight of the media, plantings, and captured rainfall. Green roofs may

be built in layers on a roof or installed as cells in crates. An example green roof section can be found in Figure 4-1.

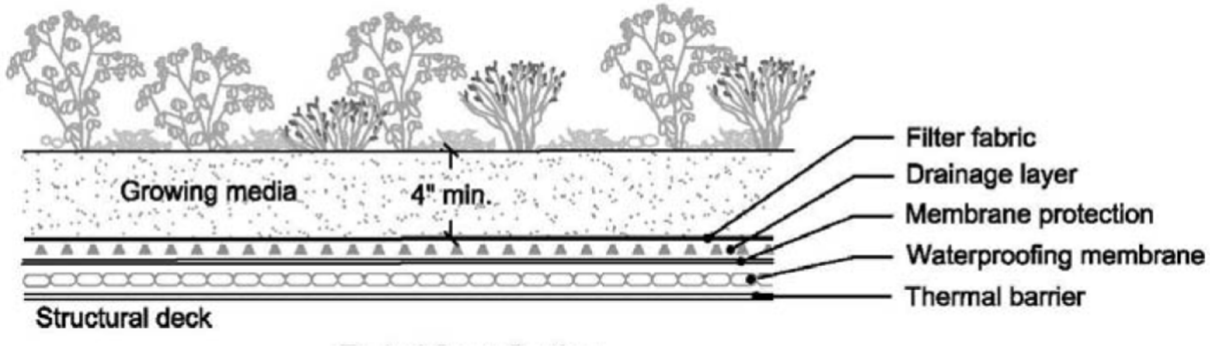


Figure 4-1: Example Green Roof Section

Green roofs may be applicable for use on buildings with flat roofs (recommended 1-2% slope) that have the structural capacity to support the weight of the media, plantings, and water. Structural improvements to an existing building to support the additional weight associated with a green roof are not typically recommended; therefore, this technology is more feasible on new construction. Green roofs can be installed in a section or across an entire roof. An overflow system is typically installed. The vegetation may require irrigation during the first 1-2 years to establish growth. Recommended maintenance for green roofs includes semi-annual maintenance of vegetation.

Many rooftop retrofits are required for this GI technology to have measurable impact. Most of the buildings in the CSS are privately owned. Implementing this technology on a scale that would have a measurable impact would require retrofits on private property. Green roofs are not considered suitable for roofs with greater than 20% slope, per the NJ Stormwater BMP Manual Chapter 9.14, November 2018 and thus will not be considered for residential areas, which typically have steeply sloped roofs and make up most of the combined area in the Village.

Green roofs could be implemented in the Village at the following locations:

- Existing municipal-owned roofs;
- Future municipal buildings; or
- New buildings in redevelopment areas.

It is difficult to retrofit existing buildings with green roofs, as it is unlikely that an existing building was designed to support the additional load. The process of certifying that an existing roof is structurally able to support the additional weight of a green roof is difficult and with an uncertain outcome. There is minimal Village owned roof area, see Figure 4-2, which could be used for green roofs. Thus, retrofit of existing Village owner buildings is not considered for inclusion in the LTCP.

The Village can investigate incorporating green roofs into future public buildings that will be constructed. However, no new municipal buildings, are currently being planned at this time, nor would any such building represent a meaningful proportion of the Village's impervious area, thus cannot provide a defined benefit for the purposes of the LTCP.

The best opportunity for implementing green roofs would be through redevelopment requirements. However, no redevelopment plans are underway in the combined sewer area. The most likely areas for redevelopment would be the industrial areas which are primarily separately sewered, or in the area east of the NJ Turnpike which is also separately sewered, leaving almost no opportunities through redevelopment to implement green roofs for CSO reduction.

The Village could encourage green roofs to realize the other benefits they provide, but they are not considered practical based on the technical constraints and anticipated redevelopment patterns in Village as a means of achieving reduction of CSOs. Accordingly, green roofs will not receive further consideration as part of the alternatives analysis.

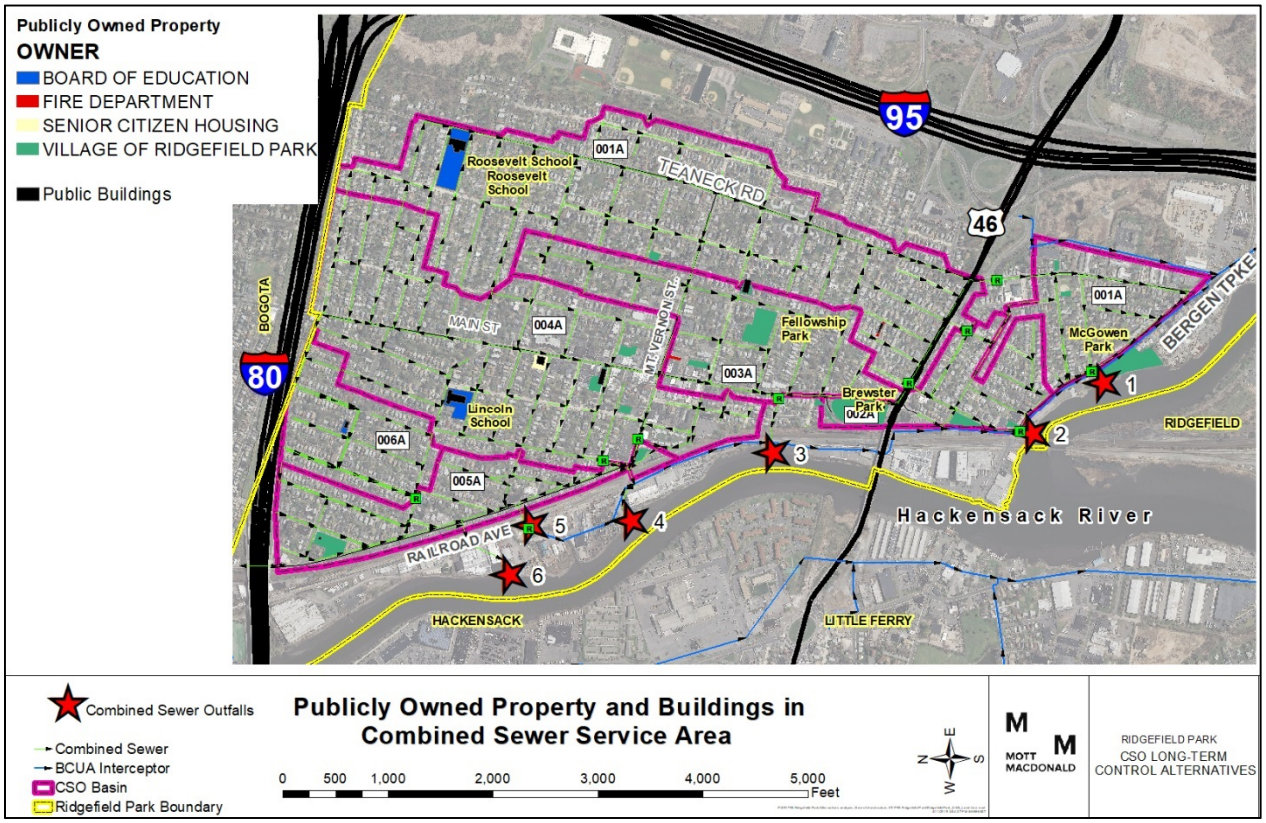


Figure 4-2: Village owned properties

4.3.2 Blue Roofs

Blue roofs collect runoff to promote evaporation (they do not have plantings) through detention. They are typically shallow in depth (4-8") based on the ability of the building to support the weight of the media and captured rainfall. Blue roofs may be built in layers on a roof or installed as cells in crates. Unlike green roofs, a blue roof may not provide any water quality benefits, unless filters or storage media are used specifically for this purpose. The water detained from blue roofs may be used on-site instead of being released with the appropriate modifications.

Blue roofs may be applicable for use on buildings with flat roofs (recommended 1-2% slope) that have the structural capacity to support the weight of the media and water. Structural improvements to an existing building to support the additional weight associated with a blue roof are not typically recommended; therefore, this option is more feasible on new construction. Blue roofs can be installed in a section or across an entire roof. An overflow system is typically installed to direct the detained water off the roof. Recommended maintenance for blue roofs includes semi-annual maintenance for clearing of debris. Similar to green roofs, blue roofs would require implementation on private property to have a measurable impact.

The Village could select to encourage green and blue roofs to realize the other benefits they provide, but these measures are not considered practical as a means of achieving reduction of CSOs based on the technical constraints, locations available under the Village's control where they could be implemented, and anticipated redevelopment patterns. Accordingly, for the same reasons as green roofs, blue roofs will not receive further consideration as part of the alternatives analysis.

4.3.3 Rainwater Harvesting

Rainwater harvesting is the collection and storage of rainfall from buildings to delay or eliminate runoff. The reduction in runoff volume varies based on the size of the rain barrel or cistern storage unit, and the reuse of the stored rainfall. A few typical reuse options are irrigation and vehicle washing. Indoor reuse options, such as toilet flushing and heating and cooling, may be possible if coordinated with building policies.

Rainwater harvesting is applicable to all types of buildings with gutters and downspouts but may be reserved for buildings where green or blue roofs are not appropriate (roof slopes greater than 2%). Storage units may be sized and installed for each downspout or for the building as a whole. Rain barrels, such as those in Figure 4-3, are typically used for residential installations and larger cisterns are typically used for non-residential applications. They are typically placed at grade but can be buried below grade if a pumping system for water reuse is provided. An overflow system is typically installed. Recommended maintenance for rainwater harvesting includes semi-annual clearing of debris in the piping or storage unit.

Figure 4-3: Rain Barrels

Similar to green and blue roofs, this technology is limited by the number of available roofs, most of which are private. Private residential uses of cisterns are much less common than on private commercial properties, but are encouraged to help reduce combined sewer overflows.

To effectively implement rainwater harvesting as part of a LTCP, the facility must be under the jurisdiction of the permittees. This is necessary to facilitate site access in perpetuity so that the controls remain fully functional and deliver the required performance to allow the permittees to comply with the permit requirements. As such, rainwater harvesting tools such as rain barrels on residential properties have not been considered feasible, limiting the number of locations where it can practically be installed as part of the LTCP. It is reiterated that this does not preclude promoting rainwater harvesting and encouraging residential rain barrel programs.

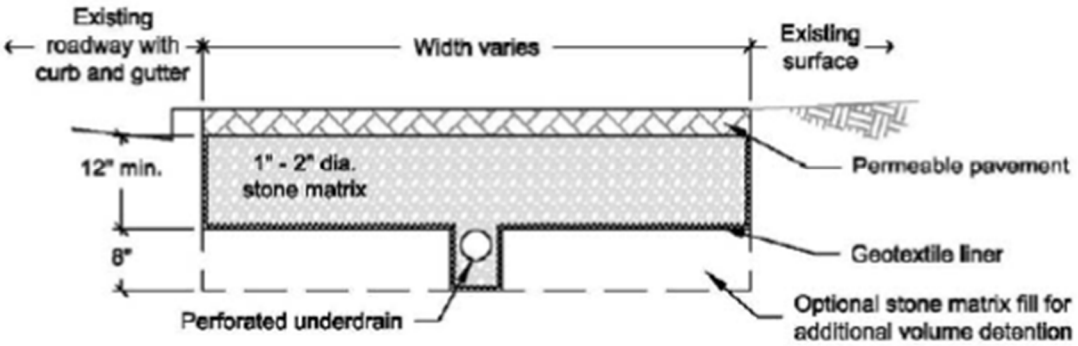
Rainwater harvesting tends to have minimal benefits to CSO reduction as the intent is to retain water for future use. Since it rains on average every three days, it is likely the rainwater harvesting storage tank would be full or partially full when the next rainfall occurs, relying on manual operation to empty the tank prior to rain, which would create an additional level of risk for the practice.

Due to the limitations associated with this technology, such as limited sites under the control of permittees, required scale, and reliance on other parties for performance, this measure is not likely to be a significant component of the LTCP and has been removed from further consideration.

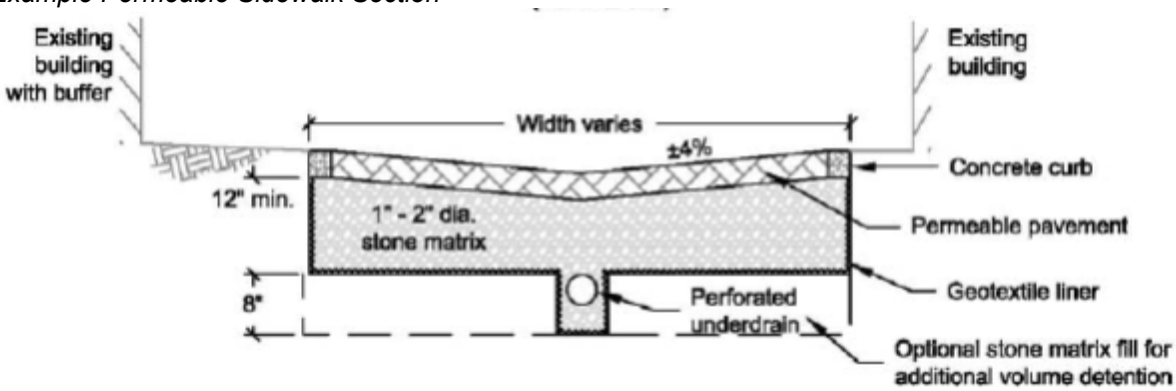
4.3.4 Permeable Pavements

Permeable pavements promote runoff infiltration and rely on a permeable substrate (engineered soils) to store runoff and remove pollutants. There are different types of permeable pavements, most commonly constructed with asphalt, concrete, or pavers. Permeable asphalt and concrete are similar to traditional mixes except that the amount of fine aggregates is reduced or eliminated. Permeable pavers are individual paver units laid together to create a paved surface. The depth of the permeable substrate, anywhere from 3-10 feet, will have the largest impact on runoff volume reduction. Substrate design may incorporate stormwater retention chambers to increase storage volume. Underdrains may be necessary depending on the local soil types, depth of substrate, and groundwater elevation.

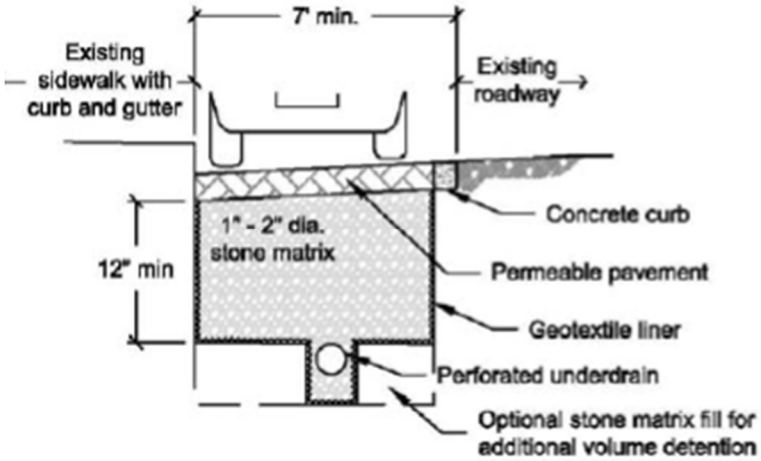
Permeable pavements are recommended for low traffic and low speed traffic areas such as sidewalks, parking lanes, parking lots, driveways, and alleys. Figure 4-4 below show slightly different permeable pavement details for each of these surfaces. Recommended maintenance for permeable pavement includes semi-annual inspection and vacuuming. Preventative maintenance is also necessary to minimize the introduction of soil and other fine particles that could clog the pavement pores.



Example Permeable Sidewalk Section



Example Permeable Alley Section



Notes:
1. Depth of stone matrix will be determined by structural requirements, storage desired, and permeable pavement type.
2. Top of existing sewer and other utilities shall be a minimum of 1' below the invert of the stone matrix.
3. All underdrains will connect to the existing storm sewer system via solid pipe.

Example Permeable Parking Lane Section

Figure 4-4: Example Permeable Pavement Sections

Permeable pavement is typically recommended for low traffic areas; thus, it may be feasible to re-pave municipal parking areas with permeable pavement, specifically the parking stalls and not the travel lanes. This is a common approach and is reflected in Example 1 of Chapter 9.7 of the New Jersey Stormwater BMP Manual (updated November 2016). A loading ratio of 4:1 (ratio of impervious area to green practice area) will be used as recommended by Table 2-1 of the NJDEP's Evaluating Green Infrastructure: A Combined Sewer Overflow Control Alternative for Long Term Control Plans, January 2018. The NJ Stormwater BMP Manual also recommends permeable pavement not be used on slopes greater than 5%, as can be seen in Figure 4-5 much of the Village is steeper than 5%, limiting locations where permeable pavement can be installed.

The New Jersey Stormwater BMP Manual also requires 1 foot of separation from the seasonal high groundwater for non-infiltrating practices and 2 feet for infiltrating practices, a choker course and adequate volume to hold the runoff from the water quality storm (1.25" of rain and 1.0" of runoff from impervious surfaces with CN-98). It is also recommended to extend the reservoir course below the frost line. These requirements may push the permeable pavement box below the seasonal high groundwater, violating the separation requirement. While thought to be deep, in some locations, the groundwater level may be shallow, the groundwater separation criterium may greatly limit locations where permeable pavement can be implemented.

Parking lanes within the Village offer some area to implement permeable pavement. It is noted that there is a high demand for street parking in the Village, and the temporary unavailability of parking associated with installation of the permeable pavement make this area less favorable. There are also numerous utilities in the parking lanes which could be very difficult to work around or relocate.

Sidewalks offer a limited opportunity to install permeable pavement. Sidewalks are generally narrow, so would offer a relatively small area to implement this practice. The sidewalks are above the roadway and roof leaders are generally piped to the street gutter, resulting in a low loading rates as adjacent impervious areas are not contributing runoff to the sidewalk areas.

As such, permeable pavement will be considered for the stall areas of municipal parking areas and selected parking lanes, but a maximum of 10% of available locations will assumed to be viable because of the issues noted above. Due to potential utility conflicts sub-surface chambers within the permeable pavement reservoir layer will not be considered. It is noted that this is just for evaluation purposes and the

proposed analysis will report on the impacts of a wider range of green infrastructure implementation. If permeable pavement is found to be functionally and economically effective, additional investigations can be undertaken.

Figure 4-5: Village of Ridgefield Park surface slopes >5%

4.3.5 Planter Boxes

Planter boxes are bioretention cells that collect runoff and promote runoff infiltration. These walled units are similar to bioswales and free-form rain gardens as vegetated depressions (12-24") that rely on ponding and a permeable substrate (engineered soils) to store runoff and remove pollutants. The depth of the permeable substrate, anywhere from 3-10 feet, will have the largest impact on runoff volume reduction. An Example Planter Bumpout Section can be found in Figure 4-6. Substrate design may incorporate stormwater retention chambers to increase storage volume. Properly designed planter boxes limit ponding to 3-6 hours after a storm. Ponding overflow pipes and underdrains may be necessary depending on the local soil types, depth of substrate, and groundwater elevation. The vegetation promotes evapotranspiration to reduce the volume of the stored runoff.

There are two primary sizes of planter boxes for use based on the drainage pattern in developed areas: sidewalk planter boxes and bumpout planter boxes. Sidewalk planter boxes may also be more specifically referred to as a Tree Well BMP, a Tree Well with Soil Panels, a Continuous Planting Strip, Mid-Sidewalk BMP, or a Back of Sidewalk BMP. Sidewalk planter boxes are depressed below the elevation of the existing sidewalk. Bumpout planter boxes are larger units that extend from the sidewalk curb into an area of a parking lane. Curb cuts into planter boxes allow roadway runoff to enter the cells and overflow to street inlets once the maximum ponding depth has been reached. Planter boxes are often suggested for use in regularly spaced intervals in the downstream drainage path in areas of impervious cover.

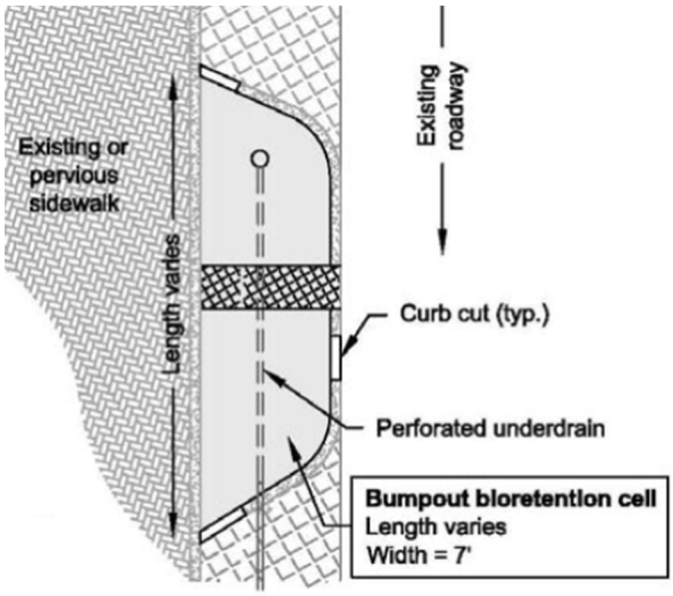


Figure 4-6: Example Bumpout Planter Box Layout

Recommended maintenance for planter boxes includes semi-annual inspections and improvements to vegetation and mulch, and annual inspection of overflow pipes and underdrains, if applicable. Inspection after a large storm is also recommended. If there is evidence of ponding after 48 hours, mulch replacement or overflow pipe cleaning may be necessary.

Planter boxes are well suited for highly developed areas where space allows. They can be installed block by-block to contain, infiltrate, and evapotranspirate stormwater runoff. It is noted that the existing Village roads are quite narrow and only limited if any opportunities for bumpouts. As planter boxes are similar in concept to bioswales and bioswales are readily incorporated in the hydraulic modeling software used, the evaluation of green infrastructure with bioswales as the representative GI technology considers the alternate use of planter boxes based on future site-specific selections.

4.3.6 Bioswales

Bioswales are vegetated channels that reduce runoff velocity and promote runoff infiltration. These are linear channels with shallow depressions (6-12”) that incorporate vegetation and a permeable substrate (engineered soils). As a channel, runoff not infiltrated does not pond, but flows through the swale and is conveyed elsewhere. The channels, especially those with slopes greater than 6%, may incorporate check dams to assist in reducing runoff velocity and promote infiltration and pollutant removal. A design example for a bioswale is found in Figure 4-7. Bioswales are typically suggested for use in parks and areas of natural cover since they primarily reduce runoff velocity and have a low volume reduction per square foot. Due to their linear nature, bioswales may also be effective in the buffer between open space areas and impervious areas with high volumes of runoff such as roads and parking lots. Recommended maintenance for bioswales includes semi-annual inspections and improvements to vegetation and mulch.

Bioswales have been widely implemented in areas such as New York and Philadelphia but may have limitations in the narrow rights-of-way. These programs are relatively new and there is not comprehensive data on the long-term maintenance costs and community wide performance of bioswales for CSO reduction. Nevertheless, they are easily modeled in InfoWorksICM and can be applied in a distributed fashion. They can also be used as a surrogate for modeling other green infrastructure practices. Accordingly, bioswales will be further evaluated in Section 7.

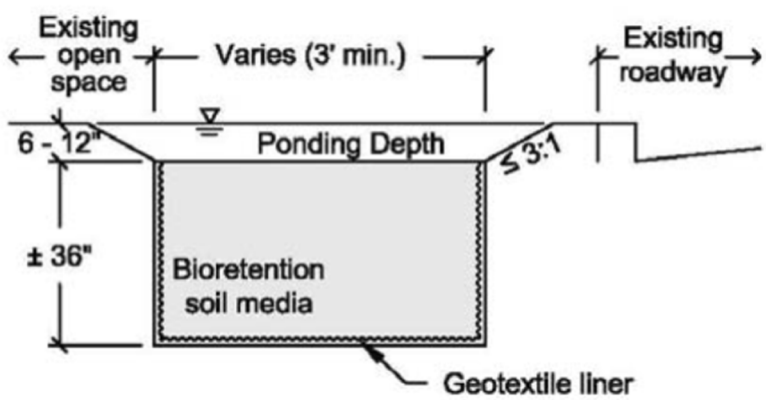


Figure 4-7: Example Bioswale Detail

4.3.7 Free-Form Rain Gardens

Rain gardens are bioretention basins that collect runoff and promote runoff infiltration. These are vegetated depressions (12-24”) that rely on ponding and a permeable substrate (engineered soils) to store runoff and remove pollutants. The size and shape of rain gardens can be tailored to site- specific needs, but the depth of the permeable substrate (anywhere from 3-10 feet) will have the largest impact on runoff volume reduction. Substrate design may incorporate stormwater retention chambers to increase storage volume. Properly designed rain gardens limit ponding to 3-6 hours after a storm. Ponding overflow pipes and/or underdrains may be necessary depending on the local soil types, depth of substrate, and groundwater elevation. The vegetation promotes evapotranspiration to reduce the volume of the stored runoff, and infiltration helps improve water quality. An example of a rain garden design is found in Figure 4-8. Rain gardens are recommended for use in low points in parks and areas of natural cover so they can blend in seamlessly with a grassed buffer and enhance the vegetation without appearing to be a stormwater control mechanism. Locations near the transition from pervious to impervious cover can provide runoff reduction for nearby impervious areas. Recommended maintenance for rain gardens includes semi-annual inspections and improvements to vegetation and mulch and annual inspection of

overflow pipes and underdrains, if applicable. Inspection after a large storm is also recommended. If evidence of ponding exists after 48 hours, mulch and/or soil replacement or overflow pipe cleaning may be necessary.

Rain gardens are functionally similar to bioswales but must be evaluated for suitability on a site-specific basis. They are a widely-used stormwater best management practice, effective at containing, infiltrating and evapotranspirating diverted runoff. They also require minimal maintenance of vegetation and mulch, provided there is regular cleaning of overflows and underdrains. Underground infiltration beds or detention tanks can also be utilized to increase storage. There are limited locations for siting rain gardens within the control of the Village. While parks offer opportunities for rain gardens, at this time they are only allowed to be used to treat onsite runoff. Since the parks are highly pervious, applying rain gardens within them will produce minimal benefits. The Village may elect to site additional rain gardens within the limited available municipal owned land and continue to promote them on private property.

As rain gardens are similar in concept and function to bioswales and bioswales are readily incorporated in the hydraulic modeling software used, the evaluation of green infrastructure with bioswales as the representative GI technology considers the alternate use of rain gardens based on future site-specific selections.

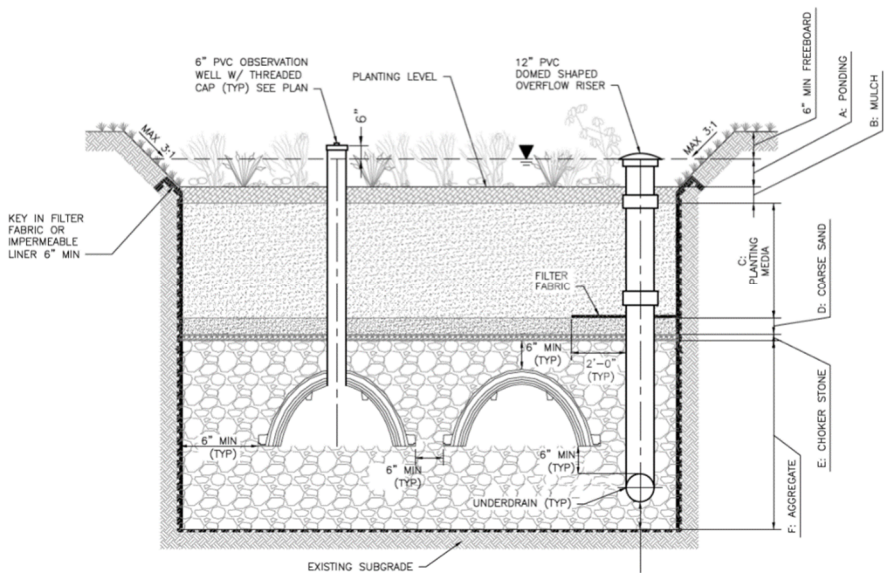


Figure 4-8: Example Rain Garden Section

4.4 Infiltration and Inflow Control

4.4.1 Infiltration/Inflow (I/I) Reduction

Excessive infiltration and inflow can consume the hydraulic capacity of a collection system and increase overall operations and maintenance costs. Inflow comes from sources such as roof drains, manhole covers, cross connections from storm sewers, catch basins, and surface runoff. Within a CSS, surface drainage is the primary source of inflow. Infiltration comes from groundwater that seeps in through leaking pipe joints, cracked pipes, manholes, and other similar sources. The flow from infiltration tends to be constant, but at a lower volume than that of inflow. Identifying I/I sources is labor intensive and requires specialized equipment. Significant I/I reductions can also be difficult and expensive to achieve. However,

the benefit of a good I/I control program is that it can save money by extending the life of the system, reducing the need for expansion, and lowering treatment costs.

The Village of Ridgefield Park has no authority beyond its borders to enforce I/I reduction. Within Ridgefield Park I/I is not known to be a significant issue. During the Characterization the groundwater inflow was estimated to be just under 0.5 MGD. If 50% of the I/I could be removed through a full rehabilitation of the entire sanitary system, it would free up 0.25 MGD of capacity which is minimal and could not justify the expense of such a program.

4.4.2 Advanced System Inspection and Maintenance

System inspection and maintenance programs can provide valuable knowledge about the condition of the CSS infrastructure, which is beneficial for planning, inspection, and maintenance activities. This can help ensure design flow capacity is consistently available to prevent CSO events. This technology offers relatively minor advances towards meeting the primary and secondary goals of the LTCP.

The Village maintains its collection system regularly and is not aware of problem areas that could materially benefit from advanced inspection and maintenance. The proper maintenance of the system is reflected in the system baseline. As an appropriate and practical program is currently in place, and will be continued, this measure will not be considered further for the LTCP. This does not preclude the adoption of progressive developments in sewer inspection and maintenance activities, which, if implemented, would be reflected in future iterations of the system characterization.

4.4.3 Combined Sewer Flushing

This type of O&M practice re-suspends solids that have settled in the CSS and flushes them downstream. This practice consists of introducing a controlled volume of water over a short duration at key points in the collection system using external water from a tank truck, pressurized feed, or by detaining the CSS flow for a period, and then releasing it. Overall, this practice helps reduce the amount of settled solids that are resuspended and discharged during significant wet weather events. This measure is most effective when applied to flat collection systems since solids are more likely to become deposited on flat grades. The Village performs sewer cleaning and flushing regularly and as needed. In general, the Village's pipes are steep and minimal accumulation of sediment occurs. The current program is implemented at a satisfactory level and will be continued. As such, this measure will not be considered further.

4.4.4 Catch Basin Cleaning

Catch basin cleaning reduces the transport of solids and floatables to the CSS by regularly removing accumulated catch basin deposits. Methods to clean catch basins include manual, bucket, and vacuum removal. Catch basin cleaning can be effective in reducing floatables in combined sewer; however, it is not effective at bacteria reduction or volume reduction, nor is it particularly effective at Biochemical Oxygen Demand (BOD) reduction. The Village has an existing catch basin cleaning program that is implemented at a satisfactory level and is reflected in the baseline conditions. As a current practice that will be continued, this measure will not be considered further.

4.5 Sewer System Optimization

Sewer system optimization involves collection system controls and modifications that affect CSO flows and loads once the runoff has entered the collection system. Options for system optimization include measures that maximize the volume of flow stored in the collection system or maximize the capacity of the system to convey flow to the treatment plant. Sewer system optimization techniques have no impact on water quality, but do have the potential to reduce the volume of CSO events.

Sewer system optimization is being investigated through possible increased conveyance and storage capacity in the collection system. The Ridgefield Park system is currently operated to maximize inline storage and conveyance to the BCUA WRRF, thus available additional benefits are likely minimal, but should be optimized where possible. Possible strategies which could be combined with pipeline alternatives include:

- Maximizing existing inline storage – Storage within existing pipes can be used to retain combined sewerage rather than letting it overflow, providing this can be done without adversely impacting the collection system's hydraulic performance.
- Additional conveyance – Additional conveyance can be costly and would require additional maintenance to keep new structures and pipelines operating.
- Regulator modifications – The capacity of the Branch Intercepting sewers limits the amount of flow that enter the interceptor system. It is unlikely that regulator modification could increase the amount of combined sewerage reaching the BCUA WRRF and receiving treatment without additional upgrades to the interceptor system.
- Outfall consolidation / relocation – Based on previous reports outfall consolidation results in lower operational requirements; reduces permitting/monitoring; and is cost-effective when used in conjunction with storage & treatment technologies. Combining and relocating outfalls should also lower operating costs and reduction of CSO discharges. Consolidation can also be used to direct flow away from specific areas such as sensitive areas.
- Real time controls (RTC) are typically used in conjunction with inline storage to minimize the rate and volume of CSO discharges. The process will require periodic inspection of flow elements; a highly automated system to minimize operator errors; and an increased potential for sewer backups. Since there is little or no additional storage capacity in the Village sewer system this control will not be considered any further in the alternative analysis.

4.5.1 Maximizing Existing Storage Capacity in the Collection System

Options for increased storage capacity rely on maximizing the volume of flow stored in the collection system. Maximizing the use of the existing system involves ongoing maintenance and inspection of the collection system, and can include minor modifications/repairs to existing structures to increase the volume of flow retained in the system, these efforts are in place and ongoing. An analysis was conducted to determine if raising the regulator weirs could provide additional storage without impacting flooding. It was assumed that any such raising would be accomplished with bending weirs because raising the existing static weirs could increase upstream hydraulic grades, increasing flooding potential and creating liability. The available volume in the existing pipe is plotted in Figure 4-9 below, as can be seen there is minimal volume available, which is to be expected since the sewer pipes in Ridgefield Park tend to be smaller in diameter and steeper.

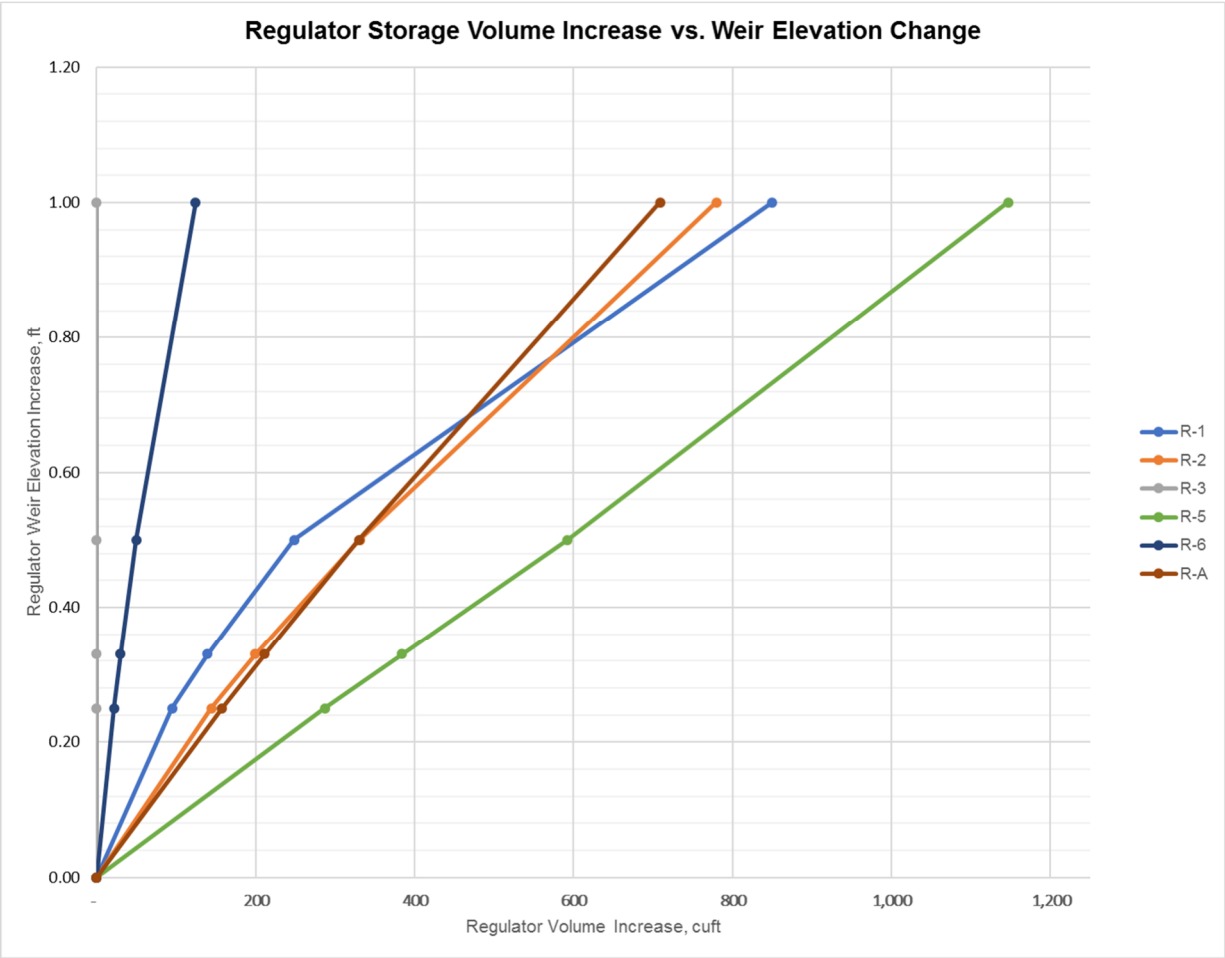


Figure 4-9: Regulator storage volume versus weir raising

4.5.2 Additional Conveyance

Conveyance is a technology that transports the combined sewage out of an area to a location where the flow can be stored, treated, or discharged where direct public contact with the water is less likely. Conveyance is accomplished by providing additional conveyance pipes or upsizing the existing conveyance pipes to a greater capacity. This practice can effectively reduce overflow volume and frequency in the affected areas. Large conveyance projects can be expensive and may require a lengthy permitting process.

Providing additional conveyance his limited benefits. There are no know conveyance issues within the existing system that would justify the increasing of pipe sizes. There is also no compelling need to relocate outfalls from one area to another as all outfalls discharge to similar waterbodies in similar areas. Additional conveyance may be employed as a complimentary technology to other alternatives to consolidate flows for storage or treatment.

4.5.3 Regulator Modifications

A CSO regulator can be uniquely configured to control combined sewer overflow frequency and volume. The existing overflow control structures may be modified based on site-specific conditions. For example, regulator modifications may include increasing the overflow weir height and length or raising the overflow

pipe elevation. This technology is especially effective for CSO outfalls with high overflow frequency and low overflow volume, because the additional volume held back in the system is small and less likely to have negative impacts on upstream conditions.

Regulator modifications may be considered in greater detail for the alternatives evaluation as it is a technology that will likely be complimentary to a primary alternative or be useful but of limited application. The BCUA has indicated there is limited capacity within the existing interceptor system so simply enlarging the regulator orifices will not allow additional flow to be conveyed for treatment and may create issues elsewhere in the system.

4.5.4 Outfall Consolidation/Relocation

Consolidation of one or multiple outfalls can help eliminate CSO discharges in sensitive areas. Outfall consolidation may require modification or relocation of an outfall, the installation of additional conveyance to accommodate new flow configurations, and may also require additional permitting with government agencies. This practice typically lowers O&M requirements for the CSS by limiting the number of outfall structures that need to be monitored. Outfall consolidation works best in areas where outfalls are in close proximity to each other and require limited additional conveyance.

Like regulator modifications, outfall consolidation can be effective at reducing high frequency, low volume CSOs. This practice typically does not add a significant amount of extra capacity to the CSS (depending on the amount of conveyance pipe associated with the consolidation project), so its impact on infrequent, large volume CSO events can be limited. Modeling can be performed to determine the level of impact that outfall consolidation will have in terms of reducing the number of CSO events.

Given the spatial distribution of the CSS, outfall consolidation/relocation will be considered in the alternatives evaluation as a potential complimentary technology to a primary alternative.

4.5.5 Real Time Control

Real Time Control (RTC) is a highly automated system in which sewer level and flow data are measured at key points in the sewer system and used to operate systems controls to maximize the storage capacity of the CSS and limit overflows. The collected data is typically transferred to a control device where program logic is used to operate gates, pump stations, inflatable dams and other control components. Local dynamic controls are used to control regulators to prevent flooding and system wide dynamic controls are used to implement control objectives, such as maximizing flow to the treatment plant or transferring flows from one portion of the CSS to another to fully utilize the system. Predicative control, which incorporates use of weather forecast data, is an optional feature, but is complex and requires sophisticated operational capabilities.

RTC involves the installation of numerous mechanical controls, which require upkeep and maintenance, and can only reduce CSO volumes where in-system storage capacity is available. Given the size and extents of the existing sewer system and its limited available storage volume, RTC programs are not considered to be an effective primary technology for the Village. However, RTC may serve as a complementary technology used with a primary alternative, such as to control tank dewatering rates or tunnel inflow gates.

4.6 Storage

The objective of storage is to reduce overflows by capturing and storing wet weather flows, greater than CSS conveyance/treatment plant capacity, for controlled release back into the system once treatment and conveyance capacity have been restored. A storage facility can attenuate peak flows in the CSS and provide a relatively constant flow into the treatment plant after peak events. Storage technologies do not prevent water from entering the CSS or treat bacterial loads in CSO discharge, but are very effective at

reducing or eliminating CSO events. Storage technologies may have high construction and O&M costs depending on site conditions, but are scalable and a very reliable means of achieving CSO control goals.

4.6.1 Linear Storage

Linear storage is provided by underground storage facilities that are sized to detain peak flows during wet weather events for controlled release back into the system after the event. In-line linear storage (storage in series with the CSS) can be provided by over-sizing the existing interceptors for conveyance, as described in the previous section, whereas off-line linear storage (storage parallel to the CSS) can be provided by installing new facilities such as tunnels and pipelines.

4.6.1.1 Pipelines

Large diameter parallel pipelines or conduits can provide significant storage in addition to the ability to convey flow. Pipelines are typically constructed between an overflow point and a pump station or treatment facility. The pipelines include discharge controls to allow flow to be stored within the pipeline during wet weather events, and slowly released by gravity following the event. The conveyance to the desired endpoint depends on the additional capacity necessary to handle the increased flow and is developed concurrently with the pipeline. A force main pipeline constructed from a pump station relies heavily on the increased flow capacity as the storage benefits are negligible. Pipelines have the advantage of requiring less area for construction compared to point storage, and are easier to site in the public right-of-way. If trenchless technologies can be utilized, such as horizontal directional drilling (HDD), land requirements can be reduced even further.

One disadvantage of pipelines is that a larger volume is typically required to accommodate combined sewer storage needs. The installation of large diameter pipelines is typically less cost effective than tunneling, and the installation of smaller diameter pipes typically requires a significant length to provide adequate storage. Additionally, the installation of pipelines is very disruptive, typically requiring open trenches and the temporary closure of public streets. Considering the required volumes for the CSO control levels, parallel pipeline storage is not practical and is eliminated from further consideration.

4.6.1.2 Tunnels

Tunnels provide more storage volume than pipelines, while maintaining the ability to convey flow. Tunnel excavation is accomplished completely underground, and therefore results in minimal surface disruption and requires little right-of-way, outside of drop shafts and conveyance piping to the drop shafts. Overall costs for tunnels can be high, but their cost per million gallons of storage for high volume storage is reasonable compared to other storage technologies, depending on local geology. Tunnels are typically used in congested urban areas where available land is scarce and connections to most, if not all, of the CSO regulators can be made.

While there are many challenges associated with constructing a tunnel in the Village, because of the large storage volume provided, relatively low permanent surface impacts and successful application of tunnels in other CSO communities, tunnels will be considered for evaluation as a feasible alternative for significant CSO control.

4.6.2 Point Storage

Point storage can be provided by above-ground or underground storage facilities such as tanks and equalization basins. These off-line facilities are placed at specific points in the system to detain peak flows for controlled return to the system, reducing CSO discharge volume and bacterial loading.

4.6.2.1 Tanks

This technology reduces overflow quantity and frequency by storing all or a portion of diverted wet weather combined flows in off-line storage tanks. Stored flows are returned to the interceptor for conveyance to the treatment plant once system capacity becomes available. Storage tanks are generally fed by gravity and the stored flow is typically pumped back to the interceptor after the storm. The benefit of off-line storage tanks is that they are well suited for early action projects at critical CSO outfalls. Storage tanks capture the most concentrated first flush portion wet weather peak flow and help to reduce the downstream capacity needs for conveyance and treatment.

A disadvantage of off-line storage tanks is that they typically require large land area for installation, which may not be available in congested urban areas. Off-line storage tanks may have higher costs per volume captured compared to other technologies. Additionally, if the existing sewers are deep, then the storage tank must also be deep, which results in additional construction costs. Operation and maintenance costs can also be high, especially if the application includes provisions for partial treatment and discharge, rather than simple storage and bleed-back to the sewer. Depending on the application, odor problems may also be an issue. However, storage tanks can be a very effective means of CSO control.

While siting of storage tanks within a densely populated areas can be challenging, there are some potential areas available in the Village as well as options for consolidated storage. Storage tanks provide effective reduction of CSO volumes and provide full treatment by allowing the retained volume to be conveyed to the treatment plant. Accordingly, storage tanks will be considered for evaluation as a feasible alternative for significant CSO control.

4.6.2.2 Industrial Discharge Detention

This technology would require industrial users to build and maintain storage basins to hold industrial discharge during wet weather events and subsequently release it back to the CSS. This would limit the peak wet weather flow to the WWTP. The effectiveness of this technology is limited to the number of industrial users upstream of CSO regulators. Significant Indirect Users (SIU) locations, associated CSO outfalls, and discharge volumes and constituents, are discussed in the BCUA report.

4.7 Sewage Treatment Plant Expansion or Storage

4.7.1 Additional Treatment Capacity

CSOs can potentially be reduced by increasing the treatment capacity of plant. Other technologies can make use of this increased treatment capacity by providing more flow to the plant instead of CSO outfalls. BCUA is investigating additional treatment capacity as part of their alternatives analysis report.

4.7.2 Wet Weather Blending

Blending is the practice of allowing portions of the wet weather peak flow to bypass certain treatment facilities at the plant. In blending, wet weather flows are typically routed through primary treatment, allowed to bypass secondary and tertiary treatment, and then recombined with effluent from all processes prior to disinfection and discharge to the environment. This practice may require increasing the capacity of primary treatment and disinfection facilities, but does not require the upsizing of secondary treatment facilities, which can be the costlier components. Other technologies can make use of the increased wet weather peak flow capacity by providing more flow to the plant instead of CSO outfalls. BCUA is investigating wet weather blending as part of their alternatives analysis report.

4.8 Sewer Separation

4.8.1 Roof Leader Disconnection

Roof leaders may be directly connected to the CSS. Roof leaders can be disconnected in order to divert stormwater elsewhere and to delay its entry into the CSS. Depending on the neighborhood, roof leaders may be run to a dry well, vegetation bed, lawn, storm sewer, or street. This technology typically has limited benefits in dense urban areas due to the lack of pervious areas available to divert flow for infiltration. Unfortunately, the most feasible rain leader disconnection scheme in these areas is usually diversion to the street. In this case, disconnection can lead to nuisance street flooding and is only able to briefly delay the water from entering the CSS through catch basins.

Roof leader disconnection is typically much more effective in areas with separate sewers where the roof leader was previously connected to a sanitary sewer, since the diverted rainwater does not have a direct path back into the system. Roof leader disconnection can be effective for both sanitary and storm sewers; however, the effect of this measure is highly contingent upon the extent of roof leaders in the system, site specific conditions, and the ability to find an adequate location to divert stormwater flow from the roof leader.

The Village is a highly urbanized area and there is limited opportunity for infiltration of storm runoff from roof leaders to pervious areas. A broad private property roof leader disconnection program requires coordination with and acceptance by property owners. Experiences in other locations indicate that enforcement is difficult to achieve. To be effective the NJ Stormwater BMP manual recommends disconnected roof leader run a minimum of 25 feet over pervious areas on a uniform slope to allow the roof leader discharge to achieve sheet flow conditions. As can be seen inFigure 4-10, in many cases there is less than 25 feet of lawn area between the house and the road, and only a partial disconnection could be accomplished due to driveways. Since most opportunities for roof leader disconnection are in residential areas, long term enforcement of separation would be difficult. Roof leader disconnection may be coupled green infrastructure technologies, but it has limited application and will not be considered further in the evaluation.

Figure 4-10: Google Earth Street View of typical Ridgefield Park Street

4.8.2 Sump Pump Disconnection

Buildings with basements below the ground water table often are kept dry by using sump pumps. In some cases, these pumps discharge to the CSS or sanitary sewers. Sump pump disconnection diverts this pumped groundwater flow to a location other than these sewers. The Village currently prohibits sump pump connection directly to the combined sewer system for new construction, which should reduce the impact on the system over time. Sump pump disconnection programs are typically more effective in separate sewer areas and are subject to the same limitations as roof leader disconnection programs (e.g., extent, site conditions, and diversion options). While sump pump disconnection is generally a good practice, the measure has limited application for the Village's combined sewer system and will not be considered further in the alternatives evaluation.

4.8.3 Combined Sewer Separation

Sewer separation is the conversion of a CSS into a system of separate storm sewers and sanitary sewers. This can be accomplished by installing a new sanitary sewer and using the existing combined sewer as a storm sewer or vice versa. This practice can be very expensive, disruptive to the public, and difficult to implement, especially in downtown areas or other densely developed urban environments. It typically requires closure of public streets while the new pipes are installed, and new connections are made. The Village has accomplished some partial separation in the northwest corner of the village and along Teaneck Road.

Historically, sewer separation has been found to have a very high cost if implemented outside of large-scale redevelopment. Creating new stormwater outfalls also present unique water quality challenges, so separated sewers are not necessarily an effective long-term water quality improvement solution. This is because stormwater contributes pollutants that affect water quality. Currently, sewer separation projects are subject to water quality requirements by the State when Land Use permits are required. Draft rules formalizing and increasing the requirements on sewer separation projects were recently issued. It is anticipated that stormwater outfalls will be subject to additional regulations in the future that will eventually require progressively more stringent treatment prior to discharge. This may make separation infeasible in the future and makes current cost estimates highly uncertain.

Despite its many challenges, sewer separation is a primary technology that would eliminate combined sewer overflows. For this reason, this alternative is maintained for future consideration in the alternatives evaluation.

4.9 Treatment of CSO Discharge

Treatment technologies are intended to reduce the pollutant loads to receiving waters by treating wet weather flows prior to discharging to the environment. Specific technologies can address different pollutant constituents, such as settleable solids, floatables, or bacteria.

Satellite end of pipe treatment has been used successfully in other places, and the potential exists for installing end-of-pipe treatment in the Village. End of pipe treatment is often operator intensive, with the permittee operating several small-scale wastewater treatment plants. It has also been indicated that providing primary treatment and disinfection through satellite end of pipe treatment may not be considered adequate in the future and additional facilities may be required.

The siting analysis described in Section 6 can be considered for possible end-of-pipe treatment sites in the alternatives evaluation. The proposed treatment facilities will consist of pretreatment (screenings), high rate primary treatment, and disinfection with interim pumping also required. To limit the alternatives, a representative set of technologies to provide the treatment train described will be selected. While an extensive list of technologies is screened below, it is understood that the LTCP may not select the same

exact set of technologies and that pilot testing will ultimately be required to select a technology for construction.

4.9.1 Course Solids and Floatable Removal

4.9.1.1 Vortex Separators

Vortex separation is a process that removes floatables and settleable solids from a wastewater stream by directing influent flow tangentially into a cylindrical tank, thereby creating a vortex. The vortex action causes settleable solids to move toward the center of the tank, where they are concentrated with a fraction of the influent flow and directed to the underflow at the bottom of the tank. The underflow is then conveyed downstream to the treatment plant. The remaining influent flow travels under a baffle plate, which traps any floatables, and then over a circular baffle located in the center of the tank. It is then discharged to receiving waters or conveyed to storage or treatment devices for further processing. This technology does not address CSO volume or bacteria reduction and will only help meet water quality and CSO control goals if used in combination with other technologies.

Vortex separators have been found to be effective at removing larger inorganic material, but the performance for the removal of smaller and lighter particles is limited. Improved performance has been achieved through chemical addition and use of large tanks to store the underflow. Due to these factors, vortex separators were not selected as the representative pretreatment technology.

4.9.1.2 Screens and Trash Racks

Screens and trash racks consist of a series of vertical and horizontal bars or wires that trap floatables while allowing water to pass through the openings between the bars or wires. They can be installed at select points within a combined sewer system to capture floatables and prevent their discharge during CSO events. Due to limited hydraulic capacity, screens are most suitable for small outfalls. Trash racks or static screens can be located on top of an overflow weir or near the outfall. These devices are inexpensive but usually incur high maintenance costs due to their tendency to become clogged. Frequent cleaning (after every storm) is usually required to prevent clogging, which can cause serious flooding and sewer backups. Mechanical screens can remove floatables and some solids without frequent manual cleaning. This can be a significant advantage when compared to the maintenance requirements and the potential for flooding caused by a clogged static screen. However, most mechanical screens (climber screens, cog screens, or rake screens) require structural modifications to the outfall chamber to house and protect the screens. If weir-mounted mechanical screens are used instead, they require much less headroom and can be retrofitted into an existing overflow chamber with little to no structural modifications. This technology does not address CSO volume or bacteria reduction and would be used in combination with other technologies to meet water quality and CSO control goals.

The Village already employees netting facilities to control solids and floatables on its outfalls. These existing solids/floatable control facilities would remain in place where feasible. For new end-of-pipe treatment considerations, a screening facility to remove solids less than 0.5 inches in size will be considered as an ancillary process to support the primary objective of solids removal and disinfection for bacteria reduction.

4.9.1.3 Netting

Netting systems involve mesh nets that are installed within a CSO outfall to capture floatable material as the CSO discharges into the receiving water. The nets are nylon mesh bags that can be concealed inside the CSO outfall until an overflow occurs. The advantage of this technology is that it captures floatables inexpensively, and can provide a base level of control at some CSO sites. However, the operation and maintenance requirements are high, and end-of-pipe netting installations can have some negative aesthetic impacts associated with the filled nets being visible at the pipe outlet until they are replaced. This

technology is strictly for floatables control and would not address water quality and CSO control goals alone.

As the Village already has netting facilities for solids and floatables control, nets would provide no additional benefit and will not be considered further in the LTCP.

4.9.1.4 Containment Booms

A containment boom is a temporary floating barrier used to contain floatables entering the waterway from a CSO outfall. Containment booms are used to reduce the spread of floatables and reduce the level of effort for post-storm cleanup. These devices are simple to install, but can be difficult to maintain. Also, there are some negative aesthetic impacts associated with visibility of collected trash in a waterbody. This technology is strictly for floatables control and will not address water quality and CSO control goals alone.

As the Village already has netting facilities for solids and floatables control, containment booms would provide no additional benefit and will not be considered further in the LTCP.

4.9.1.5 Baffles

Baffles are simple floatables control devices that are typically installed at flow regulators within the CSS. They consist of vertical steel plates or concrete beams that extend from the top of the sewer to just below the top of the regulating weir. During an overflow event, floatables are retained by the baffles while water passes under the baffles, over the regulator, and into the receiving water body. When the flow recedes below the bottom of the baffle, floatable material is carried downstream to the treatment plant. Baffles are easy to install and require little maintenance, but do require proper hydraulic configuration. This technology is strictly for floatables control and will not address water quality and CSO control goals alone.

As the Village already has netting facilities for solids and floatables control, baffles would provide no additional benefit and will not be considered further in the LTCP.

4.9.2 Disinfection and Satellite Treatment

This technology consists of disinfecting and treating sewer overflows at a local facility near the CSO outfall. Disinfection is very effective at reducing bacteria through inactivation, but provides only limited opportunities for volume reduction. Disinfection alone cannot provide reductions in total suspended solids (TSS), floatables, and nutrient loads unless other processes, such as screening and high-rate clarification, are provided upstream of the disinfection facility. The combination of these other processes with disinfection can provide a satellite location that helps reduce pollutants of concern.

Disinfection of wet weather flow is more challenging to design and control than traditional disinfection at a treatment plant, because of the complex characteristics of the flow. Intermittent occurrences and highly variable flowrates and solids loadings, make it more challenging to regulate the addition of disinfectant. One way to address the variable flow issue is to provide flow retention facilities that provide for disinfectant contact time and capture through storage of the first flush of TSS, floatables and nutrients.

Wet weather flows can vary widely in temperature, suspended solids concentrations, and bacterial composition. Therefore, pilot studies are usually needed to characterize the range of conditions that exist for an area and the design criteria that need to be considered. Experience has shown that the long contact time required for conventional wastewater treatment is not appropriate for the treatment of wet weather flows. Disinfection can be achieved by providing an increased disinfection dosage and intense mixing to ensure disinfectant contact with the maximum number of microorganisms.

Various disinfection technologies are available, both with and without chlorine compounds. In addition to disinfection effectiveness, many factors should be considered when selecting a disinfectant, including potential toxic effects to the environment, regulations for residuals, safety precautions, and ease of

operation and maintenance. Ultraviolet (UV) light and peracetic acid (PAA) are two alternatives to chlorine compounds for wet weather disinfection.

- Ultraviolet Light - The main advantages of UV are its ability to quickly respond to flow variation and the absence of a disinfectant residual, among others. The size of the UV system mainly depends on the UV transmittance (i.e., the ability of wastewater to transmit UV light) and TSS concentrations in the wastewater. One of the challenges for UV disinfection is determining how to manage the disinfection of effluent during a power outage. In addition, UV typically has higher capital cost compared to chlorine disinfection systems.
- Peracetic Acid - The main advantage of PAA over sodium hypochlorite is its long ”shelf life” without product deterioration. Due to the intermittent nature of CSO flows, stored sodium hypochlorite may degrade over time if not used. However, PAA systems generally have higher operating costs than chlorine systems.

Disinfection is considered further in the alternatives evaluation as a technology to control bacteria and organic material in the CSO discharges. As disinfection does not provide solids removal, solids removal would need to be accomplished with a separate technology. While other disinfection technologies exist, peracetic acid is considered as an appropriate disinfection approach for the alternatives evaluation.

4.9.3 High Rate Primary Treatment

4.9.3.1 High Rate Physical/Chemical Treatment

High rate physical/chemical processes, such as Veolia’s Actiflo® or Infilco-Degremont’s DENSADEG®, are treatment technologies that require a much smaller footprint than conventional processes. These two competing products have similar applications, but the processes differ from each other considerably. For brevity, only one of these processes (Actiflo®) is described in detail below.

The Actiflo® process is similar to conventional coagulation, flocculation, and sedimentation water treatment technology. The process uses coagulant for suspended solid destabilization and flocculent aid (i.e., polymer) for the aggregation of suspended materials. The primary difference between Actiflo® and conventional processes is the addition of microsand for the formation of high-density flocs that have a higher-density nucleus and thus settle more rapidly.

Clarified water exits the process by flowing over a weir in the settling tank. The sand and sludge mixture that remains is collected at the bottom of the settling tank and pumped to a hydrocyclone which separates the sludge from the microsand. Sludge is discharged out of the top of the hydrocyclone, while the sand is recycled back into the Actiflo® process for further use. This process requires upstream screening to ensure that particles larger than 3 to 6 mm do not clog the hydrocyclone.

Actiflo® performance varies, but in general, removal rates of 80 - 95% for TSS and 30 - 60% for biological oxygen demand (BOD) are typical. Phosphorous and nitrogen are also removable with this process, although the removal efficiencies are dependent on the solubility of these compounds present in the wastewater. Phosphorous removal is typically between 60 - 90%, and nitrogen removal is typically between 15 - 35%. Removal efficiencies are also dependent on startup time. Typically, the Actiflo® process takes about 15 minutes before optimum removal rates are achieved.

The LTCP primary goals are bacteria reduction and CSO volume reduction. While high rate physical/chemical treatment reduces bacteria somewhat, its principal purpose is TSS reduction. Disinfection would be required downstream for bacteria inactivation. Additionally, while disinfection can be enhanced with upstream treatment, it may be adequately accomplished without high rate physical/chemical treatment. Although technologies such as Actiflo® or DENSADEG® reduce the footprint of conventional treatment, they still require a significant amount of available space for implementation. However, given the potential future water quality goals that may be imposed for TSS and BOD levels for

CSO discharges, the Actiflo® system will be considered as a representative technology for primary treatment when evaluating CSO discharge treatment as an alternative.

4.9.3.2 High Rate Physical Treatment

The Fuzzy Filter® by Schreiber or the WesTech WWETCO FlexFilter™ are innovative filtration technologies that use a compressible filter media, allowing a much smaller footprint than conventional filtration, with typical TSS reductions of nearly 90%. Both technologies use a synthetic fiber media that can handle increased flux rates of up to 30 – 40 gallons per minute per square foot (gpm/sf), as opposed to granular media such as sand. Additionally, the process uses compressed air scour with influent flow for filter backwashing, which eliminates the need for storage tanks. The filter removes up to 80% of influent particles up to 4 microns in diameter. These high rate physical treatment processes have relatively low operational and maintenance requirement, but periodic lubrication and detergent addition for media washing is necessary.

This technology is primarily designed for TSS reduction and would need to be coupled with downstream disinfection for bacteria inactivation. As such, this measure alone does not address the LTCP primary goals of bacteria reduction and overflow volume reduction. Like the high rate physical/chemical treatment process, high rate filtration systems can be challenging to implement for intermittent operation and highly variable influent conditions, but potential future requirements for TSS removal from CSO discharges indicates that primary treatment be considered for satellite CSO treatment facilities. The alternatives evaluation will look at high rate physical/chemical treatment as the representative primary treatment technology, but if advanced beyond the evaluation stage, high rate physical treatment could be reviewed during process design.

4.10 Summary

Table 4-1, Table 4-2 and Table 4-3 provide a summary of the control technologies considered in this section, with the results from the preliminary screening indicated.

Table 4-1: Screening summary - Source Control Technologies

Source Control Technologies							
Technology Group	Practice	Primary Goals		Implementation & Operation Factors	Consider Combining w/ Other Technologies	Being Implemented	Recommendation for Alternatives Evaluation
		Bacteria Reduction	Volume Reduction				
Stormwater Management	Street/Parking Lot Storage (Catch Basin Control)	Low	Low	Flow restrictions to the CSS can cause flooding in lots, yards and buildings; potential for freezing in lots; low operational cost. Effective at reducing peak flows during wet weather events but can cause dangerous conditions for the public if pedestrian areas freeze during flooding.	No	No	No
	Catch Basin Modification (for Floatables Control)	Low	None	Requires periodic catch basin cleaning; requires suitable catch basin configuration; potential for street flooding and increased maintenance efforts. Reduces debris and floatables that can cause operational problems with the mechanical regulators.	No	Yes	No
	Catch Basin Modification (Leaching)	Low	Low	Can be installed in new developments or used as replacements for existing catch basins. Require similar maintenance as traditional catch basins. Leaching catch basins have minor effects on the primary CSO control goals.	No	No	No
Public Education and Outreach	Water Conservation	None	Low	Water purveyor is responsible for the water system and all related programs in the respective City. However, water conservation is a common topic for public education programs. Water conservation can reduce CSO discharge volume, but would have little impact on peak flows.	Yes	Yes	No
	Catch Basin Stenciling	None	None	Inexpensive; easy to implement; public education. Is only as effective as the public's acceptance and understanding of the message. Public outreach programs would have a more effective result.	Yes	Yes	No
	Community Cleanup Programs	None	None	Inexpensive; sense of community ownership; educational BMP; aesthetic enhancement. Community cleanups are inexpensive and build ownership in the city.	Yes	Yes	No
	Public Outreach Programs	Low	None	Public education program is ongoing. Permittee should continue its public education program as control measures demonstrate implementation of the NMC.	Yes	Yes	No
	FOG Program	Low	None	Requires communication with business owners; Permitee may not have enforcement authority. Reduces buildup and maintains flow capacity. Only as effective as business owner cooperation.	Yes	Yes	No
	Garbage Disposal Restriction	Low	None	Permitee may not be responsible for Garbage Disposal. This requires an increased allocation of resources for enforcement while providing very little reduction to wet weather CSO events.	No	No	No
	Pet Waste Management	Medium	None	Low cost of implementation and little to no maintenance. This is a low cost technology that can significantly reduce bacteria loading in wet weather CSO's.	Yes	Yes	No
	Lawn and Garden Maintenance	Low	Low	Requires communication with business and homeowners. Guidelines are already established per USEPA. Educating the public on proper lawn and garden treatment protocols developed by USEPA will reduce waterway contamination. Since this information is already available to the public it is unlikely to have a significant effect on improving water quality.	Yes	No	No
	Hazardous Waste Collection	Low	None	The N.J.A.C prohibits the discharge of hazardous waste to the collection system.	Yes	Yes	No

Source Control Technologies

Technology Group	Practice	Primary Goals		Implementation & Operation Factors	Consider Combining w/ Other Technologies	Being Implemented	Recommendation for Alternatives Evaluation
		Bacteria Reduction	Volume Reduction				
Ordinance Enforcement	Construction Site Erosion & Sediment Control	None	None	In building code; reduces sediment and silt loads to waterways; reduces clogging of catch basins; little O&M required; contractor or owner pays for erosion control. A Soil Erosion & Sediment Control Plan Application or 14-day notification (if Permitee covered under permit-by-rule) will be required by NJDEP per the N.J.A.C.	Yes	Yes	No
	Illegal Dumping Control	Low	None	Enforcement of current law requires large number of code enforcement personnel; recycling sites maintained. Local ordinances already in place can be used as needed to address illegal dumping complaints.	Yes	Yes	No
	Pet Waste Control	Medium	None	Requires resources to enforce pet waste ordinances. Public education and outreach is a more efficient use of resources, but this may also provide an alternative to reducing bacterial loads.	Yes	Yes	No
	Litter Control	None	None	Aesthetic enhancement; labor intensive; City function. Litter control provides an aesthetic and water quality enhancement. It will require city resources to enforce. Public education and outreach is a more efficient use of resources.	Yes	Yes	No
	Illicit Connection Control	Low	Low	Site specific; more applicable to separate sanitary system; new storm sewers may be required; interaction with homeowners required. The primary goal of the LTCP is to meet the NJPDES Permit requirements relative to POCs. Illicit connection control is not particularly effective at any of these goals and is not recommended for further evaluation unless separate sewers are in place.	Yes	Yes	No
Good Housekeeping	Street Sweeping/Flushing	Low	None	Labor intensive; specialized equipment; doesn't address flow or bacteria; City function. Street sweeping and flushing primarily addresses floatables entering the CSS while offering an aesthetic improvement.	Yes	Yes	No
	Leaf Collection	Low	None	Requires additional seasonal labor. Leaf collection maximizes flow capacity and removes nutrients from the collection system.	Yes	Yes	No
	Recycling Programs	None	None	Most Cities have an ongoing recycling program.	Yes	Yes	No
	Storage/Loading/Unloading Areas	None	None	Requires industrial & commercial facilities designate and use specific areas for loading/unloading operations. There may be few major commercial or industrial users upstream of CSO regulators.	Yes	No	No
	Industrial Spill Control	Low	None	PVSC has established a pretreatment program for industrial users subject to the Federal Categorical Pretreatment Standards 40 CFR 403.1.	Yes	Yes	No

Source Control Technologies							
Technology Group	Practice	Primary Goals		Implementation & Operation Factors	Consider Combining w/ Other Technologies	Being Implemented	Recommendation for Alternatives Evaluation
		Bacteria Reduction	Volume Reduction				
Green Infrastructure Buildings	Green Roofs	None	Medium	Adds modest cost to new construction; not applicable to all retrofits; low operational resource demand; will require the Permitee or private owners to implement; requires regular cleaning of gutters & pipes; upkeep of roof vegetation. Portions of Cities have densely populated areas, but this technology is limited to rooftops. Can be difficult to require on private properties.	Yes	No	No
	Blue Roofs	None	Medium	Adds modest cost to new construction; not applicable to all retrofits; low operational resource demand; will require the Permitees or private owners to implement; requires regular cleaning of gutters & pipes; upkeep of roof debris. Portions of the Cities have densely populated areas, but this technology is limited to rooftops. Can be difficult to require on private properties.	Yes	No	No
	Rainwater Harvesting	None	Medium	Simple to install and operate; low operational resource demand; will require the Permitees or private owners to implement; requires regular cleaning of gutters & pipes. Portions of the Cities have densely populated areas, but this technology is limited to capturing rooftop drainage. Capture is limited to available storage, which can vary on rainwater use. Can be difficult to require on private properties.	Yes	No	No
Green Infrastructure Impervious Areas	Permeable Pavements	Low	Medium	Not durable and clogs in winter; oil and grease will clog; significant O&M requirements with vacuuming and replacing deteriorated surfaces; can be very effective in parking lots, lanes and sidewalks. Maintenance requirements could be reduced if located in low-traffic areas, and can utilize underground infiltration beds or detention tanks to increase storage.	Yes	No	Yes
	Planter Boxes	Low	Medium	Site specific; good BMP; minimal vegetation & mulch O&M requirements with regular overflow and underdrain cleaning; effective at containing, infiltrating and evapotranspirating runoff in developed areas. Flexible and can be implemented even on a small-scale to any high-priority drainage areas. Underground infiltration beds or detention tanks can be utilized to increase storage.	Yes	No	No
Green Infrastructure Pervious Areas	Bioswales	Low	Low	Site specific; good BMP; minimal vegetation & mulch O&M requirements; not as flexible or infiltrate as much stormwater as planter boxes. Technology requires open space and is primarily a surface conveyance technology with additional storage & infiltration benefits. Can be modified with check dams to slow water flow. Limited open space in most Cities means land can be utilized in more effective ways with the existing infrastructure.	Yes	No	Yes
	Free-Form Rain Gardens	Low	Medium	Site specific; good BMP; minimal vegetation & mulch O&M requirements with regular overflow and underdrain cleaning; effective at containing, infiltrating and evapotranspirating diverted runoff. Rain Gardens are flexible and can be modified to fit into the previous areas. Underground infiltration beds or detention tanks can be utilized to increase storage.	Yes	No	No

Table 4-2: Screening summary - Collection System Technologies

Collection System Technologies							
Technology Group	Practice	Primary Goals		Implementation & Operation Factors	Consider Combining w/ Other Technologies	Being Implemented	Recommendation for Alternatives Evaluation
		Bacteria Reduction	Volume Reduction				
Operation and Maintenance	I/I Reduction	Low	Medium	Requires labor intensive work; changes to the conveyance system require temporary pumping measures; repairs on private property required by homeowners. Reduces the volume of flow and frequency; Provides additional capacity for future growth; House laterals account for 1/2 the sewer system length and significant sources of I/I in the sanitary sewer.	Yes	No	No
	Advanced System Inspection & Maintenance	Low	Low	Requires additional resources towards regular inspection and maintenance work. Inspection and maintenance programs can provide detailed information about the condition and future performance of infrastructure. Offers relatively small advances towards goals of the LTCP.	Yes	No	No
	Combined Sewer Flushing	Low	Low	Requires inspection after every flush; no changes to the existing conveyance system needed; requires flushing water source. Ongoing: CSO Operational Plan; maximizes existing collection system; reduces first flush effect.	Yes	No	No
	Catch Basin Cleaning	Low	None	Labor intensive; requires specialized equipment. Catch Basin Cleaning reduces litter and floatables but will have no effect on flow and little effect on bacteria and BOD levels.	Yes	Yes	No
Combined Sewer Separation	Roof Leader Disconnection	Low	Low	Site specific; Includes area drains and roof leaders; new storm sewers may be required; requires home and business owner participation. The Cities are densely populated and disconnected roof leaders have limited options for discharge to pervious space. Disconnection may be coupled with other GI technologies but is not considered an effective standalone option.	Yes	No	No
	Sump Pump Disconnection	Low	Low	Site specific; more applicable to separate sanitary system; new storm sewers may be required; interaction with homeowners required. The Cities are densely populated and disconnected sump pumps have limited options for discharge to pervious space. Disconnection may be coupled with other GI technologies but is not considered an effective standalone option.	Yes	Yes	No
	Combined Sewer Separation	High	High	Very disruptive to affected areas; requires homeowner participation; sewer asset renewal achieved at the same time; labor intensive.	No	Yes	Yes
Combined Sewer Optimization	Additional Conveyance	High	High	Additional conveyance can be costly and would require additional maintenance to keep new structures and pipelines operating.	No	No	No
	Regulator Modifications	Medium	Medium	Relatively easy to implement with existing regulators; mechanical controls requires O&M. May increase risk of upstream flooding. Permitees have an ongoing O&M program and system wide replacement program for CSO regulators and tide gates.	Yes	No	No
	Outfall Consolidation/Relocation	High	High	Lower operational requirements; may reduce permitting/monitoring; can be used in conjunction with storage & treatment technologies. Combining and relocating outfalls may lower operating costs and CSO flows. It can also direct flow away from specific areas.	Yes	No	Yes
	Real Time Control	High	High	Requires periodic inspection of flow elements; highly automated system; increased potential for sewer backups. RTC is only effective if additional storage capacity is present in the system.	Yes	No	No

Table 4-3: Screening summary - Storage and Treatment Technologies

Storage and Treatment Technologies							
Technology Group	Practice	Primary Goals		Implementation & Operation Factors	Consider Combining w/ Other Technologies	Being Implemented	Recommendation for Alternatives Evaluation
		Bacteria Reduction	Volume Reduction				
Linear Storage	Pipeline	High	High	Can only be implemented if in-line storage potential exists in the system; increased potential for basement flooding if not properly designed; maximizes use of existing facilities. Pipe storage for a CSS typically requires large diameter pipes to have a significant effect on reducing CSOs. This typically requires large open trenches and temporary closure of streets to install.	No	Yes	No
	Tunnel	High	High	Requires small area at ground level relative to storage basins; disruptive at shaft locations; increased O&M burden.	No	No	Yes
Point Storage	Tank (Above or Below Ground)	High	High	Storage tanks typically require pumps to return wet weather flow to the system which will require additional O&M; disruptive to affected areas during construction. Several CSO outfalls have space available for tank storage. There may be existing tanks in abandoned commercial and industrial areas to be converted to hold stormwater. Tanks are an effective technology to reduce wet weather CSO's.	No	No	Yes
	Industrial Discharge Detention	Low	Low	Requires cooperation with industrial users; more resources devoted to enforcement; depends on IUs to maintain storage basins. IUs hold stormwater or combined sewage until wet weather flows subside; there may be commercial or industrial users upstream of CSO regulators.	Yes	No	No
Treatment-CSO Facility	Vortex Separators	None	None	Space required; challenging controls for intermittent and highly variable wet weather flows. Vortex separators would remove floatables and suspended solids when installed. It does not address volume, bacteria or BOD.	Yes	No	No
	Screens and Trash Racks	None	None	Prone to clogging; requires manual maintenance; requires suitable physical configuration; increased O&M burden. Screens and trash racks will only address floatables.	Yes	No	No
	Netting	None	None	Easy to implement; labor intensive; potential negative aesthetic impact; requires additional resources for inspection and maintenance. Netting will only address floatables.	Yes	Yes	No
	Contaminant Booms	None	None	Difficult to maintain requiring additional resources. Contaminant booms will only address floatables.	Yes	No	No
	Baffles	None	None	Very low maintenance; easy to install; requires proper hydraulic configuration; long lifespan. Baffles will only address floatables.	Yes	No	No
	Disinfection & Satellite Treatment	High	None	Requires additional flow stabilizing measures; requires additional resources for maintenance; requires additional system analysis. Disinfection is an effective control to reduce bacteria and BOD in CSO's.	Yes	No	Yes
	High Rate Physical/Chemical Treatment (High Rate Clarification Process - ActiFlo)	None	None	Challenging controls for intermittent and highly variable wet weather flows; smaller footprint than conventional methods. This technology primarily focuses on TSS & BOD removal, but does not help reduce the bacteria or CSO discharge volume.	Yes	No	Yes
	High Rate Physical (Fuzzy Filters)	None	None	Relatively low O&M requirements; smaller footprint than traditional filtration methods. This technology primarily focuses on TSS removal, but does not help reduce the bacteria or CSO discharge volume.	Yes	No	No
Treatment-WRTP	Additional Treatment Capacity	High	High	May require additional space; increased O&M burden.	No	No	No
	Wet Weather Blending	Low	High	Requires upgrading the capacity of influent pumping, primary treatment and disinfection processes; increased O&M burden. Wet weather blending does not address bacteria reduction, as it is a secondary treatment bypass for the POTW. Permittee must demonstrate there are no feasible alternatives to the diversion for this to be implemented.	Yes	No	No
Treatment-Industrial	Industrial Pretreatment Program	Low	Low	Requires cooperation with Industrial User's; more resources devoted to enforcement; depends on IU's to maintain treatment standards. May require Permits.	Yes	No	No

5 Costing

5.1 Background

The LTCP development process requires that the permittees each evaluate a variety of CSO control alternatives and submit an Evaluation of Alternatives Report. Part of this analysis is the evaluation of costs for each alternative, at different CSO control levels. This chapter outlines the basis and assumptions upon which the cost estimates have been developed.

5.2 Cost Estimating Approach

Cost estimates for the CSO control alternatives have been developed as part of the LTCP process. The costs provided are meant to provide an order of magnitude estimate referred to as Class 5 estimates, with an accuracy of -50% to +100%, and generally include a 50% contingency to reflect the planning level. The estimates have been developed specifically for the configurations of the alternatives that have been described. It is noted that any modifications to these alternatives or their configurations may impact the cost. The information and costs presented in this report is for planning purposes only, and all assumptions and information must be verified in subsequent planning and design stages.

The costs are presented as follows:

- Capital cost – including equipment cost, installation, training, labor, electrical and water connections, structural platforms, land acquisition, design, administrative costs, construction management, etc.
    - Design costs were assumed to be 10% of the construction cost.
    - Construction Management Costs were assumed to be 10% of the construction costs.
    - Administrative/Legal costs were assumed to be 5% of the construction cost.
- Operations & Maintenance (O&M) – annual power, chemical dosing, labor, etc. Since a 20-year planning period has been selected, it does not include any larger-scale overhauls or replacements/repairs that would be completed of the life of the facility.
- Present worth – for a period of twenty years with interest rate of 2.75%, as described below.

Most costs are presented in terms of the level of CSO controls, however alternatives such as sewer separation are presented as a lump sum cost for reducing CSO events to zero per year.

In addition to itemized cost items, the costs are presented as dollars per gallon ($/gal) of CSO removed from the receiving water during the Typical Year, to provide a point for comparison between alternatives.

5.3 Present Worth Calculations

To be consistent with other permittees, PVSC guidance from the January 8th memorandum and meeting on March 21, 2019 was used to develop present worth costs for all the alternatives, to combine O&M and full capital costs for each control technology. A discount rate of 2.75% was used (Rate of Federal Water Projects, NRCS Economics, Department of the Interior) with a life span of 20 years. The following equation was then utilized to calculate the present worth factor to convert from annual O&M costs to present worth.

$$(P/A, i\%, n) = ((1+i)^n-1)/((i(1+i)^n)$$

The above was multiplied by the annual O&M costs and added to the construction costs to obtain the total life cycle cost. For the given life cycle and interest rate the P/A factor is 15.227. Salvage value was assumed to be $0, as it is assumed no resale value will result from the Control Technologies utilized.

5.4 Updated Technical Guidance Manual (January 2018)

In 2004, the NJDEP issued a General Permit (GP) for combined sewer systems that, in part, required combined sewer system owners to initiate the CSO LTCP development process and undergo a Cost and Performance Analysis for Combined Sewer Overflow Point Operation. That analysis required the permittees to evaluate alternatives at each CSO point that would provide continuous disinfection prior to discharge. To assist their communities in performing the analysis, PVSC developed a Technical Guidance Manual (TGM) that provides an overview of various screening, pretreatment, disinfection, and storage technologies along with guidance on construction and operation and maintenance costs. The original TGM was released in 2007.

The New Jersey Pollutant Discharge Elimination System (NJPDES) permits issued in 2015 requires the permittees to continue the CSO LTCP development process and perform a complete CSO control alternatives evaluation that will lead to a selected alternative and eventual implementation. While much of the information in the original TGM is still viable, a decade has passed since it was developed. To assist their permittees with the current permit, PVSC has updated the TGM to reflect new information, updated costs, and new permit requirements such as the evaluation of green infrastructure. PVSC has agreed and offered the TGM for use by members of the NJ CSO Group.

The updated TGM will be used by members of the BCUA CSO Group in developing contruction cost and operation and maintenance costs for alternative controls under consideration. The Village of Ridgefield Park is undertaking a Regional approach to the LTCP with members of the BCUA CSO Group. As previously noted each member of the Group are developing individual Development and Evaluation of Alternative Controls Reports on their individual systems, which will be incorporated into the BCUA Report as Addendums. To avoid redundancy cost information from the TGM will only be incorporated into the BCUA Region Report.

The TGM provides a methodology for developing planning level construction costs for various control alternatives, as well as a process for including contingencies, non-direct costs, overhead and profit. The TGM was used to develop capital costs for end-of-pipe storage, end-of-pipe treatment, tunnels and green infrastructure.

- End-of-pipe (off-line) storage – estimated based on cost curves provided in the PVSC TGM. The cost curves were replicated and extrapolated to represent costs for the estimated flows. Costs were developed to consolidate Outfalls 001A and 002A and 003A-006A into regional facilities. Costs were developed for the different control levels.
- Tunneling - estimated based on cost curves provided in the PVSC TGM. The cost curves were replicated and extrapolated to represent costs for the estimated tunnel diameter and length to consolidate all six outfalls.
- End-of-pipe treatment - estimated based on cost curves provided in the PVSC TGM. The cost curves were replicated and extrapolated to represent costs for the estimated flows. Costs were developed to consolidate Outfalls 001A and 002A and 003A-006A into regional facilities. Costs were developed for the different control levels.
- Green infrastructure – costs were developed for each of the control levels directing 2.5%, 5%, 7.5% and 10% of the directly connected impervious area within the combined sewer area to green stormwater infrastructure. It was assumed that property acquisition would not be required because all work would be completed in the public right-of-way. The capital cost was based on a unit cost for bioswales and permeable pavement which were provided in the TGM.

The TGM did not include cost information on sewer separation such alternative resources were consulted to develop the cost estimate. Specific considerations and supplements from reference documents were used to fill in any gaps or assumptions from the TGM. These additional tools are described below.

5.5 PVSC District CSO Permittee Meeting (March 21, 2019)

At a PVSC meeting on March 21, 2019, Greeley and Hansen presented typical operational and maintenance (O&M) costs for green infrastructure, storage tanks, and tunnels. These are summarized in the table below

Table 5-1: Typical Operation and Maintenance Costs

	Item	Unit	Cost Basis (per year)
Operation	Pump Station* Up to 100 MGD Over 100 MGD	COP COP	0.5 x $470K 2.0 x $470K
	Storage	% of construction cost	0.5% (Revised)
	Tunnels	COP	1.0 x $470K
Maintenance	Green Infrastructure	Per impervious acre managed	$8,000
	Pump Station	% of construction cost	2.0%
	Storage	% of construction cost	0.5% (Revised)
	Tunnels	% of construction cost	2.0%
	Conveyance Pipelines/ Sewer Separation	% of construction cost	2.0%

*Pump station operation for tunnels included in tunnel operation. Only add pump station operation costs if standalone pump station.

The above estimate assumes labor cost as one operator on a 24-hour year-round basis at a cost of $53.60/hr. Assuming an eight-hour workday with three shifts per day year-round, the average cost of a Continuous Operating Post (COP) would be $470,000. Upon review the Maintenance cost of 3% for storage tanks was too high and this was revised to 1% for total O&M costs.

At this meeting, guidance for calculation of present worth cost as described above was also presented.

5.6 Technology-Specific Sources

For some of the CSO control alternatives, additional sources were consulted to develop a more detailed and comprehensive cost understanding. These are described as follows:

- *Sewer separation – The approach for estimating the total projected conceptual cost of sewer separation was to derive unit costs (cost per linear foot of sewer) for a number of drainage areas, and then to apply these unit costs to the entire combined sewer system. First, drainage areas to each regulator were delineated and enumerated. Then, a proposed sanitary sewer system layout was developed for two representative regulator drainage areas. Sanitary sewers were proposed in all areas served by combined sewers. Manning's number, slope, and pipe size were considered in siting the new separate sanitary sewers. Treatment of stormwater runoff was included in the cost estimate by selecting the San-Sep solids removal technology from the TGM. It is noted that proposed rule amendments to the Stormwater Management Rules were issued by NJDEP. These amendments would require the use of green infrastructure for sewer separation projects to address the requirements of the rules for the areas within the public right-of-way. The final form of the rule amendments has not been issued as such their impact cannot be reasonable be anticipated and thus has not been incorporated into the cost estimate. The resulting sewer separation costs were found to be $295,000 per acre plus stormwater treatment costs.*

5.7 Land Acquisition Costs

There is a great deal of uncertainty when estimating land acquisition costs, as the dramatic rise in prices leading up to 2008 and the subsequent drop in real estate values demonstrated. For planning purposes, it was assumed that property could be acquired for $1.5 million per acre. This approach provides a consistent basis of cost. The actual acquisition cost will depend on the owner's willingness to sell, with additional legal costs incurred if it is necessary to acquire the property through condemnation. Site history of contamination and future plans for development will also factor in the final price of acquisition. It was also assumed that rather than acquiring the estimated footprint of the CSO control facilities, acquisition of the entire area identified in the siting analysis would be required.

5.8 Cost Index

The costs were indexed to the Engineering News Record (ENR) Construction Cost Index (CCI) for January 2019 with a CCI of 11,206.

6 Available Land Analysis

6.1 Background

As part of the development and evaluation of alternative controls, the Village undertook an evaluation of potential parcels/sites where storage or end-of-pipe treatment might be installed to reduce the frequency and volume of CSOs. This section summarizes the steps that were taken to evaluate land availability across the Village and to provide information on the process undertaken to develop a list of possible siting locations.

While open or underutilized land is preferred for siting of alternative controls, the lack of open or underutilized land will not eliminate the need for implementation of CSO controls. Nevertheless, this is the first step in the process of site selection.

6.2 Data Compilation and Processing

The Village of Ridgefield Park was first analyzed using the following publicly available GIS information:

- Aerial photography
- Land Use / Land Cover
- Parcel data, including vacant land, land ownership, and property value information
- Open Space / Green Acres
- Soil Type
- Topography
- Known Contaminated Sites
- Brownfields

This information was layered into GIS and analyzed to identify potential sites for storage or end-of-pipe treatment. Residential areas, transportation corridors and water bodies were initially subtracted out of the analysis, as it was assumed these areas would not be suitable candidates. The remaining areas were analyzed based on aerial photography, with site preferences determined based on proximity to the CSO regulator and outfall, public ownership and vacant land, as well as potential underutilized sites such as parking areas or possible abandoned sites. Other locations considered included primarily underutilized commercial, industrial, or open space areas, including known brownfields, which are near a combined sewer outfall.

A resident at a SCSO Team meeting suggested using the land beneath the Route I-80 bridge over the Hackensack River. Observations of the site indicated a number of columns supporting the bridge and the location is remote from most of the CSO outfalls. Therefore, it was not included in the siting analysis, but the suggestion is retained should other site prove untennable.

6.3 Evaluation

The overall land use of Ridgefield Park is illustrated in Figure 6-1 with residential and water bodies subtracted out. The remaining shaded areas were evaluated as potential sites by visual inspection of aerial photography. Areas that are have been approved for development and/or redevelopment have been eliminated. In addition, while some locations are currently used for other purposes (i.e. high school) these areas were viewed to see if the alternative control features could be integrated into the current use of the property.

In general storage and end-of-pipe treatment alternatives will have to be located near the CSO Regulator and Outfalls to maximize the volume and flows being treated, however storage facilities could also be in upstream areas where wet weather flows would be diverted into storage facilities to reduce the volume

and peak flows generated at the outfall. Accordingly, for storage facilities both upstream and downstream locations were considered.

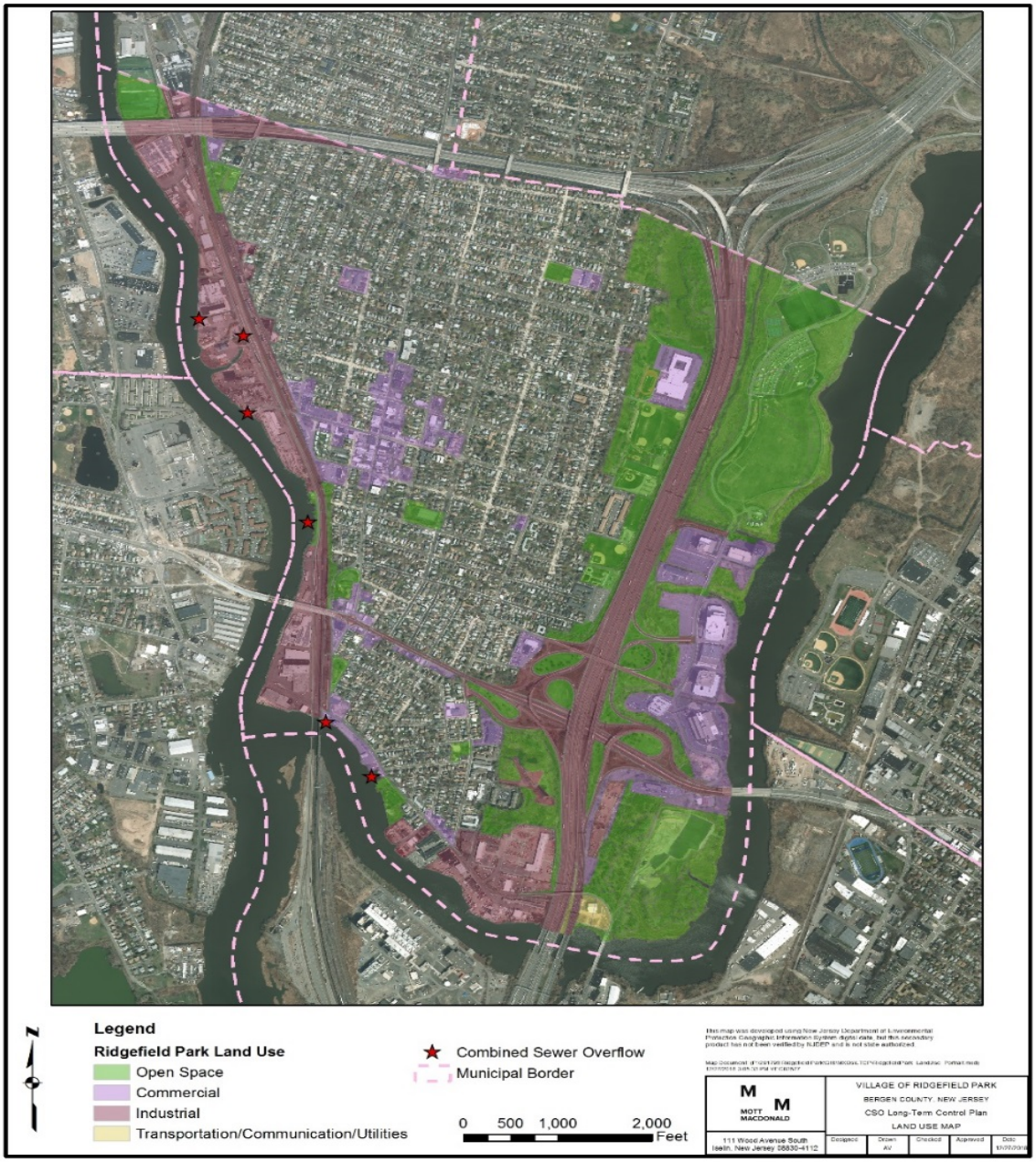


Figure 6-1: Village of Ridgefield Park Land Use Map Excluding Residential Areas

Another site location factor that needs to be considered in site selection is the elevation of the site. The 2-foot contours for the Village of Ridgefield Park are illustrated in Figure 6-2. In general, for end of pipe treatment facilities any site greater elevation greater than 28 feet wasn't considered viable due to the possible need for deep excavations or pumping that would be required to install a pipeline connecting a control facility to a sewer near the outfall. Utilizing the above general information on open/underutilized

properties and ground elevations a more detailed review/analysis was conducted in the proximity of each of the CSO outfalls as detailed below.

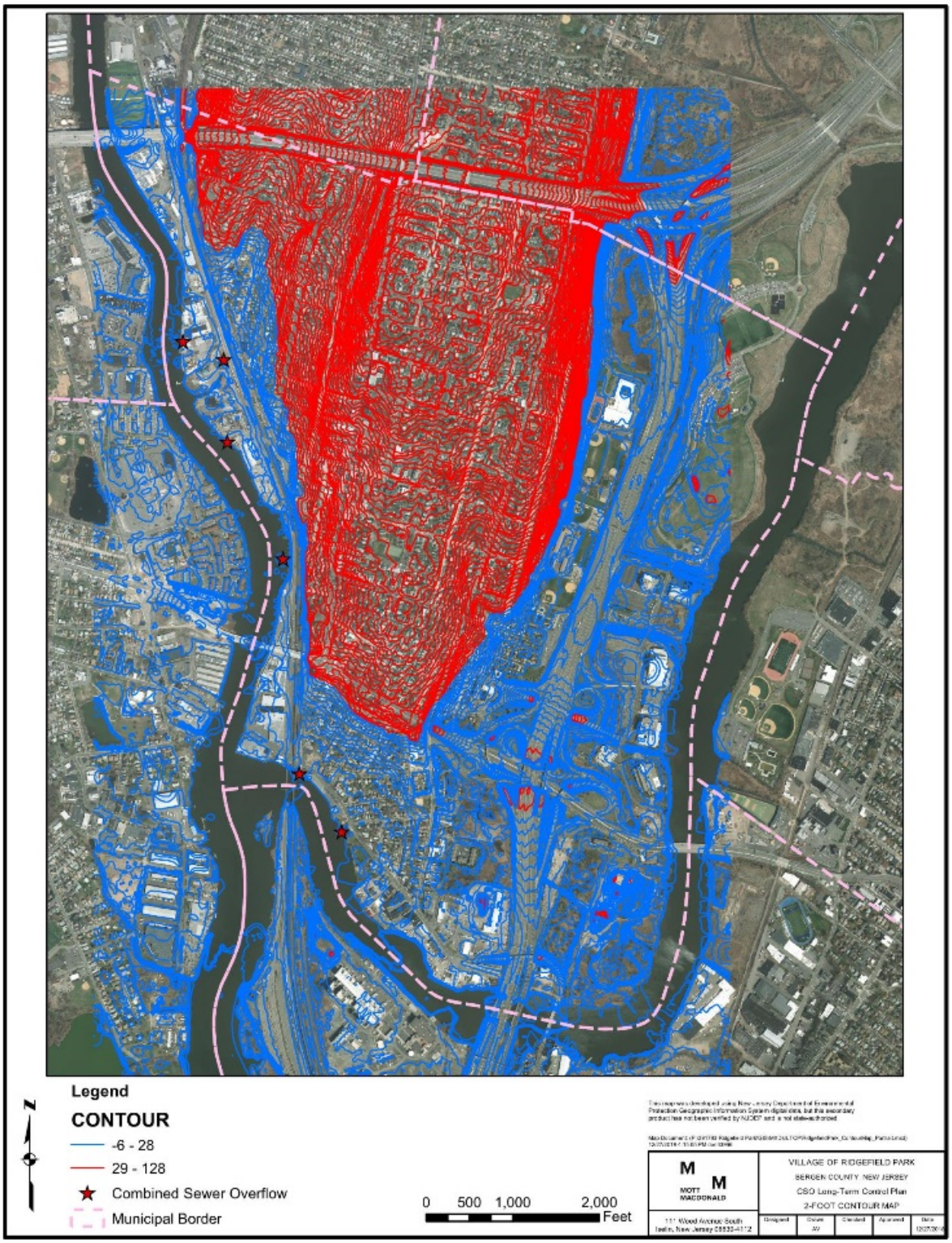


Figure 6-2: Village of Ridgefield Park Surface Elevations >28'

6.3.1 Outfall 001A

McGowan Park, see Figure 6-3, located southeast of Outfall 001A between Bergen Turnpike and the Overpeck Creek, was investigated as a possible CSO control site for Outfall 001A. It is located about 90 feet from the outfall and is currently a publicly-owned open park space. The site area as shown in Figure 6-3 is 1.6 acres. Because of the relatively new community garden—shown in the latest Google Maps aerial and street views, but not shown in this aerial—that has been established at the northern end of the park, the available space for underground storage may be limited to 1.35 acres. This site is listed on the NJDEP Recreation and Open Space Inventory (ROSI) database as a Green Acres property. The utilization of Green Acres for CSO control is still being addressed, and for this analysis it is assumed that only green infrastructure alternatives will be allowed on a Green Acres property.

Figure 6-3: McGowen Park

The VFW property located north of the outfall is currently owned by the Village and could be used for the location of a control facility, however, from a public acceptance standpoint, it would be preferable to preserve the VFW building. Both Outfalls 001A and 002A are located along Bergen Turnpike and are in relatively proximity to one another. It is anticipated based on offline storage cost estimates that were developed and reported on in 2007 that the most cost-effective approach for both outfalls would be to have one storage facility that would service both drainage areas. Outfall 001A is one of the larger outfalls in the Village. Accordingly, some or all of the flow could be handled by the proposed CSO facility that would go on the commercial open space shown in Figure 6-4.

6.3.2 Outfall 002A

There is an underutilized commercial space (former restaurant) adjacent to Outfall 002A with an area of approximately 1.5 acres, see Figure 6-4. According to the latest aerial photographs and street views from Google Maps, this site does not have any structures, but there is a parking lot on the east side. It became known through one of the CSO Supplemental Team meetings that this property had been before the planning board for review and approval of a high-rise apartment. Mott MacDonald provided the Village with a recommendation that they purchase the property before it is developed. The Village is in the process of purchasing this property at the present time to protect its ability to comply with requirements of their NJPDES permit.

The east side of this property it is only 290 feet from Outfall 001A, and the VFW Property already owned by the Village could be used in conjunction with this site, so it could be used to control overflows from both Outfalls 001A and 002A. Given the current usage of the site, its proximity to CSO outfalls, and its size, it would be strong candidate for CSO facilities. Although the area across the street appears to be a vacant lot and a good site for CSO facilities, more recent aerials and street views from Google Maps show a new building that had recently been constructed there.

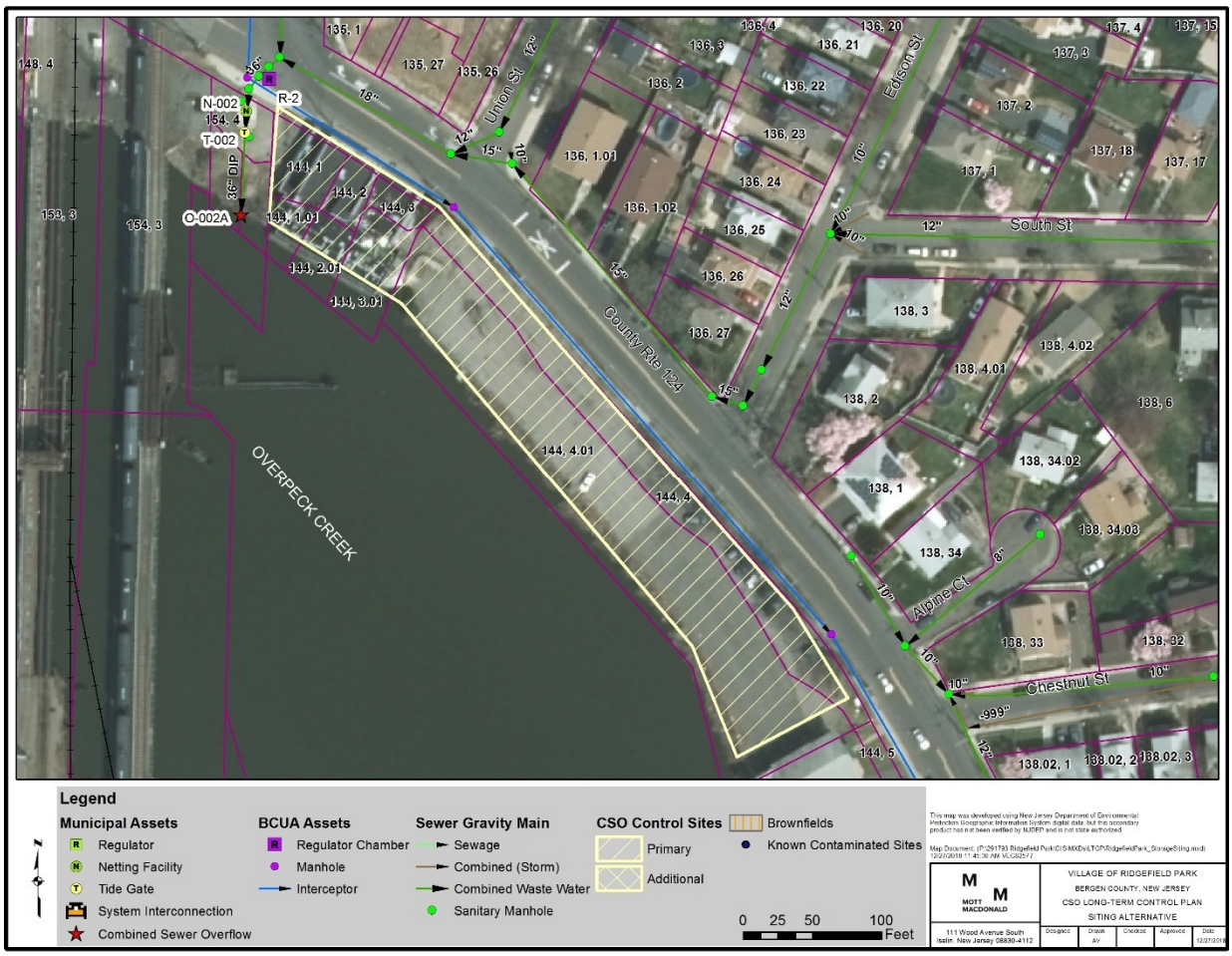


Figure 6-4: Under-Utilized Commercial Property

Ferris Park, a publicly-owned open park space, see Figure 6-5, is approximately 490 feet north of Outfall 002A. It has an area of about 0.5 acre. However, given that there are numerous trees on this property and it is listed on the NJDEP Recreation and Open Space Inventory (ROSI) database as a Green Acres property, it would likely only be suitable for limited underground facilities or aboveground green infrastructure. In addition, this site has an elevation above 15, which would make it unacceptable for a storage facility for Outfall 002A.

Figure 6-5: Ferris Park

6.3.3 Outfall 003A

There is an area of underutilized space with parking for the Ridgefield Park Elks (Lodge 1506) adjacent to the strip of vegetated open space owned by Conrail, see Figure 6-6. Regulator 3 is located on a plateau and the outfall pipe drops significant to the outfall. This site is about 510 feet north of Outfall 003A and has an area of 0.6 acre. Given that Conrail owns the land between this site and the outfall, running pipe to connect a CSO facility at this site to the sewers at the outfall may require a permit and an easement from Conrail. Nevertheless, the property as illustrated in Figure 6-6 is a possible location for off-line storage for this outfall, but would require a significant diversion of the outfall pipe in an area with significant ground slope.

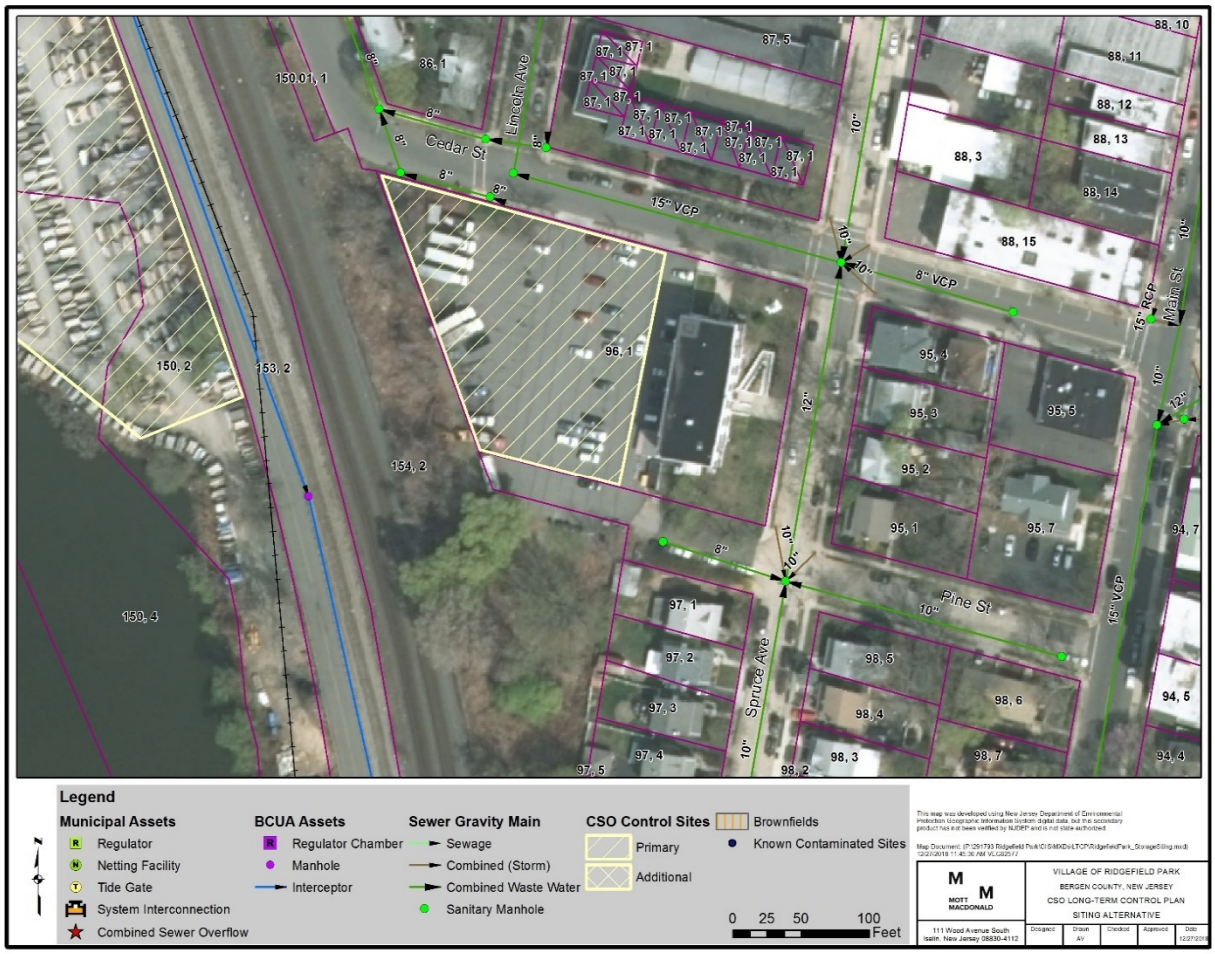


Figure 6-6: Commercial Parking Lot

Brewster Park, which is located about 720 feet southeast of Outfall 003A, was investigated as a possible CSO control site for Outfall 003A, see Figure 6-7. The property is currently publicly-owned open park space as well as a commercially zoned area for the Ridgefield Park Fire Department and Volunteer Ambulance Corps. The site area to be considered for CSO facilities is about 1.5 acres, which includes the open park space and the parking lot. However, given that this site is listed on the NJDEP Recreation and Open Space Inventory (ROSI) database as a Green Acres property, impacts to the site could be minimized by only constructing underground facilities at the oval-shaped asphalt area and the parking lot,

which constitute an area of about 0.5 acre. Another constraint may be the utility poles on site: there is one between the playground area and the parking lot and a few around the oval-shaped asphalt area.

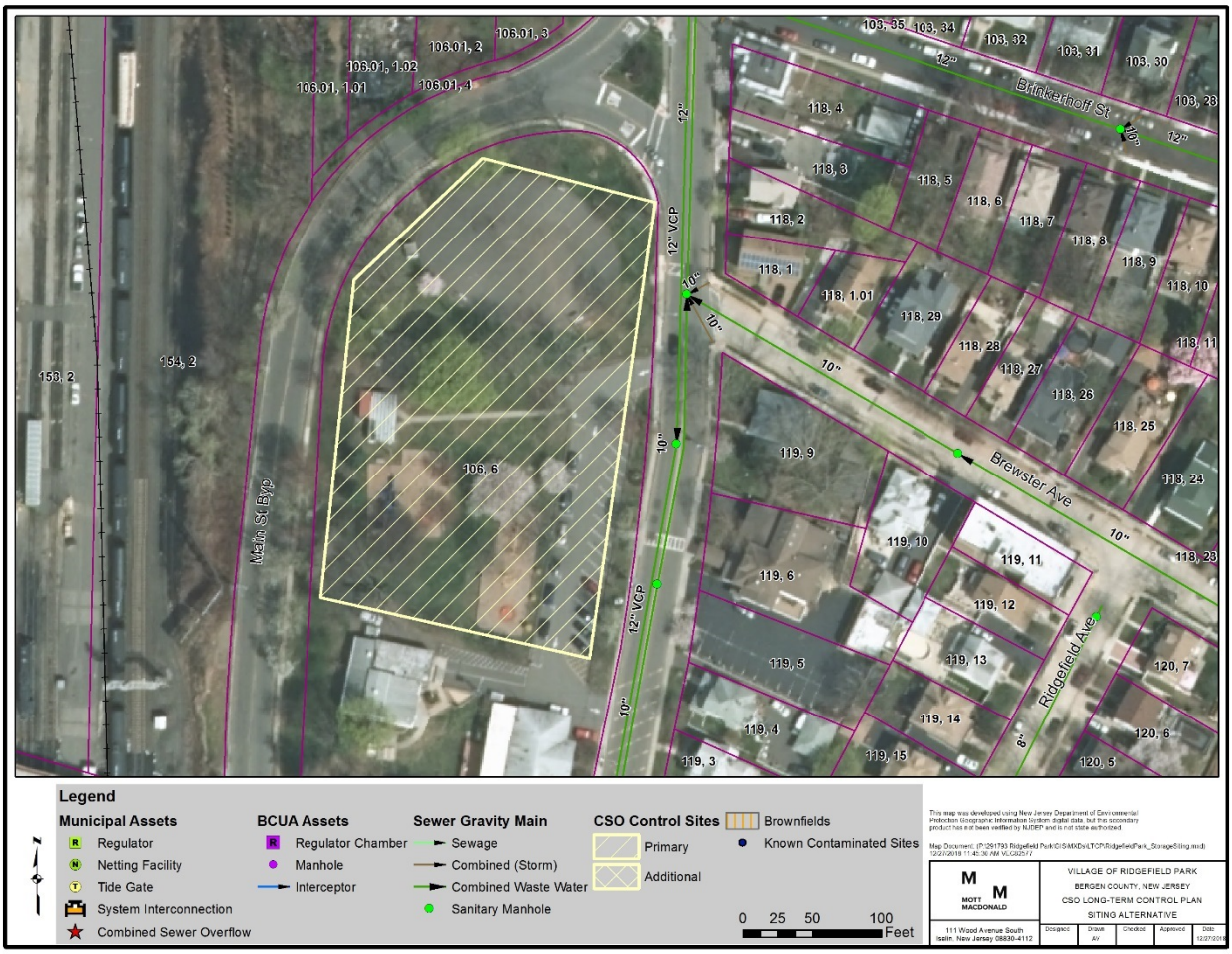


Figure 6-7: Open Space and Parking

6.3.4 Outfall 004A

Outfall 004A is the largest of the Village outfalls by volume and peak flow. There is an automobile recycling facility by Apache Auto Wreckers Inc. just north of Outfall 004A with a potential underground storage area of 1.2 acres. Given the current usage of the site, its proximity to CSO outfalls, and its size, it would be strong candidate for CSO facilities, see Figure 6-8.

On the adjacent property north of the automobile recycling, there is an area of underutilized space with truck parking for Hageman Roofing that is 0.6 acre. This property is a “Known Contaminated Site” associated with M.J. Paquet Inc. with a remedial level of C-2.

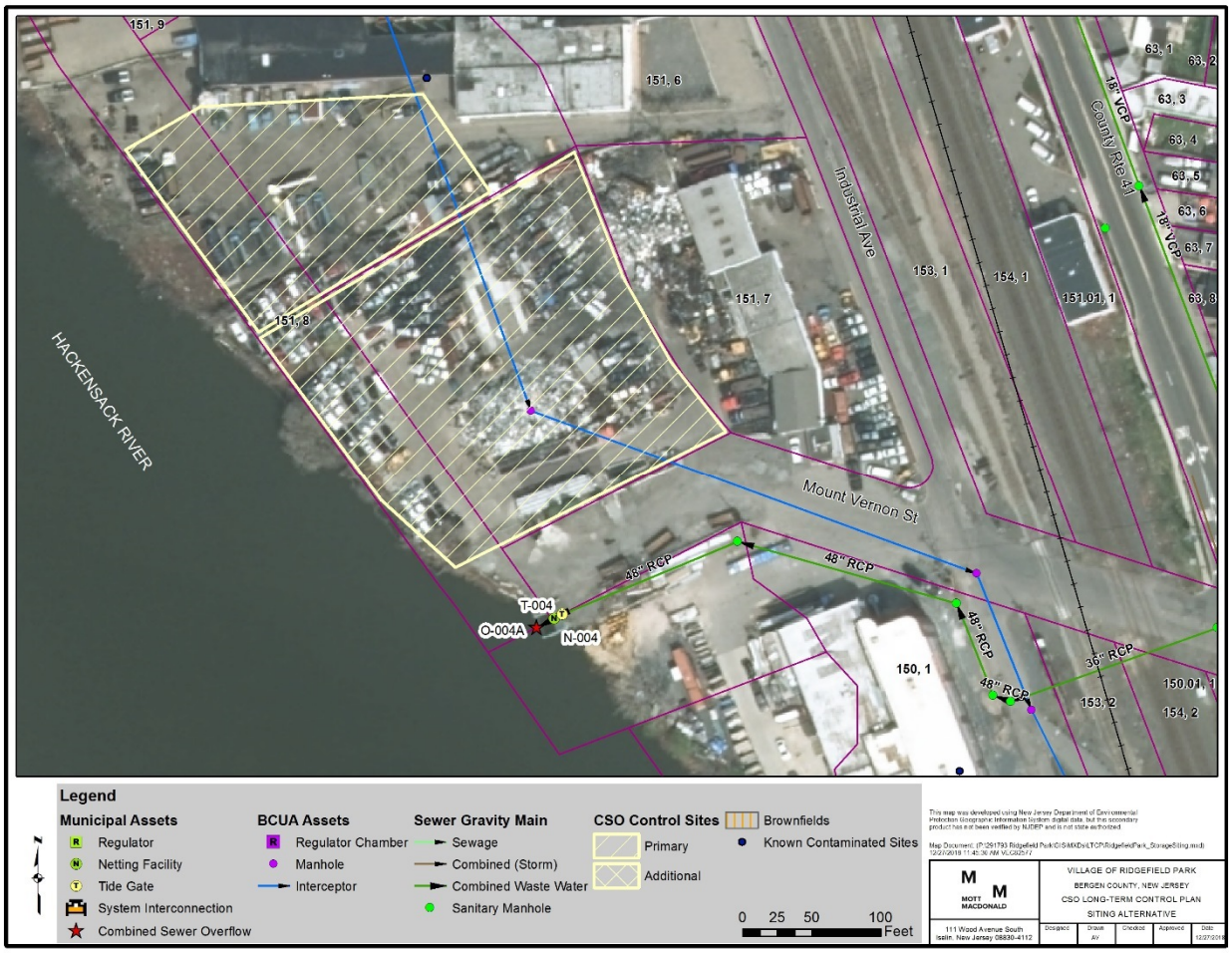


Figure 6-8: Industrial Use Scape Facility

There is underutilized space that is currently used as a materials yard (stockpiling area) for Marble.com, see Figure 6-9. This site is about 650 feet south of Outfall 004A and has an area of approximately 1.3 acres.

Figure 6-9: Industrial Materials Yard

6.3.5 Outfall 005A

There are two areas of underutilized space with car and truck parking adjacent to Outfall 005A and Regulator R-5 with areas of 0.8 acre and 0.2 acre, see Figure 6-10. The lot is owned by the Village of Ridgefield Park and is occupied by the Ridgefield Park Department of Public Works (DPW). The configuration of the site may limit the underground facilities that are feasible. Construction efforts would have to be coordinated with the DPW so that vital Village functions could carry on.

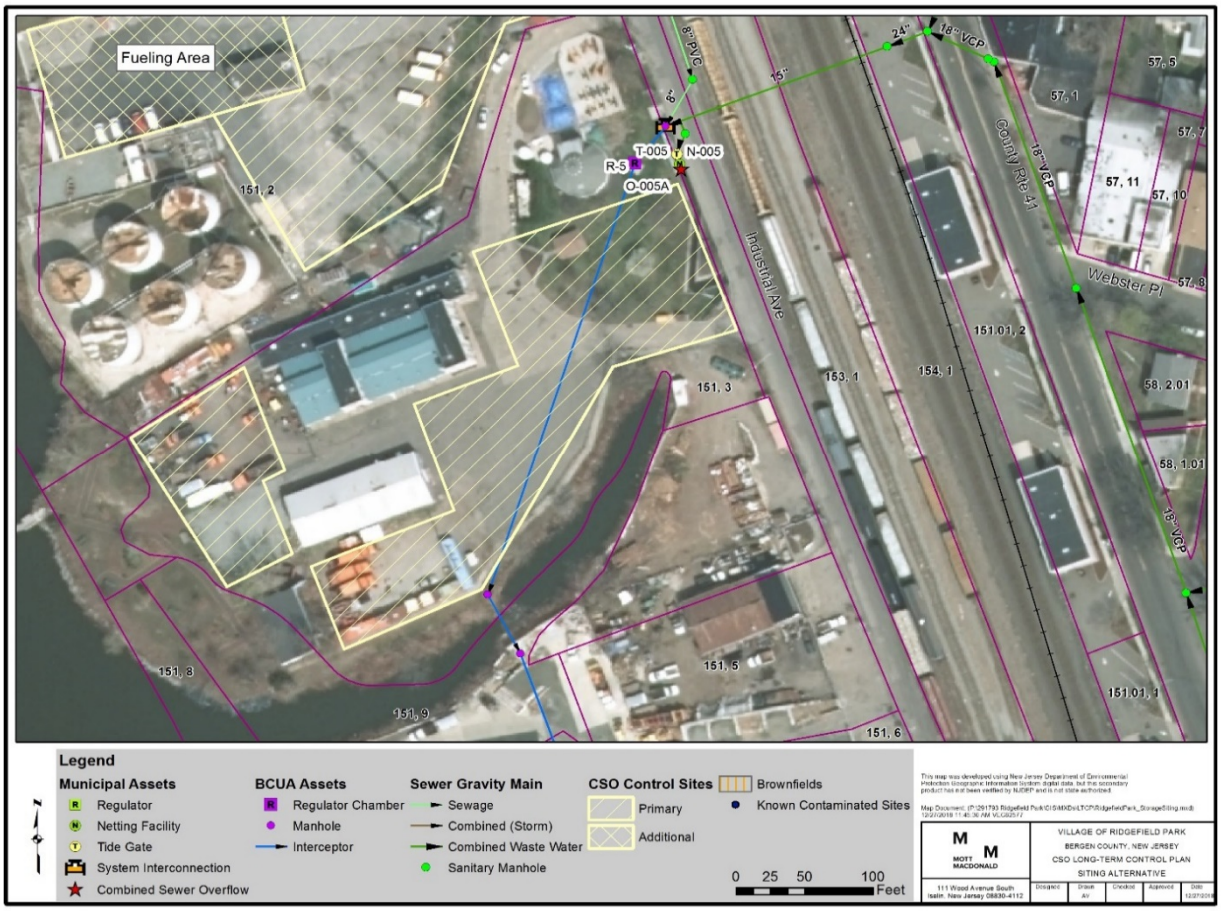


Figure 6-10: Village Department of Public Works site

6.3.6 Outfall 006A

There is an area of underutilized space with school bus parking just south of Outfall 006A with an area of 1.6 acres, see Figure 6-11. The lot is occupied by Durham School Services and appears to have a fueling station, which means that it may also have underground fuel storage tanks. Because of this, the space available for underground storage may be limited to approximately 1.2 acres. Aboveground facilities would need to be limited to reduce impacts to school bus parking and operations. It is also a “Known Contaminated Site” associated with the Dowling Fuel Company with a remedial level of C-1 (which means the soil is contaminated with a known or identified source and there is potential groundwater contamination). If this area is large enough, it could be used to store or treat combined sewage from both Outfall 006A and Outfall 005A. It should be noted that the Village of Ridgefield Park is reviewing the elimination of Outfall 006A and diverting all dry and wet weather flows to Regulator 5. If elimination is feasible, this site could be used to control flows from Outfall 005A.

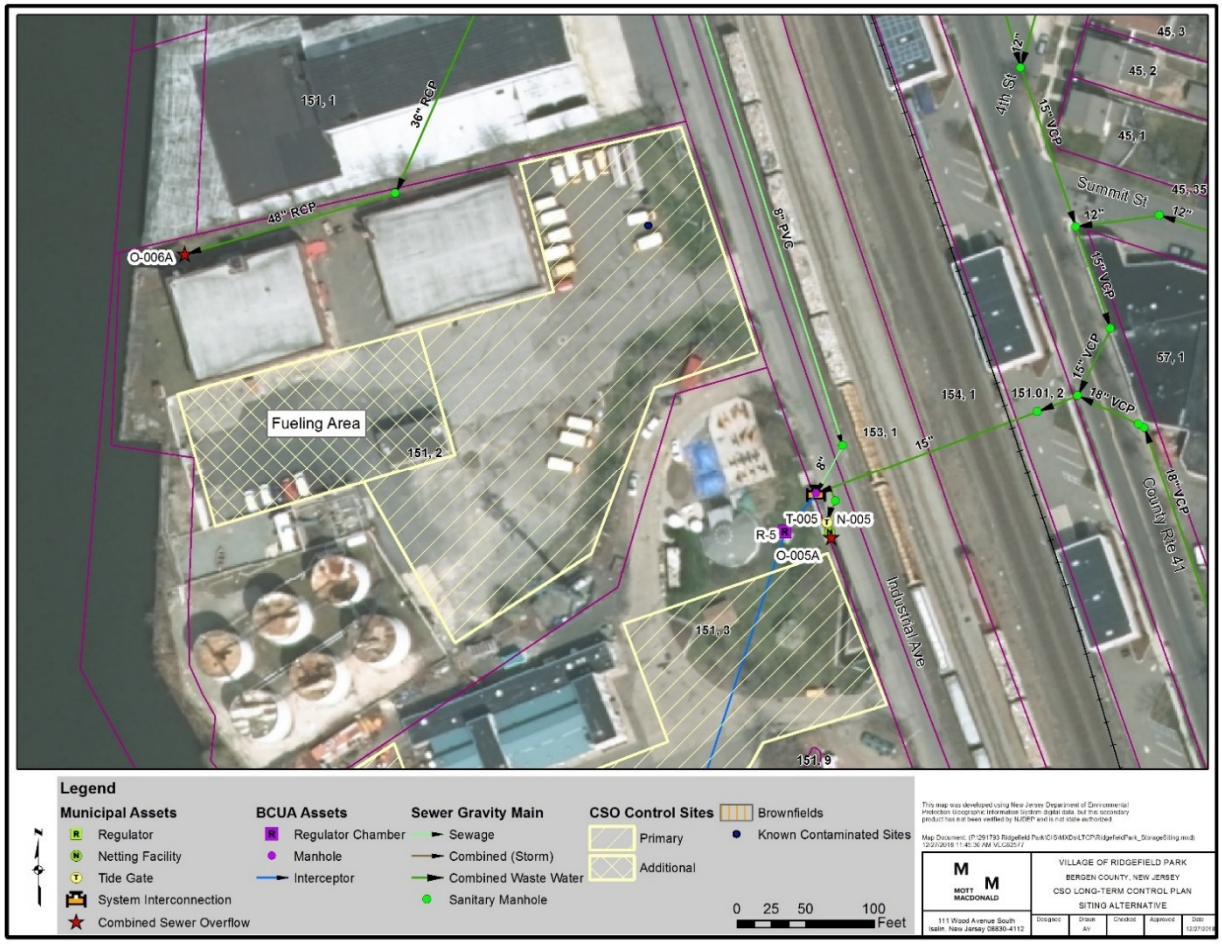


Figure 6-11: Industrial / Parking

6.3.7 Siting Analysis Summary

The results of the siting analysis are summarized in Table 6-1.

Table 6-1: Summary of End-of-Pipe Sites

Site ID	Area (ac)	General location	Dominant Land Use	Near known contamination?	In 100-yr flood zone?	In Waterfront Redevelopment Area?	Green Acres?
001A	1.35	McGowan Park, between Bergen Turnpike and Overpeck Creek	Greenspace	N	Y	Y	Y
002A	1.5	County Rte. 24 between Alpine Ct. and Main St.	Commercial	N	Y	Y	N
002A	0.5	Ferris Park, at County Rte. 24 and College Pl.	Greenspace	N	N	N	Y
003A	0.6	Cedar Street and Spruce Ave	Industrial	N	N	N	N
003A	1.5	Brewster Park, at Main Street Bypass and Brewster Ave	Greenspace	N	N	N	Y
004A	1.2 + 0.6	Mt. Vernon Street and Industrial Ave	Commercial	Y (0.6 ac site)	Y	Y	N
004A	1.3	Cedar St and Industrial Ave	Commercial	N	Y	Y	N
005A	0.8 + 0.2	Industrial Avenue and Webster Place	Industrial	N	Y	Y	N
006A	1.2	Industrial Ave and Summit Street	Industrial	Y	Y	Y	N

7 Alternatives Evaluation

7.1 Introduction

The NJDEP issued a General CSO NJPDES Permit in 2005 that required that all permittees with CSO outfalls conduct and develop a CSO LTCP Report by April 2007. The report was to evaluate and develop present worth costs for construction and operation of various alternative controls, to meet specified levels of control of CSO discharges and to report on the pollutant loads reductions based on present worth costs. CSO controls evaluated at that time included storage, satellite treatment, disinfection, and sewer separation. While the cost factors have changed, and have been updated as noted in Section 5, the cost data did reveal certain trends that are still pertinent. Important conclusions reached during this study included:

- the cost of construction and operation and maintenance of consolidated facilities are always significantly lower than developing alternative controls at each outfall provided the outfalls are relatively close; and
- the cost of satellite treatment and disinfection was relatively the same whether you were attempting to obtain a 50% or 90% reduction of pathogens. The only cost differential was the amount of chemical disinfectant used.
- The cost of sewer separation on a large-scale basis was not cost-effective and should only be evaluated where other factors such as the overall condition of the sewer system, and/or partially separation has already been undertaken in a region.

Accordingly, it was determined that evaluation of off-line storage or satellite treatment at individual outfalls would not be conducted and that satellite treatment and disinfection would be conducted at higher control levels only.

The following alternative control programs were evaluated further:

1. The elimination of Outfall 006A by removing the vortex valve at Regulator 6 and plugging removing the overflow pipe just downstream of the regulator. If needed, evaluate replacing the downstream 8” sewer segment with a larger pipe to reduce the upstream hydraulic grade.
2. Consolidated tank storage at Outfalls 001A and 002A; and at Outfalls 003A, 004A, 005A and 006A.
3. Tunnel storage with a single tunnel serving all outfalls/consolidated outfalls.
4. Consolidated end of pipe treatment at Outfalls 001A and 002A; and at Outfalls 003A, 004A, 005A and 006A.
5. Compete sewer separation
6. Green Infrastructure to treat 2.5%, 5%, 7,5% and 10% of Village’s impervious surfaces in combined sewer areas.

7.2 Presumptive and Demonstrative Approaches

As discussed in Section 3.3, the analysis is proceeding with the demonstration approach, but this may change during the Selection of Alternatives.

7.2.1 Targeted CSO Control Goal

The magnitude of the facilities in terms of CSO volume managed is the primary driver of both its cost and effectiveness. Accordingly, a procedure was developed to achieve the desired control objectives, in this case limiting the overflows to 0, 4, 8, 12 or 20 during the Typical Year. Since the permit requires the levels of control to be established based on the hydraulically connected system it is not adequate merely to achieve the desired number of overflows at each individual outfall, or within Ridgefield Park. Prior to

the evaluation it was necessary to determine for the BCUA system what storm events must be controlled for each level of control. Since the LTCP may incorporate a mix of volume-based controls (storage) as well as peak flow-based control (treatment) the same sets of storms were established for either control methodology.

Each level of control has a corresponding list of storms which would not be fully captured or treated as a result of the control. For example, for a single outfall, to achieve 4 overflows, the fifth largest storm would need to be stored and ultimately sent to the WWTP for treatment. However, since sewersheds respond to precipitation differently due to sewer system characteristics such as land use, size, and shape, the four largest storms may also vary in storm characteristics (i.e. peak intensity) between watersheds. Accordingly, a system-wide list of storms was established by identifying the events that generate the greatest volume of overflow system-wide. The Typical Year storms ranked systemwide by overflow volume are listed in Figure 7-1, which identifies the allowable overflow events for each level of control. This same list is applied to peak flow controls to establish consistent levels of control regardless of which control technology or combination of control technologies is employed. It is noted that by imposing a system-wide level of control, the control required at each outfall may be significantly higher than if the outfall was considered individually. Thus, some outfalls may be limited to one, two or three overflows to achieve the systemwide goal of four overflows, to meet the objectives of the overall CSO LTCP.

	Rank	Event	Rainfall Depth (in)	Peak 1 hr Intensity (in/hr)	Date
Top 4 Storm Events	1	49	3.68	0.53	9/28/2004
	2	46	2.21	0.63	9/8/2004
	3	48	1.99	0.32	7/12/2004
	4	36	1.67	0.25	4/12/2004
Top 8 Storm Events	5	56	1.67	0.25	4/25/2004
	6	35	1.66	0.33	7/23/2004
	7	32	1.63	0.33	2/6/2004
	8	37	1.60	0.64	7/18/2004
Top 12 Storm Events	9	6	1.50	0.85	11/28/2004
	10	23	1.45	0.41	7/27/2004
	11	38	1.44	1.33	9/17/2004
	12	15	1.39	0.40	6/25/2004
Top 20 Storm Events	13	44	1.08	0.10	11/12/2004
	14	17	1.08	0.99	5/12/2004
	15	34	1.03	0.20	11/4/2004
	16	43	1.00	0.69	7/5/2004
	17	52	1.00	0.18	12/1/2004
	18	57	0.94	0.60	8/16/2004
	19	24	0.84	0.81	8/21/2004
	20	22	0.83	0.20	12/6/2004

Figure 7-1: Ranking of top 20 overflow events in the Typical Year, by volume

7.2.2 Evaluation Procedure

Each alternative was implemented in the model, and then sized for the various desired levels of control. Its performance was evaluated based on the resulting number of overflows, reduction in CSO volume and percent capture achieved.

7.2.3 Village Specific Percent Capture Calculation

There are many ways to calculate a percent capture. For purposes of evaluating alternatives a conservative percent capture was developed for the Village's collection system. The Village is combined with limited separately sewered areas and no separate stormwater systems were modeled, while the limited number of separate sanitary areas were. This calculation methodology and result and will differ from the methodology and results that will be applied to the hydraulically connected BCUA system. It is noted the approach applied may not be applicable to a larger system with multiple combined communities and additional sanitary communities.

7.2.3.1 Wet Weather Period

InfoWorksICM allows the export of "scalar" data for subcatchments. This data represents the cumulative outflow from the subcatchment in MG. It was conservatively estimated that wet weather was occurring when runoff is being generated by the subcatchments. Accordingly, the runoff time series scalar data for each timestep was subtracted from the prior timestep. If the difference was zero, there was no runoff and the period was designated as dry, if the difference was positive, the model was producing runoff and the period was designated as wet. The result was 26.7 days of wet weather and a total of 78.6 MG of runoff entering the collection system.

7.2.3.2 Wet Weather Capture Volume

The total subcatchment outflow time series scalar data which includes groundwater infiltration, RDII, base sanitary flow and runoff was exported. Each timestep was evaluated, if the time step was designated as wet, then the flow volume from that period was summed to create an annual wet weather flow volume entering the system. The total wet weather capture volume was 164.7 MG.

$$\%\ Capture = 1 - \left(\frac{Overflow\ Volume}{Total\ Wet\ Weather\ Capture\ Volume}\right)$$

$$1 - \left(\frac{50.3}{164.7}\right) = 69.5\%$$

To achieve 85% capture the overflow volume would need to be reduced from 50.3 MG to 24.7 MG.

7.3 Development and Evaluation of Alternatives

Each alternative may correlate directly to a technology that passed through the screening or be built of several technologies, the overall alternatives should be based on the CSO control goals. There may be variations such as different tunnel alignments or alternatives that address different levels of control.

7.3.1.1 Siting Issues

Siting is one of the most challenging issues facing the implementation of CSO control. Often the controls are large and require open areas on the surface and clearance from underground utilities and facilities. While at this level of evaluation a complete surface and sub-surface survey is not possible, use of existing information and experience can help evaluate the suitability of a site for the conceptual facilities. To that

end, the available land analysis presented in Section 6 was conducted. Drawing on the available land analysis, a conceptual site plan was developed for each facility making use of aerial photographs and other readily available information.

7.3.1.2 Institutional Issues

Institutional issues refer to permitting requirements, likelihood of receiving permits, and timeline to receive permits, regulatory compliance in terms of water quality improvements, and ownership of the site (public vs. private). Regulatory considerations such as Green Acres, flood hazard area, wetlands, and threatened or endangered species are also evaluated, as well as zoning/planned development of the site by the municipality, and whether the site could be re-purposed for multiple-use (such as a parking facility over a storage tank). Institutional issues also refer to built-in limitations such as capacity in the BCUA interceptor and WWTP.

Permitting is an important Institutional Issue and is typically a major factor in a project's design schedule. The following is a list of anticipated major permits applicable to the alternatives being analyzed:

- Waterfront Development Permit – Construction will take place within the waterfront development area, which extends from inland from the mean high water (MHW) line a minimum of 100 feet and a maximum of 500 feet, with the development area being truncated at the first paved public road or surveyable property line beyond 100 feet from MHW. The portion of the project within the Waterfront Development Area would also need to comply with the applicable Flood Hazard Area requirements. Restrictions are much more stringent for in water work, including the Flood Hazard Area prohibitions regarding placement of fill in the floodway. Waterfront Development Permits are typically issued within 90-days from receipt of an approvable application (construction permits).

- Flood Hazard Area Permit – A flood hazard area permit will be required for work within the floodplain outside of the Waterfront Development Area. Since the floodplain is Ridgefield Park is tidal, much of the work will be eligible for permit-by-rule, however certain facilities may require individual permits. While most areas within Ridgefield Park are paved, there may be some small areas of riparian zone vegetation impacts. Flood Hazard Area Permits are 90-day construction permits, however, there are mechanisms which could delay the issuing of a permit beyond 90-days.

- Soil Conservation District (SCD) Certification – The New Jersey Department of Agriculture implements the New Jersey Soil Erosion and Sediment Control Act, New Jersey Statutes Annotated (N.J.S.A. 4:24-39). The program is administered by the local soil conservation district, in this case the Bergen County Soil Conservation District. The program requires soil erosion and sediment control measures be in place during land disturbance activities and requires certification of a soil erosion and sediment control plan for all disturbances greater than 5,000 sf. It also addresses the post construction and long-term site stabilization. Certification from the soil conservation district is a 90-day construction permit. If the project exceeds one acre of disturbance then authorization to discharge stormwater during construction is required from the State, generally under a NJPDES Master General Permit, Program Interest Group 5G3, which is an online approval based on the SCD certification.

- Treatment Works Approval (TWA) – Treatment Works Approval is required for modifications to the sanitary and combined sewer systems. There are regulatory thresholds for when a treatment works approval is required, however the activities associated with a LTCP would easily exceed those thresholds. Treatment Works Approval Permits are 90-day construction permits.

- Stormwater Management – While not specifically a permit, the State claims jurisdiction over major developments for projects that require Land Use permits. The Stormwater Rules (N.J.A.C. 7:8) primarily concern themselves with stormwater quantity, quality and recharge. Since Ridgefield Park is in a tidal flood hazard area quantity of discharge from the municipal separate storm sewer

system is not expected to be an issue. Recharge of groundwater, likewise should also not be an issue since Ridgefield Park is highly developed. The quality of discharge will be the largest challenge, primarily related to sewer separation projects. The NJDEP's current position is that sewer separation of an area containing more than one quarter acre of impervious area is a major development and must address the stormwater quality requirements for TSS removal. Proposed changes to the Stormwater Rules formalize the Department's policy surrounding sewer separation and implement green stormwater infrastructure requirements for separation. The proposed changes are not in effect yet, but may be finalized prior to the LTCP selection.

- Army Corps of Engineer Nationwide 404 Permit - The Army Corps of Engineers (USACE) regulates tidal waterways within New Jersey. The USACE does not regulate upland areas, as such, only disturbances below the MHW line would be regulated by USACE. Other agencies such as United States Fish and Wildlife Service (USFWS), National Oceanic and Atmospheric Administration (NOAA) and United States Coast Guard (USCG) may concurrently review the permit application. A more detailed impact analysis such as an Essential Fish Habitat Assessment may be required as part of the USACE submission. USACE permits do not have a set review timeframe.

- Wetlands Permits – Any wetland habitats identified landward of the MHW would be regulated as freshwater wetlands. A wetland delineation and investigation would be accomplished based on the 1989 Federal Manual for Identifying and Delineating Jurisdictional Wetlands, which is the recognized wetland delineation manual for the State of New Jersey. Any proposed impacts to identified freshwater wetlands or transition areas would be subject to the rules applied under N.J.A.C. 7:7A. Freshwater Wetland Permits do not have a set review timeframe; however, if submitted concurrently with a Waterfront Development Permit and/or Flood Hazard Area Permit, approvals are typically issued concurrently within 90-days.

- Tidelands – The State lays claim to all lands now or formerly flowed by the tide, where the land is held in public trust. Projects making use of the land must either obtain a tidelands license (lease) or be granted (purchase) the riparian rights. All such grants and licenses must be approved by the Tideland committee in a process that takes several months, and in case of granting riparian rights the appraised market value must be paid to the State. The State has tidelands claims along the Hackensack and Overpeck River banks, possibly the result of prior installation of bulkheads and land fill, see Figure 7-2. It is not known if any of these areas have been granted in the past. This will need to be investigated following selection of the final implementation plan.

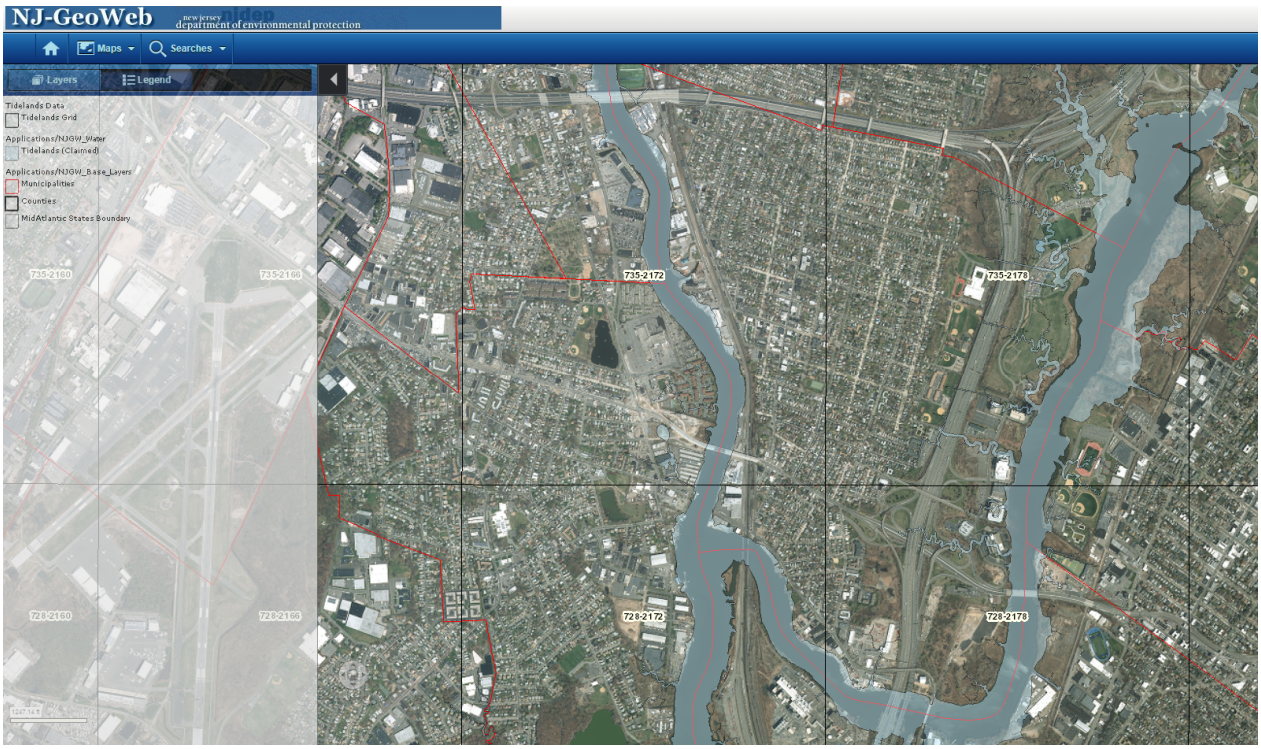


Figure 7-2: NJDEP Geo-Web showing Tidelands claims in Ridgefield Park

- Local Permits – Depending on the nature of the project there are a number of local permits that may be required. These may include zoning permits, construction permits, land use board approval, road opening permits, etc. It is assumed that since the LTCP will be conducted by the Village, they will assist in obtaining these approvals and that for certain approvals (i.e. land use board) only a courtesy review will be required. In general, local permits are not considered a major obstacle.

- Green Acres – Minimal areas appear to be impacted by Green Acres, however use of Green Acres land for CSO facilities of any sort is currently considered a diversion of use. This is a lengthy and costly process that should be avoided where possible. Accordingly, Green Acres sites will only be considered to address the stormwater within the Green Acres property through green infrastructure which is allowable under the current regulations. The State is investigating greater flexibility in the use of Green Acres property for CSO control facilities, possibly allowing them to accept offsite stormwater for treatment in green infrastructure.

- County and State Highway Permits – Approvals will be required for work impacting County and State roads. There are several County roads in the Village of Ridgefield Park which could be impacted by construction. State highways through the Village of Ridgefield Park include Route 46, I-80 and significant areas of NJ Turnpike, which is administered by the State DOT.

- Railroad Occupancy – Several industrial rail lines as well as tracks are located along the west side of the Village of Ridgefield Park. Agreements to acquire or occupy rail rights-of-way are difficult, expensive and time consuming to obtain.

7.3.1.3 Implementability

Implementability refers to considerations that could present challenges or prevent the construction of an alternative. This includes such factors as:

- Site access – If space is available, but it cannot be efficiently accessed, the cost to construct and maintain LTCP facilities could be prohibitive. This could be a consequence of geography or existing infrastructure.

- Ownership and ease of acquisition or easement – Ultimately, the Village is responsible for the operation and maintenance for LTCP facilities. Therefore, they must be able to acquire (purchase) the property the facilities are sited on, or obtain permanent easements that will allow for maintenance, as well as potential future upgrades.

- Land area available – CSO control facilities are large and often do not lend themselves to be distributed to sites remote from the CSO outfalls. While some challenges associated with land area can be overcome through diversion piping doing so may greatly increase the overall project cost.

- Environmental considerations – In addition to the permits required as discussed under institutional issues other factors such as soil type are relevant to some of the alternatives both for infiltration/dewatering, as well as for tunneling construction/excavation.

- Compatibility with existing infrastructure – This is considered in terms of any existing structures or utilities that would need to be relocated or decommissioned. Relocation of utilities can greatly increase the cost of a specific project and may have a potential impact on the local community, as it often requires shutting down of the utility while it is being relocated.

7.3.1.4 Public Acceptance

Public acceptance refers to the degree to which community residents, businesses and institutions would be impacted, or perceive the alternative to be favorable or unfavorable. While subjective and hard to predict public acceptance is important to consider. This includes contemplations such as:

- Construction disturbance – Construction brings a variety of unwelcome impacts to a community, such as traffic, dust, noise and vibration. These are unavoidable to some degree, but the facilities' construction methods selected can serve to reduce or augment these concerns. For example, an alternative that required pile driving produces much more noise and vibration than traditional excavation, or other potential methods for pile installation. The duration of the construction, and to a certain extent the method of construction can impact costs which should also be a considered.

- Visibility – Residents prefer solutions that are aesthetically pleasing and have an expectation that their community will be left looking as nice or better than it did prior to the project. There may also be concerns that the visual impact may reduce property values.

- Impact to community spaces – Public areas such as parks are amenities, and if their functionality is diminished the public will object, however opportunities exist to create green space and other community amenities as well.

- Community character – Communities are generally built around common land uses, for example industrial areas are generally separate from residential areas. Accordingly, opposition could be expected if an industrial looking CSO facility was sited in a residential area. Likewise, facilities that could be perceived as not good for business may not be accepted in a commercial area. Facilities that potentially produce noise or odor are also expected to create opposition.

- Traffic impacts – Traffic impacts may occur during construction and after construction. During construction, consideration must be given to the location, severity, and length of time of the impacts. Acceptance to impacts that may persist after construction would depend on the severity of the impact, both in terms of residents impacted and magnitude of inconvenience.

- Cultural resources – Sites of historic significance should be avoided. It is also possible that the historic significance of sites may be highly localized and not detected until the plan is well advanced.

- Environmental justice – In general the project must avoid in fact and perception that the impacts are skewed towards those with lower socio-economic standing.

- Community resources – Projects that impact community resources are likely to receive higher levels of opposition. Community resources may include, schools, houses of worship, emergency services and community centers. This may include projects that directly impact community resources such as taking part of the property, or indirectly by impeding access or function such as key routes to hospitals or for emergency services.

Public acceptance can take many forms. In some areas residents and business may not be concerned and accept the construction, however, it is also possible for stronger levels of community opposition to occur, and for a small but vocal minority to swing public perception. Opposition groups can be extremely vocal, active and well-funded. There is also the possibility that opposition groups can influence local election in favor of those that oppose the CSO LTCP or mount legal challenges. While public outreach such as the CSO Supplemental Team and public meetings can mitigate these challenges, it cannot altogether eliminate them as risks to the project.

7.3.1.5 Performance

There is no guarantee that a proposed technology will work until it is implemented. This uncertainty can be greatly mitigated through the selection of the technology. Some considerations are:

- Past performance – Is the technology well tested with a history of successful applications to CSO, with reliable data supporting its performance.
- Performance Flexibility – CSO flows are known for rapid changes, in both quantity and quality, the technology selected must not only be able to accommodate the design conditions, but also the rapid changes that take place prior to reaching design conditions. CSOs can occur anytime of the year and under a variety of meteorological conditions and the controls must function properly under all such conditions.
- Operational Flexibility – Most municipalities cannot afford highly specialized staff to operate and maintain facilities that are used intermittently. Thus, the technology must be simple to operate for available staff that must also fulfill other duties. Specialized skills should only be required infrequently and then under planned conditions.
- Reliability – While a technology may be successful, it must function consistently. CSO flows create a harsh environment for equipment. Equipment typically functions best under continual use, whereas CSOs are intermittent which can lead to seizing between uses.
- Effectiveness – Does the technology have the ability to reduce CSO volumes or loadings to the receiving water. This was largely addressed in Section 4, and technologies evaluated in this section are considered to be effective in an overall sense and results documenting their specific effectiveness will be provided.

7.4 Preliminary Control Program Alternatives

7.4.1 Control Program 1 – Elimination of CSO 006A

7.4.1.1 Description

Considering the small volume of overflow from outfall 006A and the configuration of the Ridgefield Park system it may be feasible to eliminate outfall 006A.This control program seeks to consolidate Outfalls 005A

and 006A by eliminating the wet weather discharge from the two structures that make up Regulator -006A. This would reduce the number of outfalls to be addressed by other alternatives (See Figure 7-3).

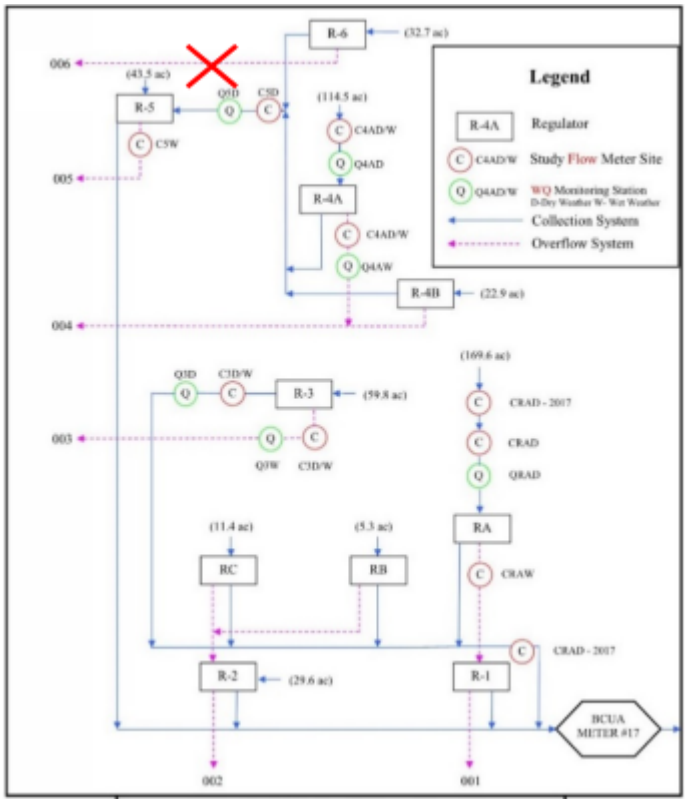


Figure 7-3: Ridgefield Park system schematic showing elimination of Outfall 006

7.4.1.2 Analysis

This was analyzed in the model by eliminating the modeled weirs at the two structures in the InfoWorksICM model, as shown in Figure 7-4.

Figure 7-4: Model Configuration for Eliminating Outfall 006A

The resulting hydraulic profiles were reviewed for the Typical Year. Surcharge and potential flooding were observed in the pipe that connects R-006 to R-005. It was determined that to complete the consolidation, addition system upgrades would be required. There is no compelling need or water quality benefit to eliminating the outfall since it does not discharge to a sensitive area, nor will the consolidation relocate Outfall 006A to a different watercourse. Accordingly, this control program will receive no further consideration, since it would impose additional costs without generating water quality benefits.

7.4.2 Control Program 2 – Consolidated Tank Storage

7.4.2.1 Description

This control program consists of siting storage tanks consolidating outfalls to capture overflows in the tank, retain the overflows and then return them the interceptor to receive treatment at the BCUA WWTP. Each facility consists of:

- Diversion structures with fine screens;
- Consolidation piping
- An offline below grade tank equipped with a flushing system and odor control;
- Tank overflow to an outfall;
- Dewatering pumping station; and
- Discharge connection back to the interceptor.

Storage tanks were input into the model to identify any impacts to CSO reduction. The tanks were scaled to achieve the desired level of control. The cumulative dewatering rate from the tanks was limited by the capacity of the Ridgefield Park intercepting sewer. This was achieved by tying the operation of the dewatering pump to the water level in the Ridgefield Park Branch Interceptor just upstream of the

Overpeck Creek crossing and limiting discharge rates to the capacity of the branch intercepting sewer. As discussed, the performance levels have been coordinated with the overall BCUA hydraulically connected system. The system hydraulics were refined to prevent adverse impacts on the collection system upstream of the regulator. The regulator weir heights were not changed so that the system continues to maximize flow to the treatment plant. However, to allow for conveyance of peak runoff rates the weirs may be lengthened. The evaluation of practicality and feasibility draws on the siting analysis to identify locations for each facility, and drives the consolidation of select facilities. Modeling was performed for the range of control alternatives 0, 4, 8, 12 and 20 overflows.

This control program offers some advantages over placing tanks at individual outfalls:

- The result will be only leave two active discharge outfalls; the consolidated outfall for Outfall 001A and 002A and an outfall for Outfall 003A through 006A. This will simplify future permitting and effectively eliminate four outfalls.
- This control program will result in fewer facilities for the Village to maintain.
- It reduces the number of parcels impacted and reduces the number of properties the Village would need to acquire.
- With some limitations, existing land uses can be maintained over the tank with minimal surface disturbance after construction. It may also be possible to create public amenities on top of the tanks.

There are also some potential disadvantages:

- There will be more disturbance and interruptions to local streets because of the consolidation piping.
- There will be additional costs associated with the consolidation piping, however it is anticipated that these would be offset by fewer pumping stations and the greater construction and operating cost efficiency of larger tanks.

7.4.2.2 Analysis

The consolidated tanks were implemented in the model by sizing pipes ranging from 18 inches to 6 feet in diameter to convey the modeled runoff from Outfalls 001A and 002A, and Outfalls 003A through 006A to the consolidated tank sites. The consolidation piping capacity was evaluated for the Typical Year and it was sized such than there would be no adverse impacts (water surface increases) to the upstream system. The overall required volume was initially estimated using a spreadsheet analysis on the individual outfalls. The consolidated tank volumes were initially estimated by adding the volumes of the individual tanks these volumes were then refined using the model. Pump controls in the model were tied to the interceptor to adhere to the requirement not to increase peak flows to BCUA. The consolidated tank was modeled with an overflow weir and outfall equipped with a tide gate to allow the discharge of excess flows without causing upstream flooding.

The sizing of storage control facilities in the InfoWorks model for multiple outfalls is time consuming to model, computationally expensive to run test simulations, and requires the processing of massive amounts of data. Operationally, a storage tank captures overflows until it is full, once it is full, excess volumes are discharged as overflows. When the storm is over, the storage volume is dewatered back to the interceptor at a set flow rate. Thus, initially the storage can be sized based on the Typical Year baseline overflow rates and a set of rules for dewatering. A spreadsheet analysis was used to perform this analysis and the resulting volume modeled in InfoWorks and refined to address hydraulic issues, and to achieve the desired level of control.

Time series data at a 15 min timestep for each overflow is available from the InfoWorksICM model. To this data a series of rules was applied to divert the overflows into a conceptual storage facility. The evaluation consisted of calculating the volume of overflow for each timestep, and provided there is room in the storage, the overflow is diverted to the storage facility. The tank volume is tracked and once the volume is exceeded the tank is considered full and no additional volume is accepted. Once the storage facility is calculated to be full, remaining flows are tracked as overflows.

The storage was assigned a dewatering rate such that the dewatering rate would be sufficient to empty the tank in 24 hours or the capacity of the interception sewer. The capacity of the interception sewer was about 1.5 MGD. In the upper reaches, the dewatering rate was limited to 1.2 MGD to allow room for sanitary flows. The dewatering was only applied to the storage if there was no overflow from the regulator for a minimum of 2 hours, to allow the interceptor hydrograph to recede and create available capacity in the interceptor. If the overflow resumed or a new overflow began, then dewatering ceased until there was another period of 2 hours with no overflow. Sample output can be seen below in Figure 7-5. Overflow events beyond the tank volume were tracked and a list of storms identifying remaining overflows generated. The list of remaining overflows generated was compared to the list of allowable storms (Figure 7-1), and the storage volume increased until the remaining storms consisted only of those allowable on the systemwide basis.

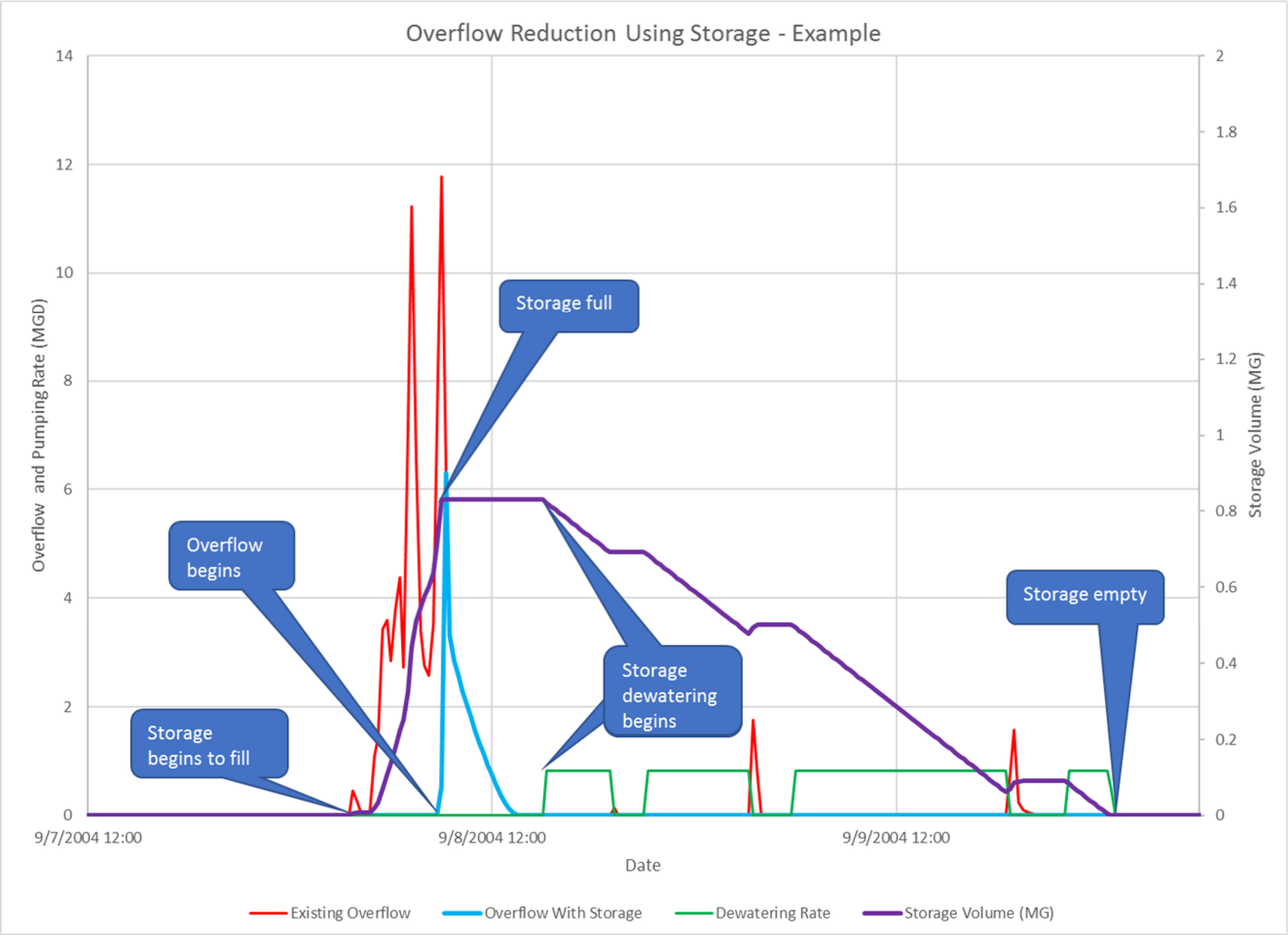


Figure 7-5: Example hydrograph of tank storage

The resulting tank volumes were then modeled in the InfoWorks model. A typical model configuration is shown below in Figure 7-6 and Figure 7-7.

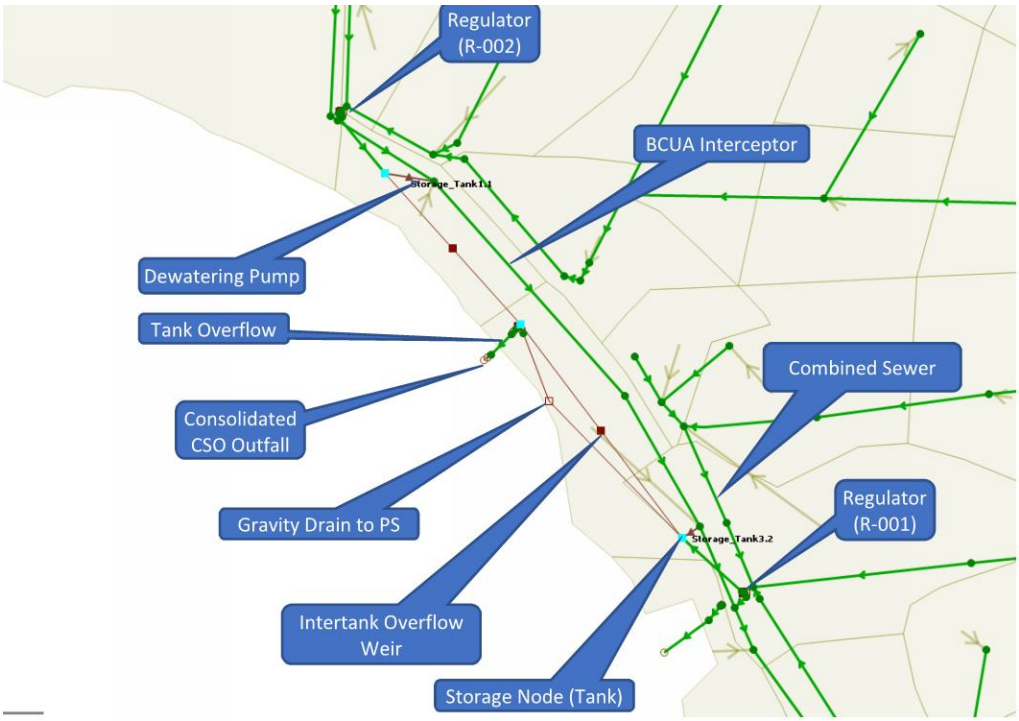


Figure 7-6: Point Storage - Sample Model Configuration at Outfalls 001A and 002A

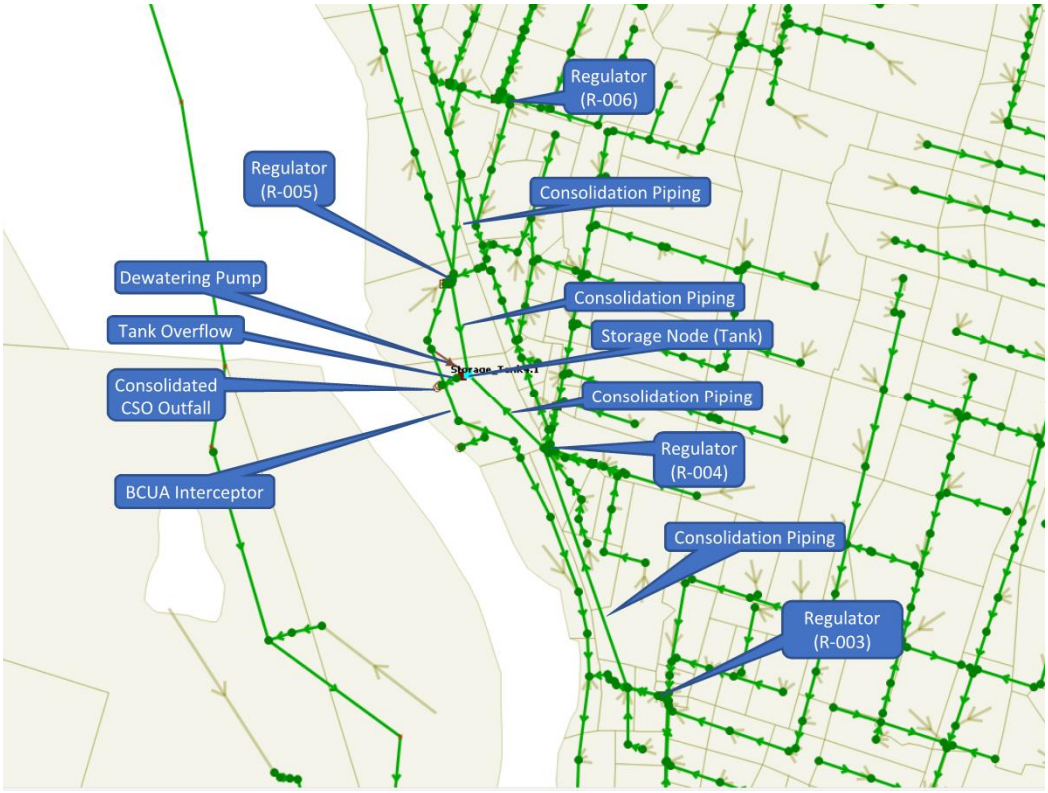


Figure 7-7: Point Storage - Sample Model Configuration at Outfalls 003A-006A

7.4.2.1 Siting Issues

A review of the availability of land in the proximity of each outfall is summarized in Section 6 of this report. This section notes that there is an underutilized commercial space (former restaurant) adjacent to Outfall 002A with an area of approximately 1.5 acres, The Village is in the process of purchasing this property to protect its ability to comply with requirements of their NJPDES permit. The east side of this property is only 290 feet from Outfall 001A, and adjacent to the VFW Property which is already owned by the Village. The consolidated Outfall 001A and 002A Storage Facility and diversion piping is illustrated in Figure 7-8.

In the same manner a commercial auto recycling yard was identified just north of Outfall 004A that appears large enough to provide a consolidated storage facility for Outfalls 003A, 004A, 005A and 006A, The consolidated storage and diversion piping for Outfalls 003A, 004A, 005A, and 006A is illustrated in Figure 7-9. The consolidated storage unit for these outfalls is illustrated in Figure 7-10. Since the elimination of 006A does not appear feasible as previously discussed, this outfall could also be diverted to the storage facility as noted.

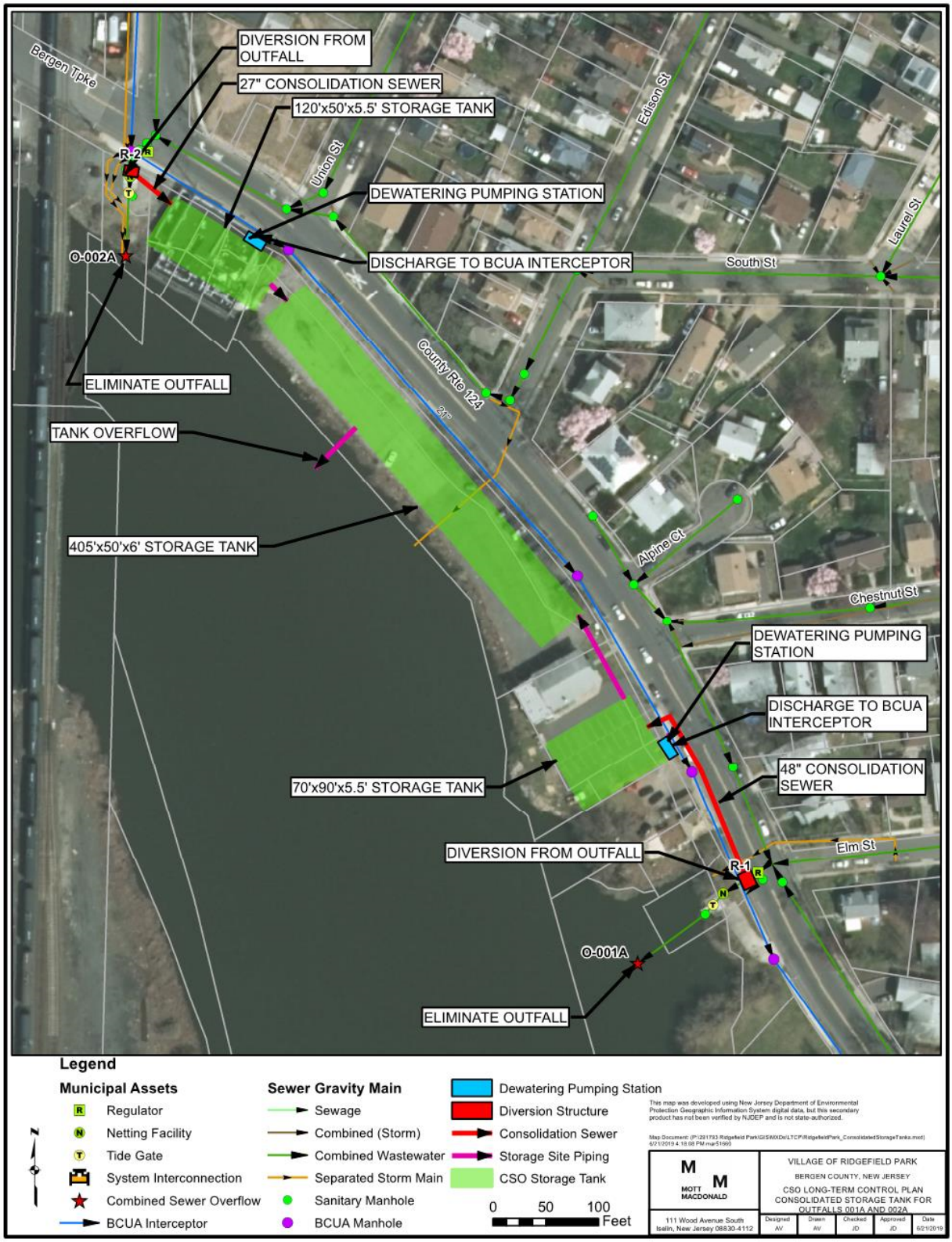


Figure 7-8: Consolidated Storage 001A and 002A

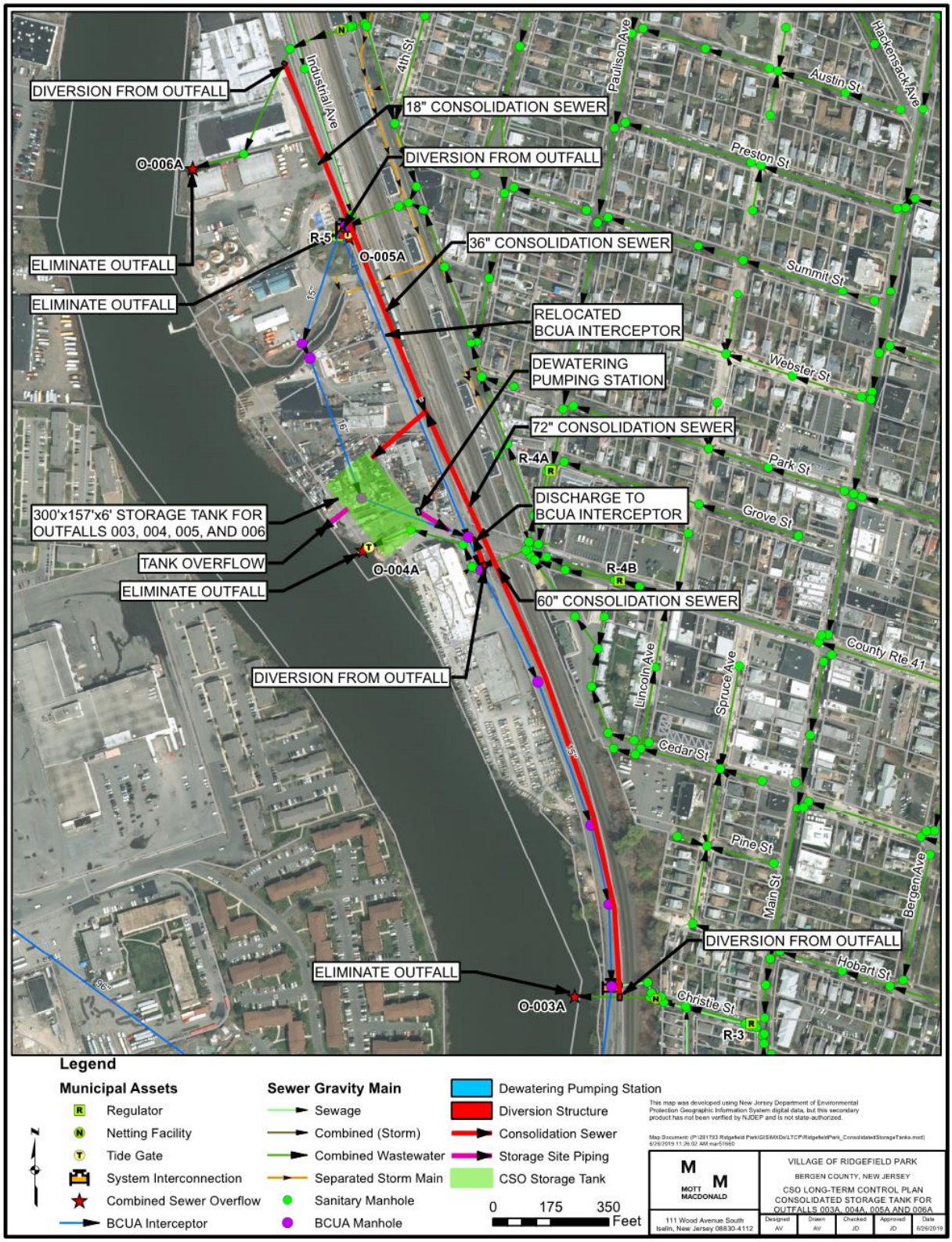


Figure 7-9: Consolidated Storage Outfalls 003A, 004A, 005A and 006A

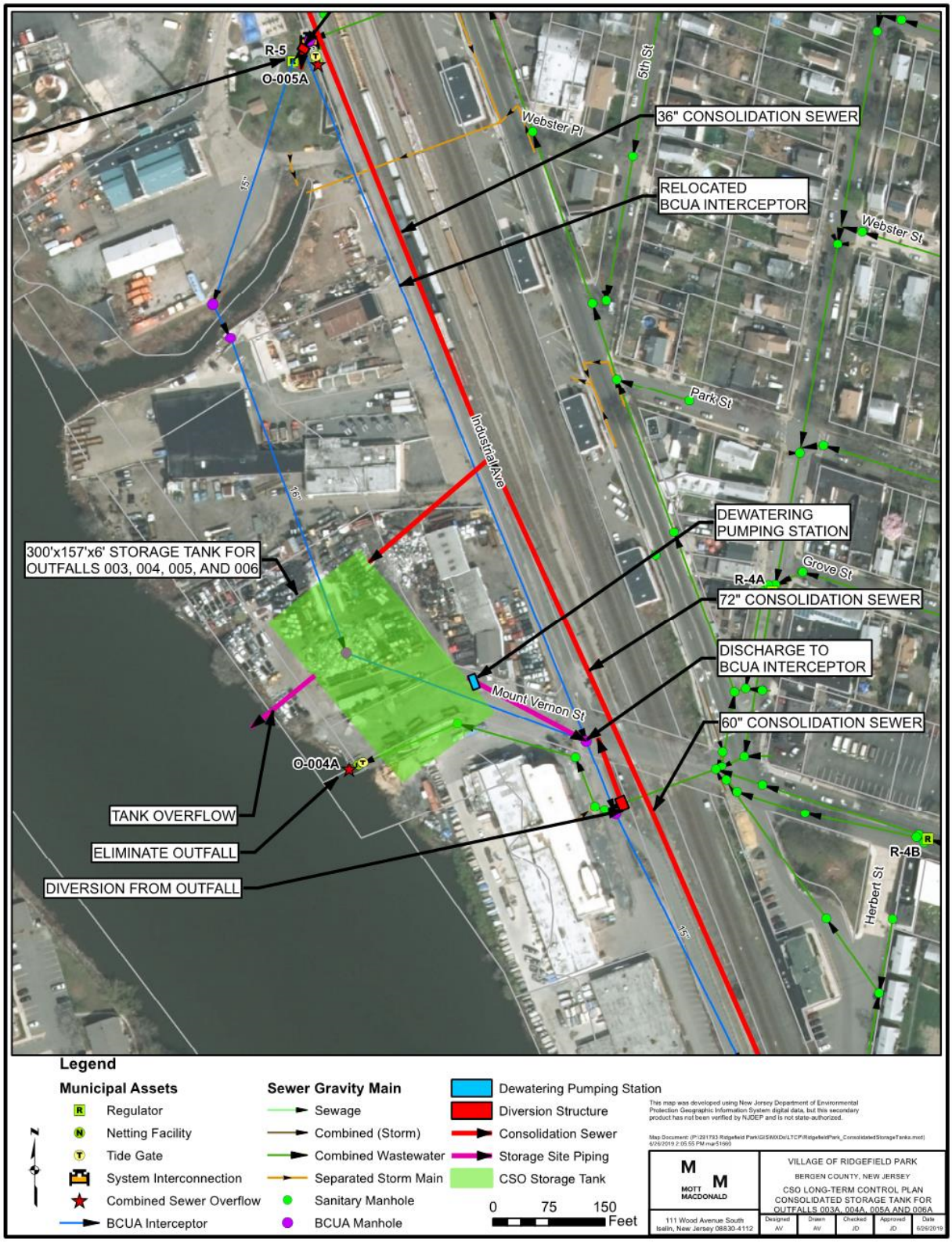


Figure 7-10: Consolidated Storage Outfalls 003A, 004A, 005A, and 006A.

7.4.2.2 Institutional Issues

The institutional issues surrounding Control Program 2 are typical of a large-scale construction project in an urban area. While located in an urban area, construction of the facilities associated with this control program will require environmental permits. Below is list of anticipated permits/approvals required:

- Waterfront Development Permit – required at both tanks and some diversion piping.
- Flood Hazard Area Permit – required at Outfalls both tanks and some diversion piping.
- USACE Nationwide 404 Permit – required for new outfalls.
- Local Permits – both tanks.
- SCD Certification
- Stormwater Management Compliance
- NJDEP Tidelands
- NJDEP Treatment Works Approval
- Coordinate discharge rates from storage facilities with BCUA.
- Railroad occupancy (possible for consolidating piping)

These permits are standard permits and while they must be obtained, they do not appear to have the potential to greatly extend the project schedule or add excessive risk to the project.

7.4.2.3 Implementability

Installation of storage tanks in urban areas can be challenging. The tanks have been sized assuming a 6-foot water storage depth. Since the intent is to fill them by gravity and the existing outfalls are approximately 8 feet below grade a total excavated depth of 14 feet is generally required. Excavating to this depth requires costly dewatering and support of the excavation, which is made more challenging by adjacent buildings which must be protected and monitored throughout construction. In addition, utilities impacted by construction must be relocated, protected, or supported, including the BCUA's Ridgefield Park Branch Interceptors. The tanks to be sited at Outfalls 004A through 006A are in proximity to existing buildings, whereas the tanks at 001A and 002A are further from buildings, or buildings that may be demolished as part of construction.

Control of groundwater will be a significant challenge. As noted previously noted, the tank sites are near the Hackensack River, or Overpeck Creek, which create additional issues and risks with keeping the site dewatered during construction.

There is little available information on the soil characteristics at the tank sites, however, given the proximity of the sites to the floodplain, soil conditions could be poor, and the tanks may need to be situated on piles. Piles may also be required to anchor the tanks, so they do not become buoyant in the event of a flood, or periods of high groundwater. Tidal flooding is a concern because high storm surge levels could produce inundation with little rainfall meaning the tank could be empty and prone to floatation.

7.4.2.4 Public Acceptance

The construction required for storage tanks is large and invasive making public acceptance of the project a concern. The consolidated tank near 001A is located on a parcel of land that was slated for redevelopment and the construction may be more acceptable in terms of public acceptance. The construction at 003A-006A is in an industrial area and may raise fewer concerns from the public, however, there could be a significant impact on the property owner during construction.

Once construction is completed, tanks are generally preferable from the stand point of public acceptance since most of the facility is underground. Aboveground features will still be required such as electrical facilities, odor control facilities and access points to pumps, flushing systems and the tank. There may be concerns with odors along Bergen Turnpike, where the opposite side of the road is residential. It is anticipated that the land above the consolidated tank at 001A and 002A will be converted to a park providing a public amenity and enhancing public acceptance, but this makes odor control even more important. It may be possible to reinstate the current business at the consolidated 003A-006A site, however, the load of the recycling operations may exceed the allowable loading on the tank.

7.4.2.5 Performance

The performance associated with Control Program 2 is summarized below in Table 7-1 through Table 7-6 , which provided performance details for the selected levels of control 0, 4, 8, 12 and 20 overflows during the Typical Year. These results represent the modeled performance of the control program when implemented in the 2050 baseline model as compared to the 2015 baseline modeling. Note that Consol 1 and Consol 2 refer to the new outfalls that the tank overflow weir discharge to. The existing outfalls will be abandoned.

Table 7-1: CP-2 Consolidated Storage, Typical Year Overflow Summary 0 Overflows

	Baseline 2015			Consolidated Storage 0 Overflows				Change		
Outfall	# of Events	Volume (MG)	Duration (HR)	Storage Volume (MG)	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)
001A	19	6.3	74	—	0	0.0	0	-19	-6.3	-74
002A	13	0.4	11	—	0	0.0	0	-13	-0.4	-11
Consol 1	—	—	—	2.6	0	0.0	0	0	0.0	0
003A	45	15.3	300	—	0	0.0	0	-45	-15.3	-300
004A	53	24.1	614	—	0	0.0	0	-53	-24.1	-614
005A	25	3.6	121	—	0	0.0	0	-25	-3.6	-121
006A	12	0.5	39	—	0	0.0	0	-12	-0.5	-39
Consol 2	—	—	—	5.5	0	0.0	0	0	0.0	0
Total	53	50	--		0	0.0		-53	-50.3	

Table 7-2: CP-2 Consolidated Storage, Typical Year Overflow Summary 4 Overflows

	Baseline 2015			Consolidated Storage 4 Overflows				Change		
Outfall	# of Events	Volume (MG)	Duration (HR)	Storage Volume (MG)	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)
001A	19	6.3	74	—	0	0.0	0	-19	-6.3	-74
002A	13	0.4	11	—	0	0.0	0	-13	-0.4	-11
Consol 1	—	—	—	1.4	1	0.0	1	1	0.0	1
003A	45	15.3	300	—	0	0.0	0	-45	-15.3	-300
004A	53	24.1	614	—	0	0.0	0	-53	-24.1	-614
005A	25	3.6	121	—	0	0.0	0	-25	-3.6	-121
006A	12	0.5	39	—	0	0.0	0	-12	-0.5	-39
Consol 2	—	—	—	2.1	4	5.7	35	4	5.7	35
Total	53	50	--		4	5.7		-49	-44.5	

Table 7-3: CP-2 Consolidated Storage, Typical Year Overflow Summary 8 Overflows

	Baseline 2015			Consolidated Storage 8 Overflows				Change		
Outfall	# of Events	Volume (MG)	Duration (HR)	Storage Volume (MG)	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)
001A	19	6.3	74	—	0	0.0	0	-19	-6.3	-74
002A	13	0.4	11	—	0	0.0	0	-13	-0.4	-11
Consol 1	—	—	—	1.2	1	0.2	4	1	0.2	4
003A	45	15.3	300	—	0	0.0	0	-45	-15.3	-300
004A	53	24.1	614	—	0	0.0	0	-53	-24.1	-614
005A	25	3.6	121	—	0	0.0	0	-25	-3.6	-121
006A	12	0.5	39	—	0	0.0	0	-12	-0.5	-39
Consol 2	—	—	—	2.1	4	5.7	34	4	5.7	34
Total	53	50	--		4	5.8		-49	-44.4	

Table 7-4: CP-2 Consolidated Storage, Typical Year Overflow Summary 12 Overflows

	Baseline 2015			Consolidated Storage 12 Overflows				Change		
Outfall	# of Events	Volume (MG)	Duration (HR)	Storage Volume (MG)	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)
001A	19	6.3	74	—	0	0.0	0	-19	-6.3	-74
002A	13	0.4	11	—	0	0.0	0	-13	-0.4	-11
Consol 1	—	—	—	1.0	1	0.3	6	1	0.3	6
003A	45	15.3	300	—	0	0.0	0	-45	-15.3	-300
004A	53	24.1	614	—	0	0.0	0	-53	-24.1	-614
005A	25	3.6	121	—	0	0.0	0	-25	-3.6	-121
006A	12	0.5	39	—	0	0.0	0	-12	-0.5	-39
Consol 2	—	—	—	1.6	10	9.4	60	10	9.4	60
Total	53	50	--		10	9.7		-43	-40.6	

Table 7-5: CP-2 Consolidated Storage, Typical Year Overflow Summary 20 Overflows

	Baseline 2015			Consolidated Storage 20 Overflows				Change		
Outfall	# of Events	Volume (MG)	Duration (HR)	Storage Volume (MG)	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)
001A	19	6.3	74	—	0	0.0	0	-19	-6.3	-74
002A	13	0.4	11	—	0	0.0	0	-13	-0.4	-11
Consol 1	—	—	—	0.4	7	1.8	25	7	1.8	25
003A	45	15.3	300	—	0	0.0	0	-45	-15.3	-300
004A	53	24.1	614	—	0	0.0	0	-53	-24.1	-614
005A	25	3.6	121	—	0	0.0	0	-25	-3.6	-121
006A	12	0.5	39	—	0	0.0	0	-12	-0.5	-39
Consol 2	—	—	—	0.8	20	19.7	108	20	19.7	108
Total	53	50	--		20	21.5		-33	-28.7	

Table 7-6: CP-2 Consolidated Storage Overall CSO Volume Reduction Summary

Control Level	Baseline 2015 (MG)	Control Program 2	% Reduction from 2015	% Capture
2015 Baseline	NA	NA	NA	69.5%
0 Overflows	50.3	0.0	100.0%	100.0%
4 Overflows	50.3	5.7	88.6%	96.5%
8 Overflows	50.3	5.8	88.4%	96.5%
12 Overflows	50.3	9.7	80.8%	94.1%
20 Overflows	50.3	21.5	57.1%	86.9%

7.4.2.6 Cost Summary

The Class 5 (+100%, -50%) cost estimates for Control Program 3 are summarized in Table 7-7.

Table 7-7: Control Program 2 - Consolidated Storage Cost Summary

Control Program 2 - End of Pipe Storage (Consolidated Sites)					
Overflows per Year	0	4	8	12	20
Capital Cost ($ Million)	$73.8	$46.6	$45.4	$40.6	$29.1
O&M Cost ($ Million)	$0.7	$0.4	$0.4	$0.4	$0.3
Net Present Worth ($ Million)	$83.9	$53.9	$51.8	$46.6	$34.2

7.4.3 Control Program 3 – Consolidated Tunnel Storage

7.4.3.1 Description

This Control Program calls for a tunnel to follow Industrial Avenue and Bergen Turnpike and for the consolidation of all outfalls into the tunnel for storage. The tunnel will be dewatered into the interceptor and include an overflow to the river. The result will be only one outfall. The available route fixed the tunnel length at 5,900 feet. The tunnel system will consist of:

- Consolidation piping from Outfall 006A
- Diversion piping from each outfall
- Control Gates
- Drop shafts along Industrial Avenue and at intersection of 2nd Avenue, and Bergen Turnpike.
- Deaeration chambers
- A dewatering pumping station
- Grit and screening facilities
- Force main connection back to the BCUA Main Trunk Sewer.
- A tunnel overflow with tide gate

7.4.3.2 Analysis

The anticipated facilities including the diversion, consolidation piping, tunnel, dewatering pumping station and force main were implemented in the InfoWorksICM model. The model configuration is shown in Figure 7-11. Since InfoWorksICM dynamically tracks storage, the tunnel volumes could be modeled explicitly as large diameter pipes rather than as storage nodes. Initial tunnel sizes were developed by adding the required storage volumes for the tanks and then calculating the tunnel diameter to provide the required volume. The tunnel was scaled to provide the targeted number of overflows for each level of control.

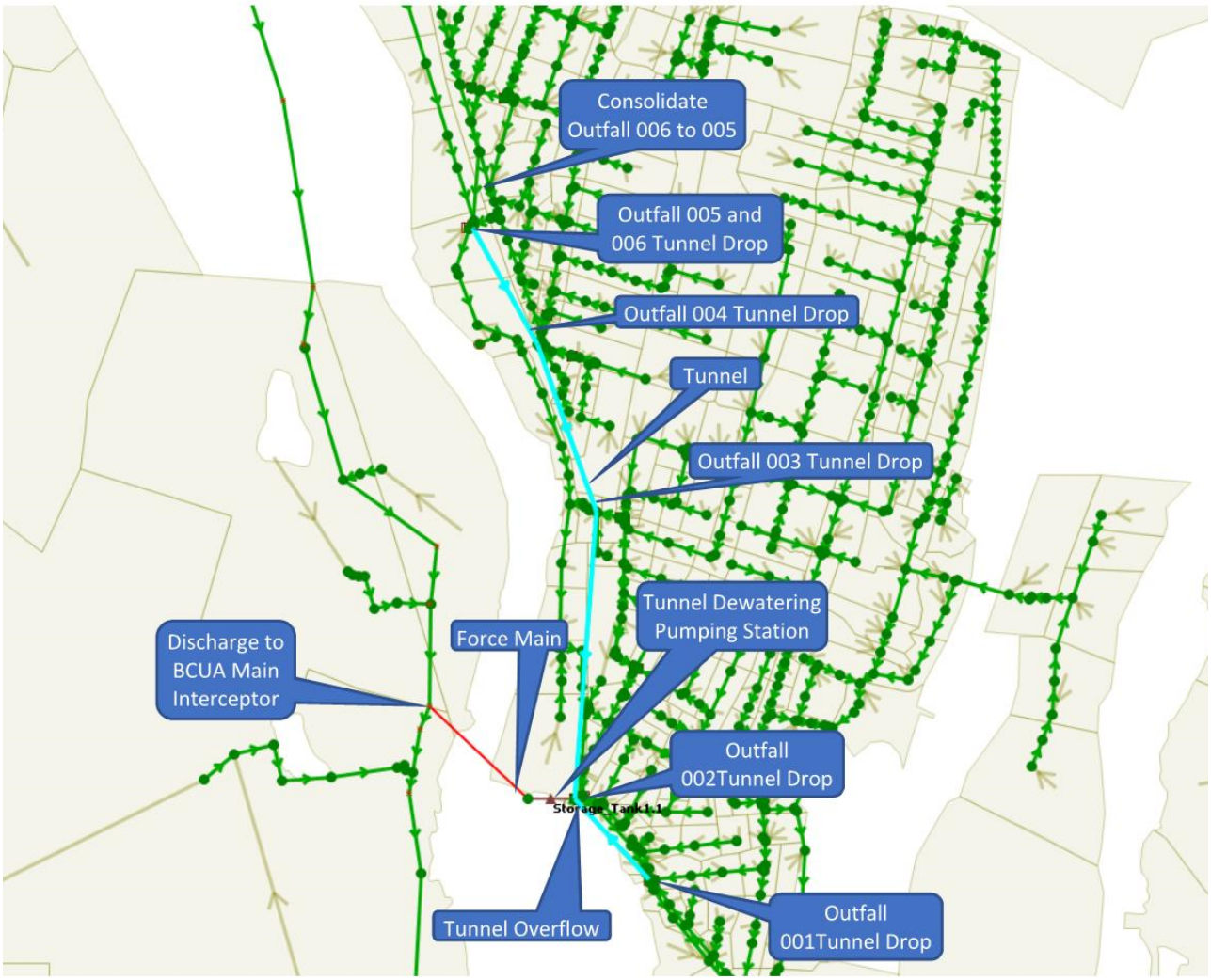


Figure 7-11: Tunnel Storage - Sample Model Configuration

7.4.3.3 Siting Issues

The overflows for the Village of Ridgefield Park are all located along the Hackensack River and Overpeck Creek. Consolidation piping would be needed to bring relatively small flows from Outfall 006A to the area of Regulator 005 where it would be diverted to the tunnel. The general layout of the tunnel would be from Regulator 005 near Industrial Avenue to the intersection of Main Street and Bergen Turnpike and Regulator 002. Accordingly, it is anticipated that a consolidation tunnel would follow the same general route. The BCUA Ridgefield Park Branch Intercepting Sewer has a limited capacity and thus on a conceptual basis it may be better to pump across the Hackensack River and directly into the BCUA Main Trunk Sewer. Accordingly, the conceptual plan calls for a tunnel under the Hackensack River to the BCUA Hackensack interceptor. The pumping station would then be constructed in the vicinity of the intersection. It is noted that the layout and feasibility of tunnels is highly dependent on geotechnical information. The

available soils information indicate that the tunnel may be in soft ground which increases both risk and expense. An illustration of the conceptual tunnel storage facilities is provided in Figure 7-12.

As noted above the conceptual storage tunnel would have two sections to the tunnel. One section would be constructed from the intersection of Industrial Avenue and Bergen Turnpike north to Outfall 005A, and the second section starting in the same location and proceeding east to Outfall 001A. As noted above, the Ridgefield Park Branch Intercepting Sewer has limited flow capacity and thus for a tunnel the point of connection may need to move to the BCUA Trunk Sewer across the Hackensack River. As illustrated in Figure 7-13 it appears that there would be adequate room in the area for a dewatering pumping station and force main to BCUA. This area was not identified in the siting analysis as it does not consist generally of open or unused space. The use of this space would require taking a portion of the industrial property on which it is sited and significant disruption during construction.

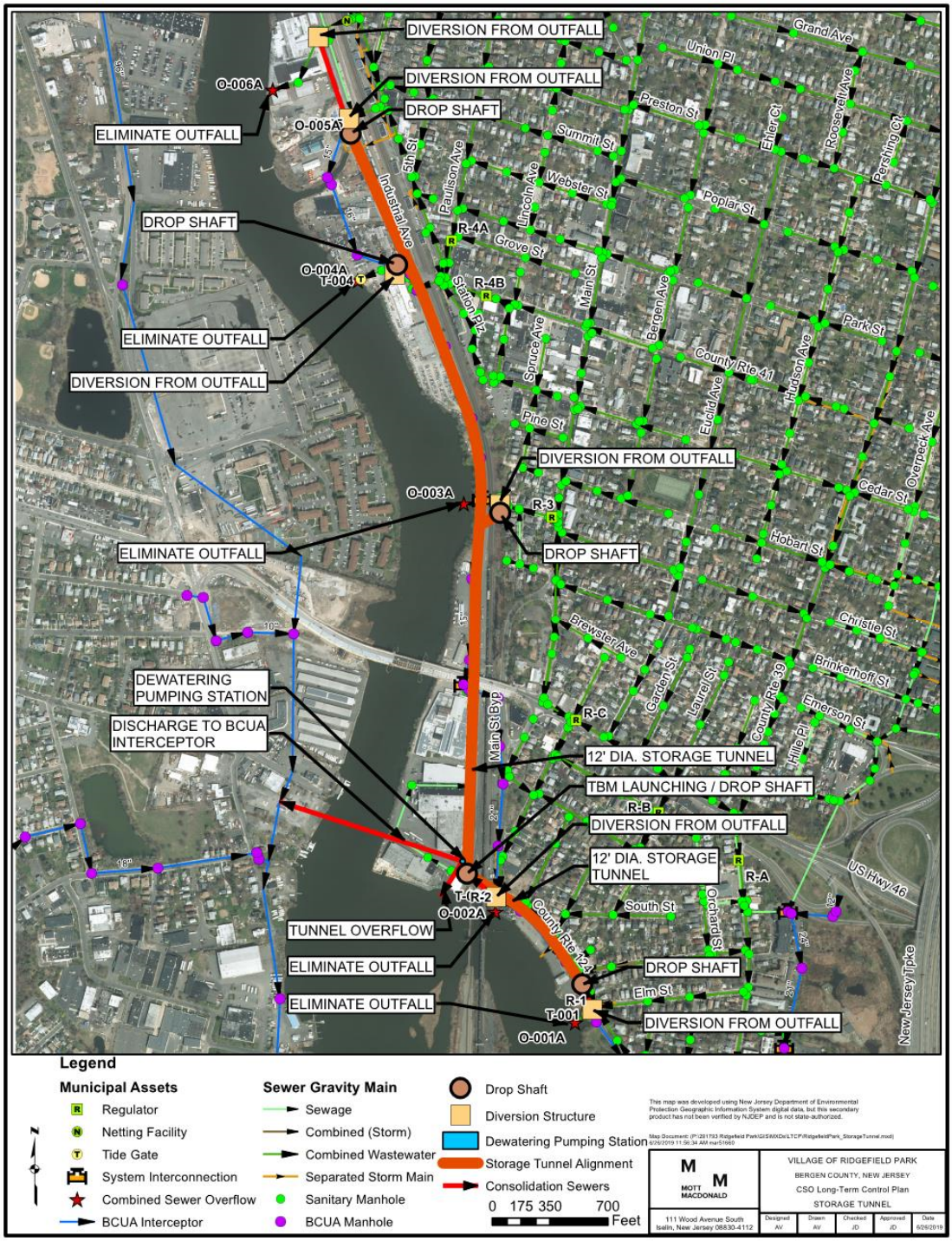


Figure 7-12: Tunnel Storage Conceptual Layout

Figure 7-13: Conceptual Tunnel Siting Restraints

7.4.3.4 Institutional Issues

The institutional issues surrounding Control Program 3 are typical of a large-scale construction project in an urban area. While located in an urban area construction the facilities associated with this control program will require environmental permits. Below is list of anticipated permits required:

- Waterfront Development Permit
- Flood Hazard Area Permit
- SCD Certification
- USACE Nationwide 404 Permit
- Local Permits
- NJDEP Treatment Works Approval
- NJDEP Tidelands
- Permits and coordination with railroads and State DOT

These permits are standard permits and while they must be obtained, they do not appear to have the potential to greatly extend the project schedule or add excessive risk to the project.

7.4.3.5 Implementability

Implementing a tunnel within the confines of an urban area is challenging. Mining and recovery shaft areas are required for this alternative to be feasible, and available area in Ridgefield Park for this purpose is minimal. This alternative also requires area to site a dewatering pumping station and a tunnel overflow, and available area in the Village are limited. While it is possible to control the flow into the tunnel using automated gates and level sensors, the tunnel must still be provided with a relief point. The limited space available and other constraints tend to drive costs higher. Tunnels may also be subject to highly complex hydraulic transients. Typically, these are controlled by limiting the tunnel inflow and preventing the tunnel from filling completely and by providing a tunnel overflow structure to relieve the excess flow.

7.4.3.6 Public Acceptance

The construction required for tunnels is large and invasive making public acceptance of the project a concern. In general, the construction is limited to industrial areas, however the area near Main Street and Bergen Turnpike is a high traffic area and a main artery out of the Village. Most of the facilities will be underground, however, some facilities such as electrical equipment and pumping station controls will be above grade. The impact on existing industrial/commercial establishments during construction will need to be considered and the Village may wish to consider purchasing the land needed for these improvements.

Following construction, tunnels are generally preferable from the stand point of public acceptance since most of the facility is underground. Aboveground features will still be required such air release, electrical facilities, odor control facilities and access points to pumps.

7.4.3.7 Performance

The performance associated with Control Program 3 are summarized below in Table 7-8 through Table 7-13 , which provided performance details for the selected levels of control 0, 4, 8, 12 and 20 overflows during the Typical Year. These results represent the modeled performance of the control program when implemented in the 2050 baseline model as compared to the 2015 baseline modeling.

Table 7-8: CP-3 Tunnel Storage, Typical Year Overflow Summary 0 Overflows

	Baseline 2015			5,900 LF Tunnel Storage 0 Overflows				Change		
Outfall	# of Events	Volume (MG)	Duration (HR)	Tunnel Diamater (ft)	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)
001A	19	6.3	74	-	0	0.0	0	-19	-6.3	-74
002A	13	0.4	11	-	0	0.0	0	-13	-0.4	-11
003A	45	15.3	300	-	0	0.0	0	-45	-15.3	-300
004A	53	24.1	614	-	0	0.0	0	-53	-24.1	-614
005A	25	3.6	121	-	0	0.0	0	-25	-3.6	-121
006A	12	0.5	39	-	0	0	0	-12	-0.5	-39
Tunnel	—	—	—	12	0	0.0	0	0	0.0	0
Total	53	50.3	--		0	0.0		-53	-50.3	

Table 7-9: CP-3 Tunnel Storage, Typical Year Overflow Summary 4 Overflows

	Baseline 2015			5,900 LF Tunnel Storage 4 Overflows				Change		
Outfall	# of Events	Volume (MG)	Duration (HR)	Tunnel Diamater (ft)	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)
001A	19	6.3	74	-	0	0.0	0	-19	-6.3	-74
002A	13	0.4	11	-	0	0.0	0	-13	-0.4	-11
003A	45	15.3	300	-	0	0.0	0	-45	-15.3	-300
004A	53	24.1	614	-	0	0.0	0	-53	-24.1	-614
005A	25	3.6	121	-	0	0.0	0	-25	-3.6	-121
006A	12	0.5	39	-	0	0	0	-12	-0.5	-39
Tunnel	—	—	—	8	4	4.7	12	4	4.7	12
Total	53	50.3	--		4	4.7		-49	-45.6	

Table 7-10: CP-3 Tunnel Storage, Typical Year Overflow Summary 8 Overflows

	Baseline 2015			5,900 LF Tunnel Storage 4 Overflows				Change		
Outfall	# of Events	Volume (MG)	Duration (HR)	Tunnel Diamater (ft)	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)
001A	19	6.3	74	-	0	0.0	0	-19	-6.3	-74
002A	13	0.4	11	-	0	0.0	0	-13	-0.4	-11
003A	45	15.3	300	-	0	0.0	0	-45	-15.3	-300
004A	53	24.1	614	-	0	0.0	0	-53	-24.1	-614
005A	25	3.6	121	-	0	0.0	0	-25	-3.6	-121
006A	12	0.5	39	-	0	0	1	-12	-0.5	-38
Tunnel	—	—	—	8	4	4.7	12	4	4.7	12
Total	53	50.3	--		4	4.7		-49	-45.6	

Table 7-11: CP-3 Tunnel Storage, Typical Year Overflow Summary 12 Overflows

	Baseline 2015			5,900 LF Tunnel Storage 12 Overflows				Change		
Outfall	# of Events	Volume (MG)	Duration (HR)	Tunnel Diamater (ft)	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)
001A	19	6.3	74	-	0	0.0	0	-19	-6.3	-74
002A	13	0.4	11	-	0	0.0	0	-13	-0.4	-11
003A	45	15.3	300	-	0	0.0	0	-45	-15.3	-300
004A	53	24.1	614	-	0	0.0	0	-53	-24.1	-614
005A	25	3.6	121	-	0	0.0	0	-25	-3.6	-121
006A	12	0.5	39	-	0	0		-12	-0.5	-39
Tunnel	—	—	—	7	7	7.9	22	7	7.9	22
Total	53	50.3	--		7	7.9		-46	-42.4	

Table 7-12: CP-3 Tunnel Storage, Typical Year Overflow Summary 20 Overflows

	Baseline 2015			5,900 LF Tunnel Storage 20 Overflows				Change		
Outfall	# of Events	Volume (MG)	Duration (HR)	Tunnel Diamater (ft)	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)
001A	19	6.3	74	-	0	0.0	0	-19	-6.3	-74
002A	13	0.4	11	-	0	0.0	0	-13	-0.4	-11
003A	45	15.3	300	-	0	0.0	0	-45	-15.3	-300
004A	53	24.1	614	-	0	0.0	0	-53	-24.1	-614
005A	25	3.6	121	-	0	0.0	0	-25	-3.6	-121
006A	12	0.5	39	-	0	0	0	-12	-0.5	-39
Tunnel	—	—	—	6	10	11.4	29	10	11.4	29
Total	53	50.3	--		10	11.4		-43	-38.8	

Table 7-13: CP-3 Tunnel Storage Overall CSO Volume Reduction Summary

Control Level	Baseline 2015 (MG)	Control Program 3	% Reduction from 2015	% Capture
2015 Baseline	NA	NA	NA	69.5%
0 Overflows	50.3	0.0	100.0%	100.0%
4 Overflows	50.3	4.7	90.7%	97.2%
8 Overflows	50.3	4.7	90.7%	97.2%
12 Overflows	50.3	7.9	84.3%	95.2%
20 Overflows	50.3	11.4	77.2%	93.1%

7.4.3.8 Cost Summary

The Class 5 (+100%, -50%) cost estimates for Control Program 3 are summarized in Table 7-14.

Table 7-14: Control Program 3 - Tunnel Summary Costs

Control Program 3 - Tunnel					
Overflows per Year	0	4	8	12	20
Capital Cost ($ Million)	$88.4	$72.3	$72.3	$67.3	$62.3
O&M Cost ($ Million)	$2.0	$1.7	$1.7	$1.7	$1.6
Net Present Worth ($ Million)	$118.5	$98.6	$98.6	$92.5	$86.3

7.4.4 Control Program 4 – Consolidated End of Pipe Treatment

7.4.4.1 Description

This control program consists of siting a treatment facility on consolidation piping from 001A and 002A and 003A-006A. According to the national CSO Policy, overflows that meet the minimum required treatment are no longer considered overflows. Thus, by providing a treatment train capable of providing disinfection and the accompanying solids removals (primary treatment), the number of overflows can be reduced by removing all overflows that discharge at flow rates less than the treatment provided. Partial treatment would be provided for storms whose peak flow exceeds to treatment rate.

For purposes of evaluation the following treatment train was established:

1. Divert flows downstream of the regulator, and if possible downstream of the existing netting facility.
2. Provide fine screening (removal of solids >0.5 inch) of the flows to remove additional floatables and course particles.
3. Provide interim pumping to offset the head loss associated with the treatment processes.
4. Provide high rate primary treatment of the flows to remove solids in advance of disinfection. For evaluation purposes, ActiFlo was used as a representative technology given it widespread use.
5. Disinfection is then provided by peracetic acid, by providing a six-minute contact time.
6. The flow is then discharged through a new or possibly modified outfall.

The evaluation of practicality and feasibility draws on the siting analysis to identify locations for each facility and drives the consolidation of select facilities. Figure 7-15 through Figure 7-17 below depict conceptual site layouts size to limit the Typical Year number of overflows to 4 events. Facilities were sized for the range of control levels 0, 4, 8, 12 and 20 overflows.

Consolidation offers some advantages over siting individual facilities at each outfall:

- The result will be only two consolidated outfalls. This will simplify future permitting and effectively eliminate four outfalls.
- This control program will result in fewer facilities for the Village to maintain.
- For the most part it makes use of public rights-of-way and land that will be under the control of the town.

There are also some potential disadvantages:

- There will be more disturbance to local streets from consolidation piping.
- There will be additional costs associated with the consolidation piping.

- The larger above ground facility would have a greater impact, possibly reducing the usable area available for the park at Outfall 001A. The benefit would be reduced impacts on the rest of the Village.
- The larger above ground facility would have a greater impact Outfalls 003A through 006A, possibly requiring taking and demolishing the entire auto recycling facility. The benefit would be reduced impacts on the rest of the Village.

7.4.4.2 Analysis

The sizing for treatment facilities are often the same to achieve 4, 8 and 12 overflows, and sizing is difficult to combine with storage-based control programs. This is because peak flows are generally driven by the peak rainfall intensity coinciding with the time of concentration of the basin, whereas the total overflow volume is driven by the total rainfall depth for the event. As such, the sizing of the end of pipe treatment facilities are driven by a different set of storms than end of pipe storage facilities. Achieving a consistent level of control for peak flows, requires a much higher level of control to be achieved through end of pipe treatment. This can be seen in the output in Table 7-15 through Table 7-20.

An analysis was performed on the flows entering the consolidation piping generated in the InfoWorksICM model. The evaluation consisted of peak flow rate of overflows diverted to the treatment facility, and the corresponding treatment. Once the treatment facility has exceeded the peak treatment rate, remaining flows are tracked as overflows.

As sample treatment hydrograph can be seen below in Figure 7-14. Overflows beyond the treatment capacity were tracked and a list of storms identified as producing overflows was generated. The list of remaining overflows generated was compared to the allowable list, and the treatment capacity increased until the remaining storms consisted only of those allowable on the systemwide basis.

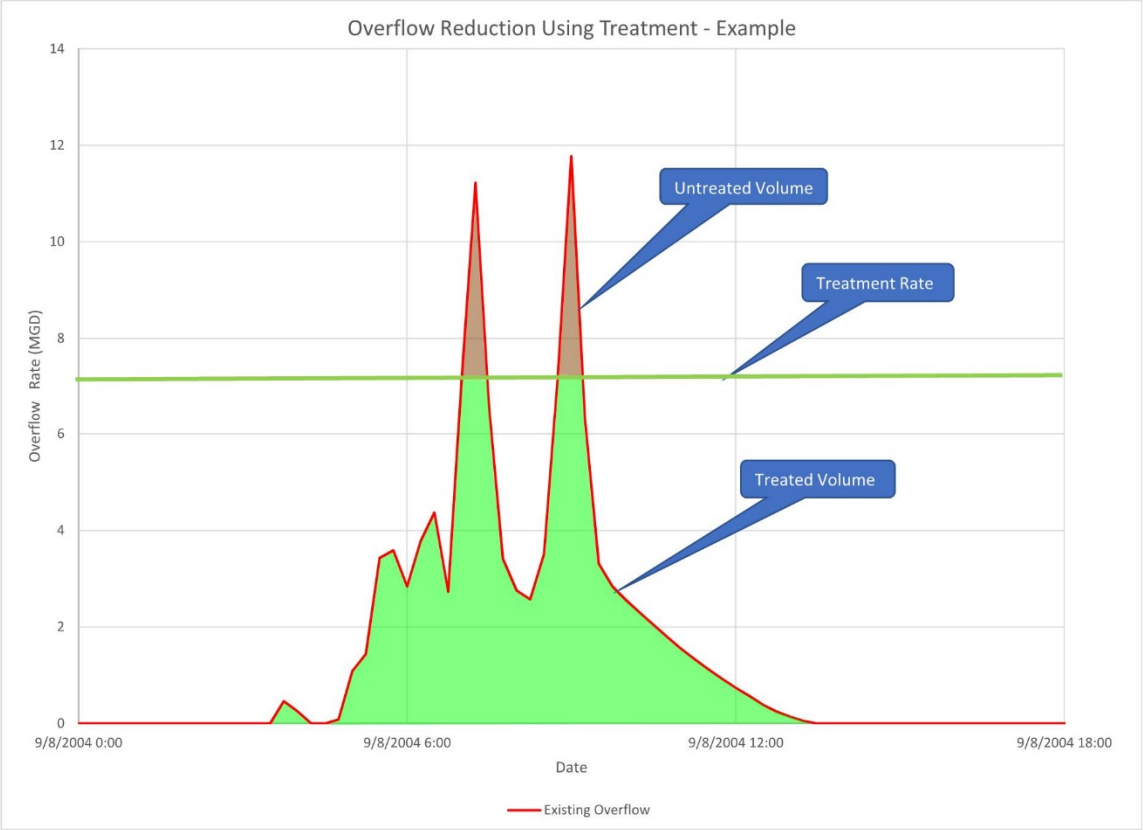


Figure 7-14: Sample Treatment Hydrograph

It is anticipated that two separate end-of-pipe treatment facilities will be required for Ridgefield Park. One located at the property along Bergen Turnpike (001A and 002A), and the other at the end of Mount Vernon (003A, 004A, 005A, and 006A). The consolidated end-of-pipe treatment was implemented in the model by sizing pipes ranging from 3 feet to 6 feet in diameter to convey the modeled runoff from each of the respective outfalls to the local consolidated facility tank site. The consolidation piping capacity was evaluated for the Typical Year and it was sized such than there would be no adverse impacts (water surface increases) to the upstream system. The treatment rate was established by analyzing the flow in the consolidation piping. The peak flow rate for each storm was listed and sorted highest to lowest. The systemwide storms corresponding to the level of control were placed next to this list and the treatment rate was set at the flow rate of the highest storm that was not allowed to overflow.

7.4.4.3 Siting Issues

The property previously discussed for consolidated offline storage facilities is the same property that could be used for consolidated treatment of the CSO discharges. As illustrated in Figure 7-15, the conceptual plan would be to divert wet weather flows from Outfall 002A to a new 27” consolidation sewer that would junction with a new 48” consolidation sewer from Outfall 001A, which would then transfer the combined flow to the new treatment facility. Since this property is currently owned or is being acquired by the Village

land availability should not be an issue. Nevertheless, the area across the street is residential and having an above the ground treatment in the area will need special treatment to blend in with the community.

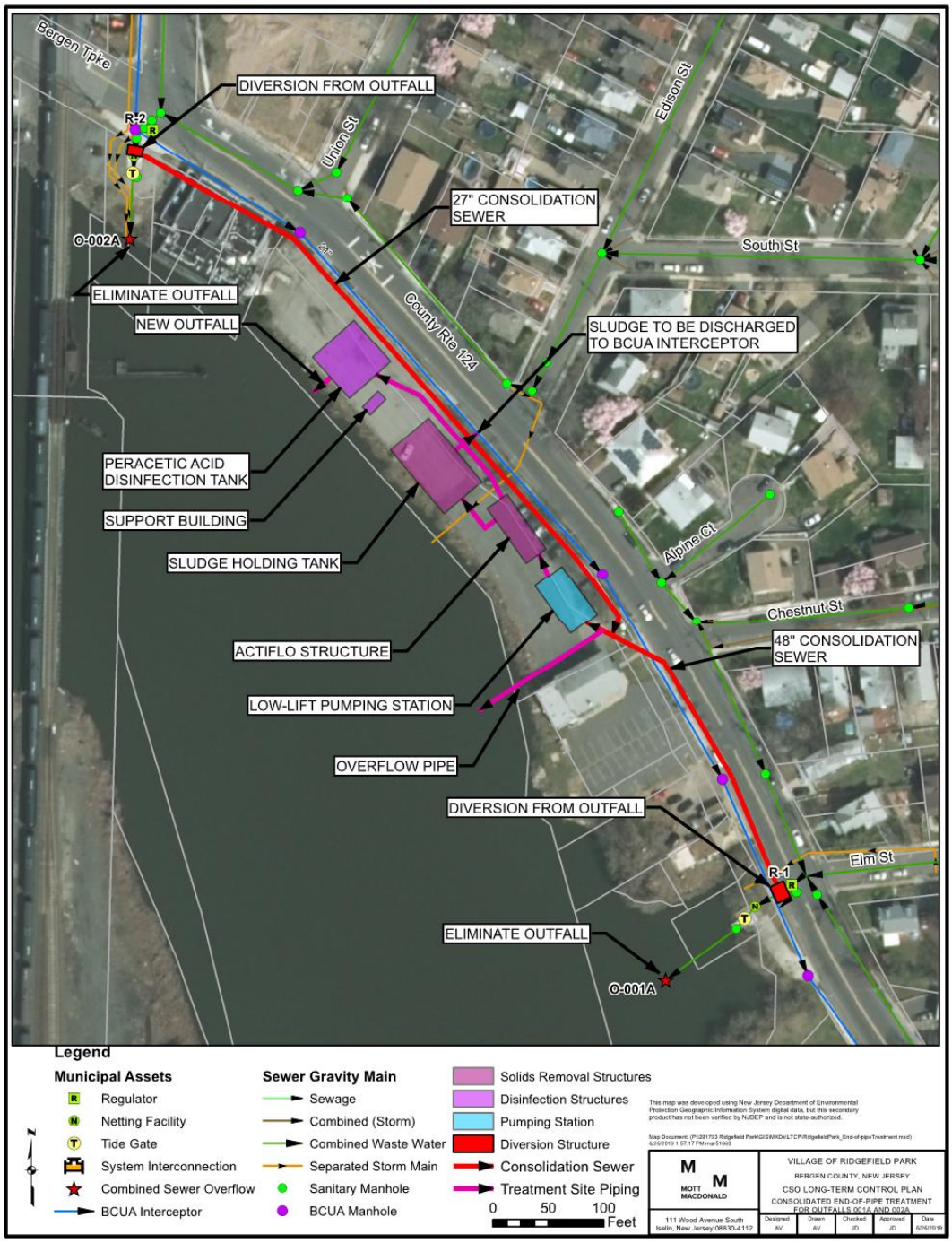


Figure 7-15: Consolidated Treatment for Outfalls 001A and 002A

The consolidated treatment for the other outfalls within the Village could be sited at the end of Mount Vernon Street, as illustrated in Figure 7-16 (Outfalls 003A, 004A, and 005A) and Figure 7-17 (Outfalls 003A, 004A, 005A and 006A), if additional investigations indicate that 006A should not be eliminated as previously proposed.

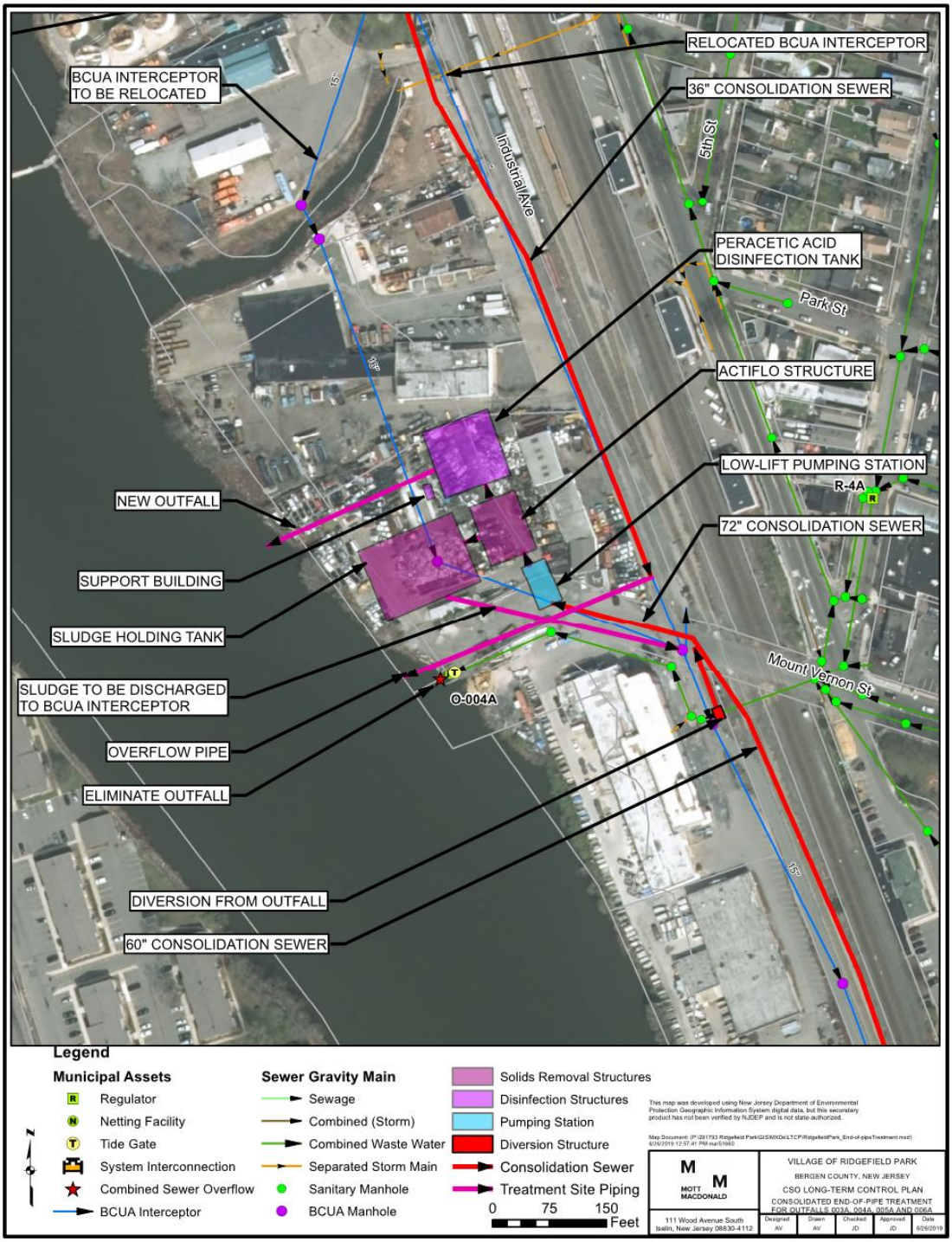


Figure 7-16: Consolidate Treatment for Outfalls 003A, 004A and 005A.

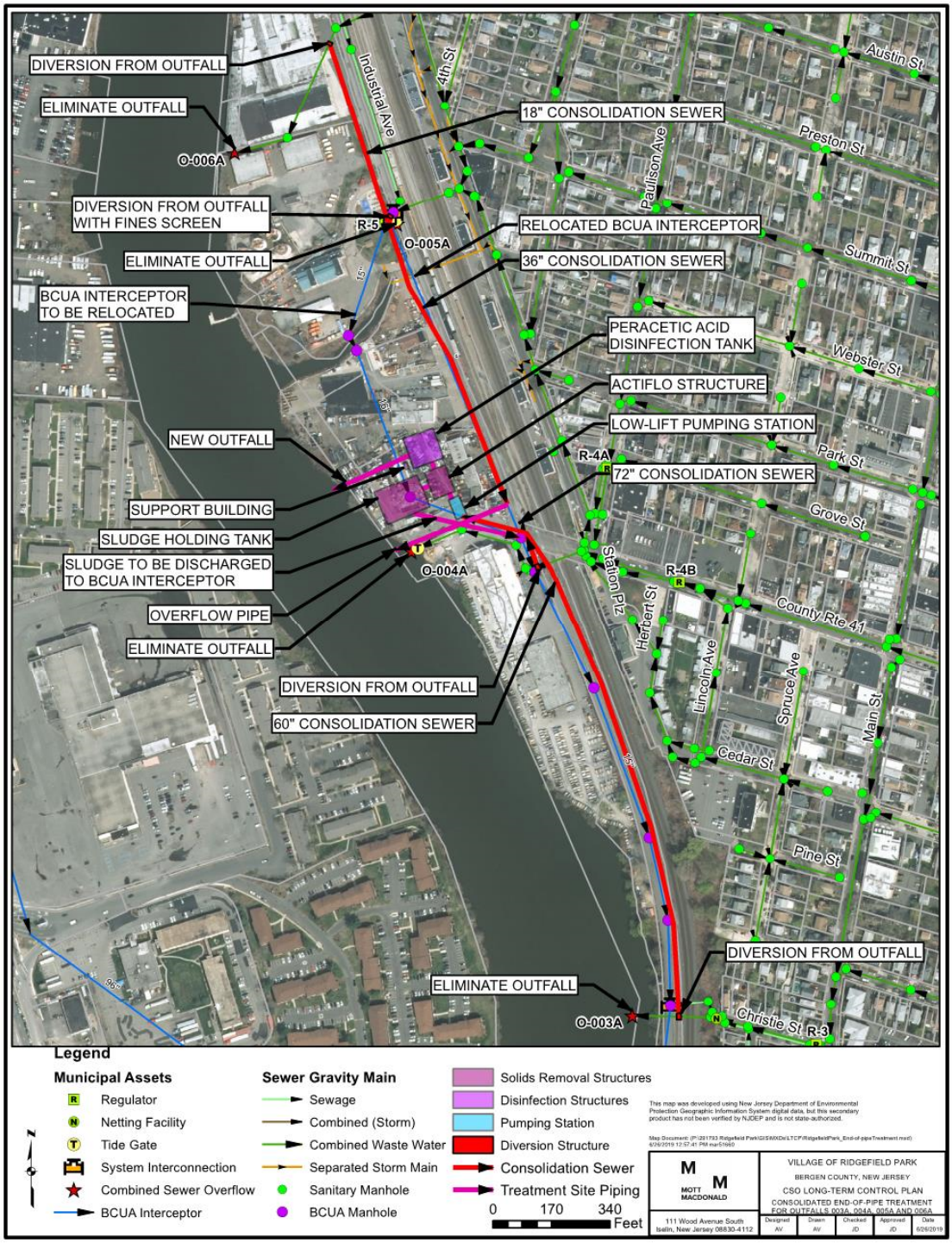


Figure 7-17: Consolidate Treatment for Outfalls 003A, 004A, 005A, and 006A

7.4.4.4 Institutional Issues

The institutional issues surrounding Control Program 5 are typical of a large-scale construction project in an urban area. While located in an urban area construction the facilities associated with this control program will require environmental permits. Below is list of anticipated permits required:

- Waterfront Development Permit
- Flood Hazard Area Permit
- SCD Certification
- USACE Nationwide 404 Permit
- NJDEP Tidelands Program
- Local Permits
- NJDEP Treatment Works Approval
- Highway and Railroad coordination and approvals for consolidation piping.

These permits are standard permits and while they must be obtained, they do not appear to have the potential to greatly extend the project schedule or add excessive risk to the project.

7.4.4.5 Implementability

Installation of end-of-pipe treatment facilities in urban areas can be challenging due to space and access limitations. Unlike end-of-pipe storage tanks, end-of-pipe treatment facilities are generally above-grade, or at grade. If located above grade, extensive excavation is not required thus reducing cost as well as complexity of the excavation near the receiving waters. Depth to groundwater is also not a consideration, thus, the possibility of a floating subsurface structure is not of concern. Flow would be pumped into the treatment units, but possibly could be discharged by gravity. Above-grade facilities however would be susceptible to possible damage from freezing. Accordingly, these facilities may need to be housed in a structure. There is little available information on the soil conditions at the sites, however, given the proximity to the floodplain, soil conditions may be poor, and the facilities may need to be situated on piles.

As noted above the proposed treatment facilities for Outfalls 001A and 002A is across the street from a residential area and freezing is a concern. Thus, it is anticipated that the facilities in this location would need to be primarily constructed below grade. Installation of treatment tankage in urban areas can be challenging. The tanks have been sized assuming an 8-foot tank depth. The intent is to fill them with pumps, but to have them discharge by gravity. It is estimated that a total excavated depth of 12 feet is generally required. Excavating to this depth requires costly dewatering and support of the excavation, which is made more challenging by proximity to the receiving waters. In addition, utilities impacted by construction must be relocated, protected, or supported, including the BCUA's Ridgefield Park Branch Interceptor, which in this vicinity is in the sidewalk area of Bergen Turnpike.

The treatment facilities to be sited at Outfalls 003A through 006A are in proximity to existing buildings, however it is anticipated that these building may need to be demolished to make room for the proposed units. All the issues raised for the Bergen Turnpike treatment facility apply with the exception that this facility is not located near residential housing. Control of groundwater may be a significant challenge. As previously noted, the tanks are near the Hackensack River, which creates additional issues and risks with keeping the site dewatered during construction.

There is little available information on the soil characteristics at the treatment sites, however, given the proximity of the sites to the floodplain, soil conditions could be poor, and the treatment units/tankage may need to be situated on piles. Piles may also be required to anchor the units, so they do not become buoyant in the event of a flood, or periods of high groundwater. Tidal flooding is a concern because high storm surge levels could produce inundation with little rainfall meaning the tank could be empty and prone to floatation. In addition, installing the large diameter consolidation piping within the Industrial Avenue could be challenging. There are numerous other utilities in the street including an existing stormwater outfall that must be crossed and the BCUA Branch Interceptor which must be avoided, or relocated.

7.4.4.6 Public Acceptance

The construction required for an end-of-pipe facility is large and invasive, making public acceptance of the project a concern. Because the facilities proposed are generally above or at grade, they have the potential to produce visual impacts, odors, and noise, making them more difficult to site in residential and commercial areas. There may be concerns with odors at the proposed site on Bergen Turnpike due to proximity to commercial and residential areas.

Following construction, end-of-pipe treatment facilities are less preferable than tanks due to the permanent visibility of the structure. They also use land area that could otherwise be utilized by the community for other purposes. The consolidated sites are located on industrial parcels of land slated for redevelopment, and the construction may be more acceptable in terms of public acceptance than other sites.

7.4.4.7 Performance

The performance associated with Control Program 4 is summarized below in Table 7-15 through Table 7-20, which provide performance details for the selected levels of control 0, 4, 8, 12 and 20 overflows during the Typical Year. These results represent the modeled performance of the control program when implemented in the 2050 baseline model as compared to the 2015 baseline modeling.

Per the National CSO Policy, discharges receiving the minimum required treatment are not considered overflows. Accordingly, to align with the systemwide levels of control (0, 4, 8, 12 and 20 overflows) treatment rates were set to treat all events smaller than those allowed to discharge under the various levels of control. While the outfalls will continue to discharge many times a year, the flows will not be considered overflow unless they exceed the treatment rate. This may create some confusion for the public who may observe discharges and not be certain if the flow is treated or not. Additional indicators, such as warning lights which would flash when full treatment is not being provided may be required. The performance of consolidated treatment is summarized in Table 7-15 through Table 7-19, which present the results for the equivalent treatment for 0, 4, 8, 12, and 20 overflows per year.

Table 7-15: CP-4 Consolidated Treatment, Typical Year Overflow Summary 0 Overflows

	Baseline 2015			Consolidated Treatment 0 Overflows				Change		
Outfall	# of Events	Volume (MG)	Duration (HR)	Treatment Rate	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)
001A	19	6.3	74	—	0	0.0	0	-19	-6.3	-74
002A	13	0.4	11	—	0	0.0	0	-13	-0.4	-11
Consol 1				28.0	0	0.0	0	0	0.0	0
003A	45	15.3	300	—	0	0.0	0	-45	-15.3	-300
004A	53	24.1	614	—	0	0.0	0	-53	-24.1	-614
005A	25	3.6	121	—	0	0.0	0	-25	-3.6	-121
006A	12	0.5	39	—	0	0.0	0	-12	-0.5	-39
Consol 2	—	—	—	94.6	0	0.0	0	0	0.0	0
Total	53	50	--		0	0.0		-53	-50.3	

Table 7-16: CP-4 Consolidated Treatment, Typical Year Overflow Summary 4 Overflows

	Baseline 2015			Consolidated Treatment 4 Overflows				Change		
Outfall	# of Events	Volume (MG)	Duration (HR)	Treatment Rate	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)
001A	19	6.3	74	—	0	0.0	0	-19	-6.3	-74
002A	13	0.4	11	—	0	0.0	0	-13	-0.4	-11
Consol 1				21.5	1	0.0	0	1	0.0	0
003A	45	15.3	300	—	0	0.0	0	-45	-15.3	-300
004A	53	24.1	614	—	0	0.0	0	-53	-24.1	-614
005A	25	3.6	121	—	0	0.0	0	-25	-3.6	-121
006A	12	0.5	39	—	0	0.0	0	-12	-0.5	-39
Consol 2	—	—	—	71.9	1	0.2	0	1	0.2	0
Total	53	50	--		1	0.2		-52	-50.0	

Table 7-17: CP-4 Consolidated Treatment, Typical Year Overflow Summary 8 Overflows

	Baseline 2015			Consolidated Treatment 8 Overflows				Change		
Outfall	# of Events	Volume (MG)	Duration (HR)	Treatment Rate	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)
001A	19	6.3	74	—	0	0.0	0	-19	-6.3	-74
002A	13	0.4	11	—	0	0.0	0	-13	-0.4	-11
Consol 1				21.5	1	0.0	0	1	0.0	0
003A	45	15.3	300	—	0	0.0	0	-45	-15.3	-300
004A	53	24.1	614	—	0	0.0	0	-53	-24.1	-614
005A	25	3.6	121	—	0	0.0	0	-25	-3.6	-121
006A	12	0.5	39	—	0	0.0	0	-12	-0.5	-39
Consol 2	—	—	—	71.9	1	0.2	0	1	0.2	0
Total	53	50	--		1	0.2		-52	-50.0	

Table 7-18: CP-4 Consolidated Treatment, Typical Year Overflow Summary 12 Overflows

	Baseline 2015			Consolidated Treatment 12 Overflows				Change		
Outfall	# of Events	Volume (MG)	Duration (HR)	Treatment Rate	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)
001A	19	6.3	74	—	0	0.0	0	-19	-6.3	-74
002A	13	0.4	11	—	0	0.0	0	-13	-0.4	-11
Consol 1				21.5	1	0.0	0	1	0.0	0
003A	45	15.3	300	—	0	0.0	0	-45	-15.3	-300
004A	53	24.1	614	—	0	0.0	0	-53	-24.1	-614
005A	25	3.6	121	—	0	0.0	0	-25	-3.6	-121
006A	12	0.5	39	—	0	0.0	0	-12	-0.5	-39
Consol 2	—	—	—	71.4	2	0.2	1	2	0.2	1
Total	53	50	--		2	0.2		-51	-50.0	

Table 7-19: CP-4 Consolidated Treatment, Typical Year Overflow Summary 20 Overflows

Outfall	Baseline 2015			Consolidated Treatment 20 Overflows				Change		
	# of Events	Volume (MG)	Duration (HR)	Treatment Rate	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)
001A	19	6.3	74	—	0	0.0	0	-19	-6.3	-74
002A	13	0.4	11	—	0	0.0	0	-13	-0.4	-11
Consol 1				12.3	9	0.3	2	9	0.3	2
003A	45	15.3	300	—	0	0.0	0	-45	-15.3	-300
004A	53	24.1	614	—	0	0.0	0	-53	-24.1	-614
005A	25	3.6	121	—	0	0.0	0	-25	-3.6	-121
006A	12	0.5	39	—	0	0.0	0	-12	-0.5	-39
Consol 2	—	—	—	35.0	10	2.7	4	10	2.7	4
Total	53	50	--		10	3.0		-43	-47.2	

Table 7-20: CP-4 Consolidated Treatment Overall CSO Volume Reduction Summary

Control Level	Baseline 2015 (MG)	Control Program 4	% Reduction from 2015	% Capture
2015 Baseline	NA	NA	NA	69.5%
0 Overflows	50.3	0.0	100.0%	100.0%
4 Overflows	50.3	0.2	99.6%	99.9%
8 Overflows	50.3	0.2	99.6%	99.9%
12 Overflows	50.3	0.2	99.5%	99.9%
20 Overflows	50.3	3.0	93.9%	98.2%

7.4.4.8 Cost Summary

The Class 5 (+100%, -50%) cost estimates for Control Program 4 are summarized in Table 7-21.

Table 7-21: Control Program 4 - Consolidated Treatment Summary Costs

Control Program 4 - End of Pipe Treatment (Consolidated Sites)					
Overflows per Year	0	4	8	12	20
Capital Cost ($ Million)	$75.2	$65.8	$65.8	$65.5	$49.7
O&M Cost ($ Million)	$0.8	$0.7	$0.7	$0.7	$0.6
Net Present Worth ($ Million)	$87.3	$77.0	$77.0	$76.7	$59.5

7.4.5 Control Program 5 – Sewer Separation

7.4.5.1 Description

This control program constitutes constructing a new sanitary sewer system and converting the existing combined sewer into a storm sewer. This would effectively remove the Village of Ridgefield Park from being a CSO community.

The benefits of this alternative include:

- Work remains in public right-of-way, no new land required,

- Opportunity for system renewal, reconstruction, and
- Elimination of outfalls

The challenges include:

- Highly disruptive to roads and traffic,
- Need to redirect every sanitary service connection on each street, and
- Possible stormwater controls and treatment in the future.

7.4.5.2 Analysis

The system was modeled in InfoWorksICM, by converting the combined sub-catchments into sanitary sub-catchments.

7.4.5.3 Siting Issues

New sanitary or storm sewers would be constructed within the existing right-of-way, however utility conflicts may be significant especially within those areas that already have multiple sewers within the roadways.

7.4.5.4 Institutional Issues

The institutional issues surrounding sewer separation are typical of a large-scale construction project that takes place within existing roadways in an urban area. While located in an urban area construction the facilities associated with this control program will require environmental permits. Below is list of anticipated permits required:

- Local Permits
- Stormwater Management
- NJDEP Tideland Program
- USACE Nationwide 404 Permit
- NJDEP Treatment Works Approval

These permits are standard permits and while they must be obtained, they do not appear to have the potential to greatly extend the project schedule or add excessive risk to the project.

In addition, it is noted that separating out stormwater flow may not be an effective long-term solution. This is because stormwater contributes to pollution of the receiving waters, and as such will eventually need to be treated or controlled. Under the NJDEP's current enforcement practices TSS removal would be required for the separate stormwater outfalls. Recently proposed stormwater regulations include increased treatment requirements for creating separately sewered areas that could greatly increase the costs and impacts of performing separation.

7.4.5.5 Implementability

In terms of land acquisition, this alternative ranks high, since the proposed work would mostly be completed within the existing right-of-way. However, installation of separate sewers in urban areas can be challenging due to traffic impacts, utility conflicts, and space limitations. Such an undertaking will result in road closures across the Village and resulting traffic redirection over the course of construction. Assuming this alternative will be implemented over the course of 30 years, this means that about 16 acres would need to be addressed each year. Installation of a new sanitary connections to each residence and business will also be a very extensive undertaking.

7.4.5.6 Public Acceptance

The construction required for sewer separation is large and invasive, making public acceptance of the project a significant concern. Installation of a new sanitary sewer system and connections will result in road closures and resulting impacts on traffic as well as access to local business and institutions during construction, which will not be received favorably by residents in this urban area. This also a very costly alternative, as such may not be preferred.

Following construction, sewer separation might be preferable from the stand point of public acceptance since the resulting facilities would be underground.

7.4.5.7 Performance

It should be noted that modeling of the Village of Ridgefield Park as a separated sewer system without stormwater controls indicates that the total pollutant load to the receiving waters, with the except of pathogens, would increase some 40%. The reason for this is that combined sewers do not discharge during low intensity or volume storms, wherein stormwater discharge whenever it rains. Even during certain events the discharge from combined sewer is first captured and transferred to the WRRF for treatment and not continuous during the rainfall event. Separation of the sewers within the Village of Ridgefield Park will not eliminate most of the pollutant loads, and stormwater controls are not effective against all pollutant types.

Table 7-22: CP-5 Sewer Separation, Typical Year Overflow Summary 0 Overflows

Outfall	Baseline 2015			Baseline 2050			Change		
	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)
001A	19	6.3	74	0	0.0	0	-19	-6.3	-74
002A	13	0.4	11	0	0.0	0	-13	-0.4	-11
003A	45	15.3	300	0	0.0	0	-45	-15.3	-300
004A	53	24.1	614	0	0.0	0	-53	-24.1	-614
005A	25	3.6	121	0	0.0	0	-25	-3.6	-121
006A	12	0.5	39	0	0.0	0	-12	-0.5	-39
Total	53	50.3	--	0	0.0		-53	-50.3	

7.4.5.8 Cost Summary

The Class 5 (+100%, -50%) cost estimate for Control Program 5 is summarized in Table 7-23.

Table 7-23: Control Program 5 – Sewer Separation Summary Costs

Control Program 5 - Sewer Separation					
Overflows per Year	0	4	8	12	20
Capital Cost ($ Million)	$192.7	NA	NA	NA	NA
O&M Cost ($ Million)	$0.0	NA	NA	NA	NA
Net Present Worth ($ Million)	$192.7	NA	NA	NA	NA

7.4.6 Control Program 6 – Green Infrastructure

7.4.6.1 Description

This control program consists in installing green infrastructure to provide storage or detention to contribute to meeting the overflow requirements. Green infrastructure (GI) refers to practices which reduce stormwater volume or flow rate by allowing the stormwater to infiltrate, be stored, or be treated by vegetation or soils. As mentioned previously, bioswales have been selected as the representative type of GI to evaluate for the purposes for model calculations, while the anticipated green infrastructure is expected to consist primarily of bioswales and permeable pavement. If this alternative is selected for inclusion in the LTCP, further refining of types and specific locations of GI will be determined in future planning stages.

7.4.6.2 Analysis

For purposes of evaluation, directing 2.5%, 5%, 7.5% and 10% of the impervious area within the combined sewer area to green stormwater infrastructure was evaluated. However, evaluating fixed amounts of impervious to green stormwater infrastructure ignores whether such an approach is practical or technically feasible. Using the guidance documents previous discussed, an attempt was made to determine the maximum amount of impervious area that could be directed from impervious areas to green infrastructure. It is noted that experience from New York City has shown that the clear majority of sites identified through a desktop GIS study are deemed unsuitable once field investigations and geotechnical (infiltration) testing are conducted. An analysis conducted of sites in one basin showed that of the sites identified at the planning level, only 17% were found suitable to proceed to construction. As previously noted, the available data on soils and groundwater levels in Ridgefield Park indicate that ground conditions are likely not conducive to infiltrating green stormwater infrastructure, thus bioswales were assumed to be non-infiltrating and equipped with a sub-drain to drain back into the collection system.

Suitability of a site for green infrastructure was determined at a high-level based on desktop studies of land use (Figure 7-18), areas of impervious cover (Figure 7-19), Hydrologic Soil Group (HSG) (Figure 7-20), and publicly owned land (Figure 4-2). It is noted that the HSG shown on the mapping is the dominant component in the soil group and that the detail description indicates both A and D HSG soils are present.

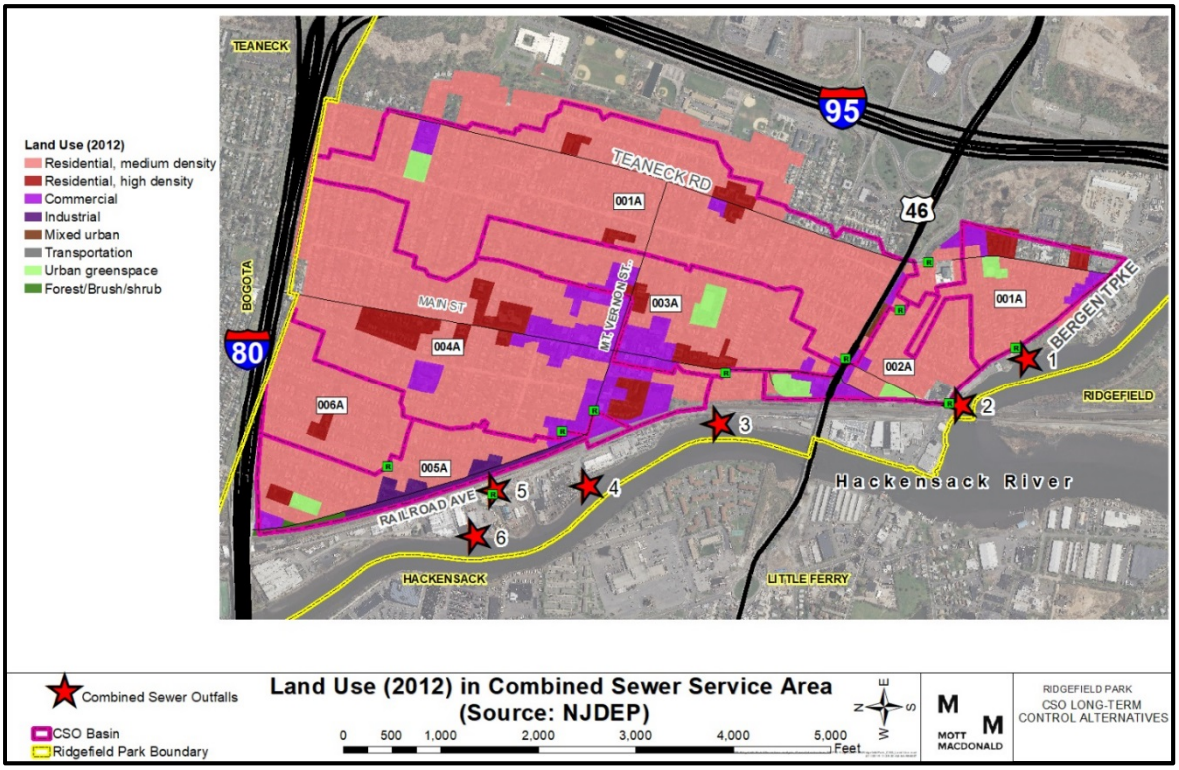


Figure 7-18: Ridgefield Park Land Use Map

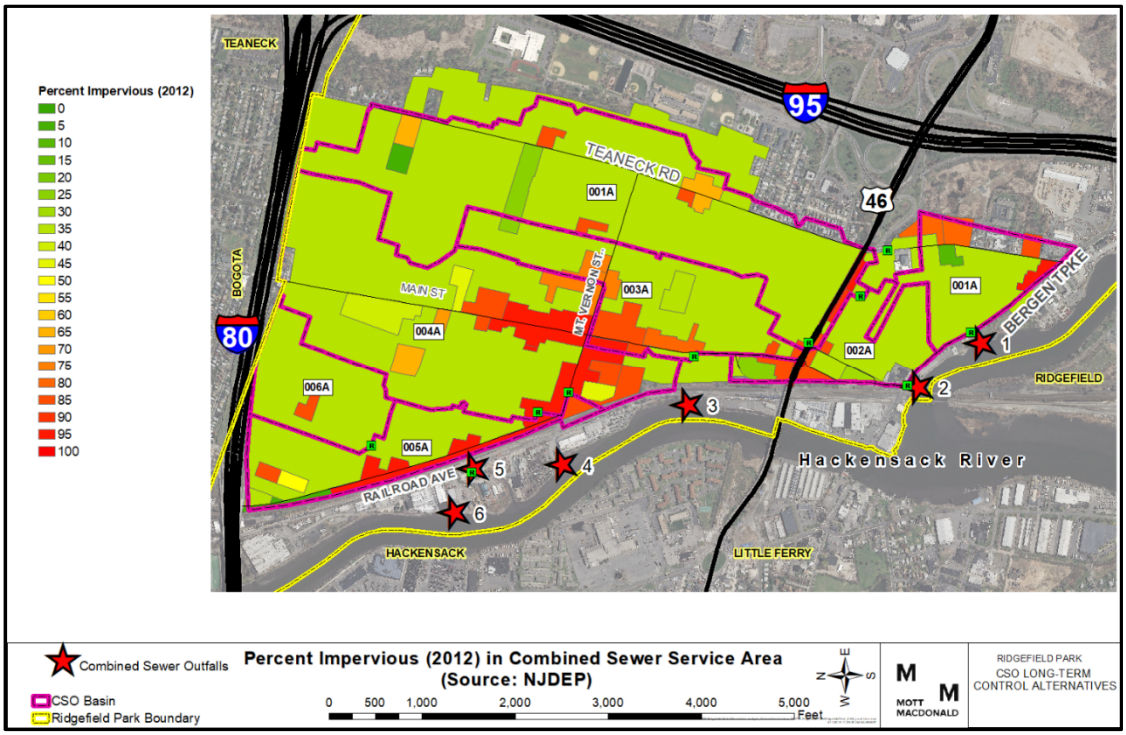


Figure 7-19: Ridgefield Park Impervious Cover Map

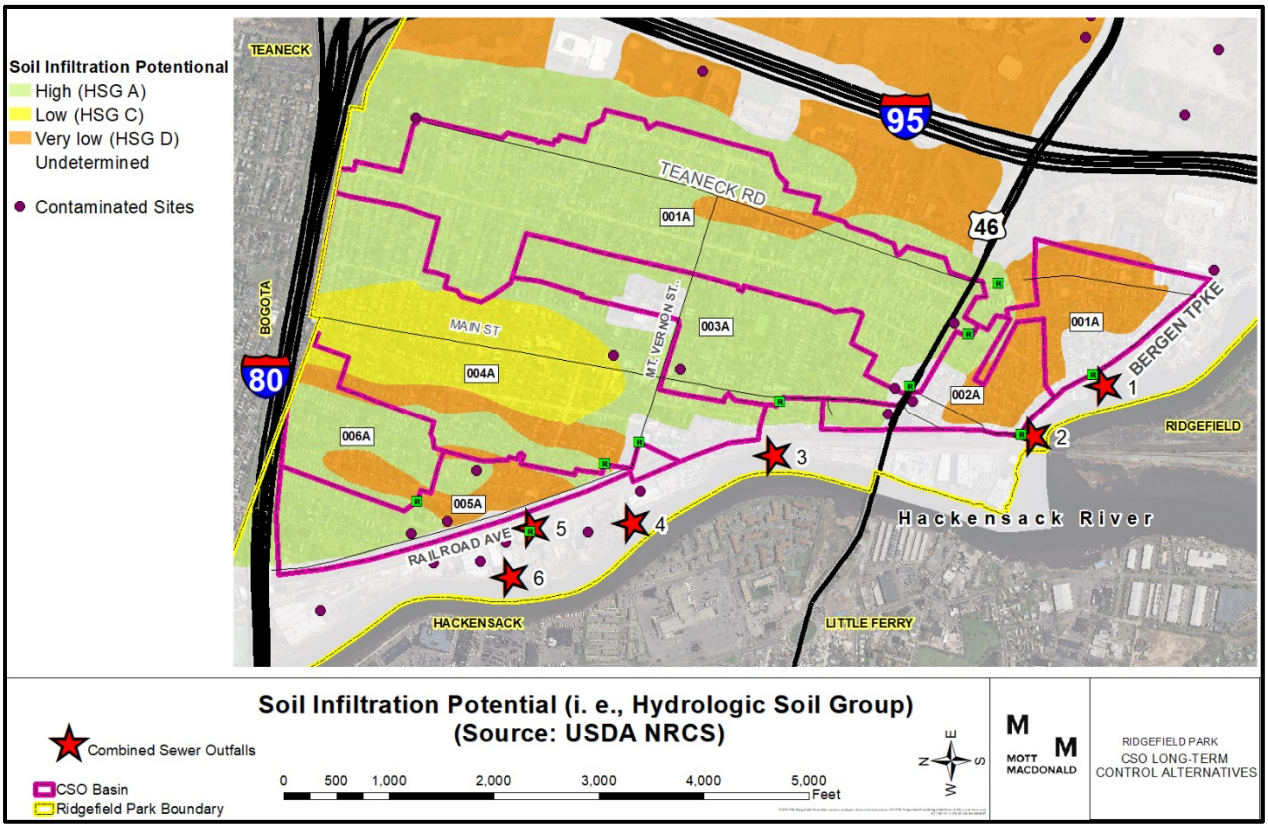


Figure 7-20: Ridgefield Park HSG Map

As was shown in Section 4.3, the public right-of-way offers the best opportunity for green stormwater infrastructure. Accordingly, a typical street segment within the city was examined to estimate the potential for implementing green stormwater infrastructure. It is noted that much of the curb space is consumed with driveway entrances and walkways to houses with limited grass areas between the sidewalk and street. Many of the available areas between the sidewalk and street are also occupied by mature trees, which typically are not removed to install green stormwater infrastructure. Accordingly, it was assumed that only one bioswale could be installed per each side of the street segment (see Figure 7-21) so that a typical street segment would have two bioswale one on each side, and a typical block would have one on each side or four per block. The typical bioswale is 20'x3' and using a 15:1 loading ratio it would treat 900 sf of impervious area. Through GIS analysis it was determined the Village has approximately 287 street segments which results in 574 bioswales. Conservatively, applying a planning level installation rate of 20% (versus 17% from New York City) results in 115 bioswales with a treatment area of 103,500 sf or 2.4 acres of impervious area treated.

The other remaining green stormwater infrastructure practice is permeable pavement. The recommended practice is to apply the permeable paving to parking lanes. Again, referring to a typical street segment which is approximately 310 feet long. It is assumed that the last 50 feet at either end of the block would be reserved for turning lanes resulting in 210 linear feet of parking area available for permeable pavement on either side of the street. The parking lane is assumed to be 6 feet wide for a total area of 2,520 sf per block. Given the groundwater and soil condition it was assumed that only 10% of the Village is suitable for installation of permeable pavement resulting in 252 sf per street segment. This results in a maximum of

72,300 sf of permeable paving in the Village. Applying the recommended loading ratio of 4:1 result in a total of 294,800 sf or 6.6 acres of impervious area treated.

Figure 7-21: Typical street segment with green stormwater infrastructure

When combined bioswales and permeable paving a total of 9.0 acres of impervious are out of a total of 205 acres of impervious area in the existing combined area, or a maximum of 4% of the total impervious area or could be directed to green stormwater infrastructure.

Bioswales were modelled in the InfoWorksICM as a representative 20'x3' unit with and 18" soil depth and 3.5' storage layer. To account for limited soil information, an infiltration rate of 0.25 in/hr was assigned to the underlying soils. This was input in the InfoWorksICM SUDS module (Figure 7-22) to create a typical green infrastructure unit to evaluate the impact that green infrastructure would have on the frequency and volume of CSO events. It can be seen from the representative figure (Figure 7-23) below that GI has a very minimal impact on both peak flow and volume mitigation. As such, it is understood that a high level of

proliferation of GI is required to provide a significant improvement in CSO reduction and volume mitigation.

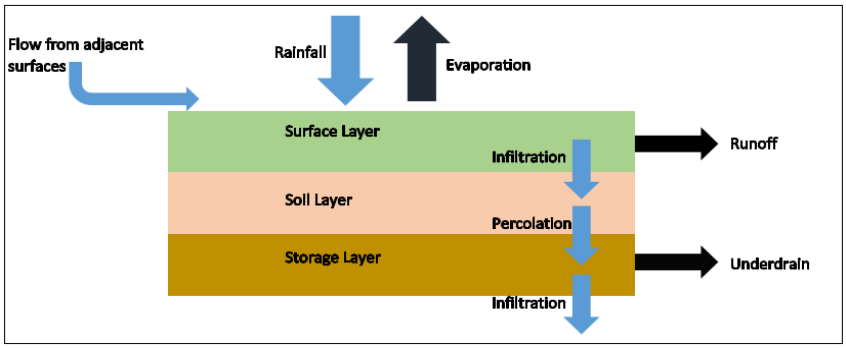


Figure 7-22: InfoWorks SUDS diagram

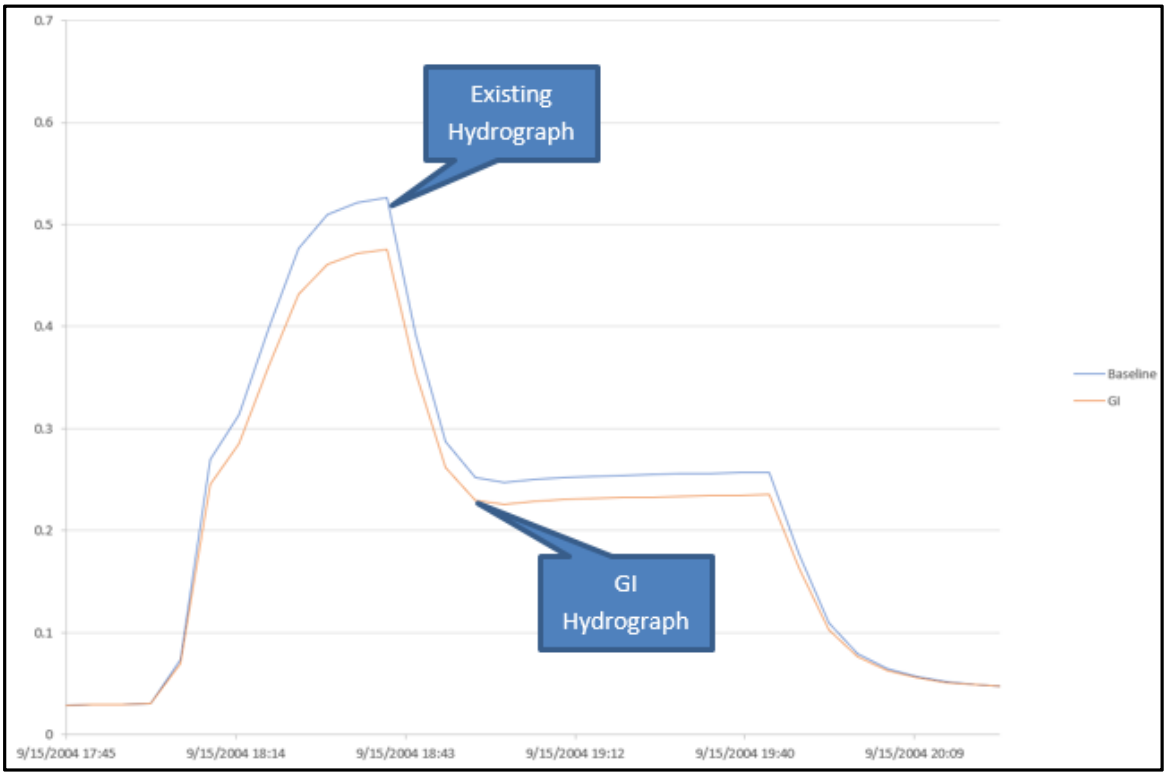


Figure 7-23: Representative green infrastructure hydrograph

7.4.6.3 Siting Issues

Siting issues are anticipated to be minimal as the GI program would be sited primariliy in the public right-of-way. There will be issues with existing driveways, walkway, trees and utilities for the bioswales. The permeable pavement will face challenges with existing utilities, particularly supporting the utilities during construction when significant depths of the road subgrade will be replaced with the reservoir course. As noted, experience has shown that there is a very high attrition rate from the planning stage to actual

construction and as the desired number of GI installations goes up siting issues play a greater role in the attrition rate.

7.4.6.4 Institutional Issues

Typically, the institutional issues associated with green stormwater infrastructure are minimal. Their construction would generally fall within the overall goals of the Village's planning by providing additional green space. Permit requirements would be minimal and may include the following based on the location of the green stormwater infrastructure.

- Waterfront Development Permit if located in the waterfront zone
- Local Permits, likely minimal requirements since project will be conducted by the Village
- SCD Certification

Additional permits and coordination may be required if green stormwater infrastructure is implemented on State or County property.

7.4.6.5 Implementability

From a land acquisition standpoint, green infrastructure would rate highly for implementability. The intent is to site the green stormwater infrastructure in the public right-of-way which is owned by the Village. Accordingly, no land acquisition would be required. However, there are other implementability challenges associated with green stormwater infrastructure to be considered. As has been experienced by other entities such as New York City, there are myriad of field conditions that can prevent construction of green stormwater infrastructure on a site identified through a desktop study, including soil conditions, utility locations, and proximity to trees, building entrances, or bus stops. New York City implements a multi-layered planning approach consisting of desktop studies, field visits, utility mark outs and infiltration testing, and at each phase, several potential sites are eliminated due to factors not identified in the desktop study. It is assumed a similar multi-layered approach will be implemented with similar rates of attrition. This high level of attrition has been reflected in the estimate of green stormwater infrastructure proposed, to realistically reflect this implementability challenge.

7.4.6.6 Public Acceptance

It is generally assumed that public acceptance of green stormwater infrastructure will be high since it serves as an amenity to the community. This is likely true for implementation of bioswales as they provide additional green space and the construction footprint is relatively small. The implementation of permeable pavement on which the green infrastructure alternative may be less accepted by the public as the construction is more invasive. However, upon completion of the project the area will closely resemble the existing condition. Accordingly, the likelihood of public acceptance for green stormwater infrastructure should be considered high.

7.4.6.7 Performance

The performance associated with Control Program 6 are summarized below in Table 7-24 through Table 7-28, which provided performance details for the selected levels of control 0, 4, 8, 12 and 20 overflows during the Typical Year. These results represent the modeled performance of the control program when implemented in the 2050 baseline model as compared to the 2015 baseline modeling.

Table 7-24: CP-6 GI, Typical Year Overflow Summary 2.5% Impervious to GI

Outfall	Baseline 2015			Green Infrastructure 2.5%			Change		
	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)
001A	19	6.3	74	19	6.3	74	0	0.0	0
002A	13	0.4	11	13	0.4	11	0	0.0	0
003A	45	15.3	300	45	15.1	298	0	-0.2	-2
004A	53	24.1	614	53	24.0	605	0	-0.1	-9
005A	25	3.6	121	25	3.6	121	0	0.0	0
006A	12	0.5	39	12	0.5	39	0	0.0	0
Total	53	50.3	--	53	49.9		0	-0.3	

Table 7-25: CP-6 GI, Typical Year Overflow Summary 5% Impervious to GI

Outfall	Baseline 2015			Green Infrastructure 5%			Change		
	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)
001A	19	6.3	74	19	6.3	74	0	0.0	0
002A	13	0.4	11	13	0.4	11	0	0.0	0
003A	45	15.3	300	43	15.0	294	-2	-0.4	-5
004A	53	24.1	614	53	23.7	589	0	-0.4	-25
005A	25	3.6	121	25	3.5	118	0	0.0	-2
006A	12	0.5	39	12	0.5	39	0	0.0	0
Total	53	50.3	--	53	49.4		0	-0.8	

Table 7-26: CP-6 GI, Typical Year Overflow Summary 7.5% Impervious to GI

Outfall	Baseline 2015			Green Infrastructure 7.5%			Change		
	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)
001A	19	6.3	74	19	6.3	74	0	-0.1	-1
002A	13	0.4	11	13	0.4	11	0	0.0	0
003A	45	15.3	300	42	14.7	291	-3	-0.6	-9
004A	53	24.1	614	53	23.4	582	0	-0.7	-33
005A	25	3.6	121	25	3.5	118	0	-0.1	-3
006A	12	0.5	39	12	0.5	39	0	0.0	0
Total	53	50.3	--	53	48.9		0	-1.4	

Table 7-27: CP-6 GI, Typical Year Overflow Summary 10% Impervious to GI

Outfall	Baseline 2015			Green Infrastructure 10%			Change		
	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)	# of Events	Volume (MG)	Duration (HR)
001A	19	6.3	74	19	6.2	74	0	-0.1	-1
002A	13	0.4	11	13	0.4	11	0	0.0	0
003A	45	15.3	300	40	14.5	272	-5	-0.8	-28
004A	53	24.1	614	53	23.1	580	0	-1.0	-34
005A	25	3.6	121	25	3.5	114	0	-0.1	-6
006A	12	0.5	39	12	0.5	39	0	0.0	0
Total	53	50.3	--	53	48.3		0	-2.0	

Table 7-28: Control Program 6 – GI Summary Costs

% Impervious Treated by GI	Baseline 2015 (MG)	Control Program 6	% Reduction from 2015	% Capture
2015 Baseline	NA	NA	NA	69.5%
2.5% Impervious	50.3	49.9	0.6%	69.7%
5% Impervious	50.3	49.4	1.7%	70.0%
7.5% Impervious	50.3	48.9	2.8%	70.3%
10% Impervious	50.3	48.3	3.9%	70.7%

7.4.6.8 Cost Summary

The Class 5 (+100%, -50%) cost estimates for Control Program 5 are summarized in Table 7-29.

Table 7-29: Control Program 6 – GI Summary Costs

Control Program 6 - Green Infrastructure				
% of Impervious Area Managed	2.5%	5%	7.50%	10%
Capital Cost ($ Million)	$2.4	$5.0	$7.6	$10.0
O&M Cost ($ Million)	$0.0	$0.1	$0.1	$0.1
Net Present Worth ($ Million)	$2.7	$5.8	$8.8	$11.6

7.4.7 Summary of Control Program Cost Opinions

The Class 5 (+100%, -50%) 20-year net present worth cost opinions for the various alternatives are summarized below in Table 7-30.

Table 7-30: 20-Year Net Present Worth for All Control Programs

Control Program	NPW Summary - Overflows per Year ($M)				
Level of Control	0	4	8	12	20
1) Eliminate Outfall 006	NA	NA	NA	NA	NA
2) Storage (Consolidated)	$84	$54	$52	$47	$34
3) Tunnel	$118	$99	$99	$92	$86
4) Treatment (Consolidated)	$87	$77	$77	$77	$60
5) Sewer Separation	$193	NA	NA	NA	NA
	NPW Summary - % of Impervious Area Managed ($M)				
	2.50%	5%	7.50%	10%	
6) Green Infrastructure	$2.7	$6	$9	$12	

7.5 Preliminary Selection of Alternative

7.5.1 Evaluation Factors

The Control Programs were evaluated on several factors that include:

- Cost – Costs were normalized by $/gal of annual CSO reduction based on the Typical Year and level of control corresponding to 4 overflows and 5% of directly connected impervious areas being directed to green stormwater infrastructure. Cost is a primary driving factor and was assigned a weighting of 25% of the overall score. The following ratings were assigned based on the normalized cost.
    - 5: $0-$1.00 per gallon of CSO removed
    - 4: $1.00-$2.00 per gallon of CSO removed
    - 3: $2.00-$3.00 per gallon of CSO removed
    - 2: $3.00-$4.00 per gallon of CSO removed
    - 1: over $4.00 per gallon of CSO removed
- CSO Reduction – Since the outfalls in Ridgefield Park all discharge to the Hackensack River and adjacent portion of the Overpeck Creek along a relatively short reach, it is appropriate to consider the overall reduction of CSO volume achieved by the control alternatives during the Typical Year. CSO reduction was considered a key factor and was assigned a weighting of 15%. The following ratings were applied to the CSO volume reductions:
    - 5: over 40 MG of CSO volume reduction in the Typical Year
    - 4: 30 MG - 45 MG of CSO volume reduction in the Typical Year
    - 3: 20 MG - 30 MG of CSO volume reduction in the Typical Year
    - 2: 10 MG - 20 MG of CSO volume reduction in the Typical Year
    - 1: under 10 MG of CSO volume reduction in the Typical Year
- CSO Frequency – The frequency of overflow during the Typical Year is an important metric both in regard to regulatory compliance under the Presumptive Approach and in terms of public

acceptance. Since overflow frequency is closely related to overflow volume it is assigned a weighting of 15%. The following ratings were applied to the CSO volume reductions:

- 5: 4 or fewer overflows during the Typical Year
- 4: 5 to 8 overflows during the Typical Year
- 3: 9 to 12 overflows during the Typical Year
- 2: 13 to 20 overflows during the Typical Year
- 1: over 20 overflows during the Typical Year

- Institutional Issues (Permitting) – Institutional issues particularly permitting can have a significant impact on a project particularly the schedule of design which can then delay the commencement of construction. If institutional issues cannot be overcome, the project may need to be redesigned potentially affecting not just the schedule, but the cost. Experience has shown for important projects just as CSO LTCP institutional issues can generally be overcome due to the overall need for the project. Accordingly, institutional issues are assigned a weighting of 15%. The following ratings were assigned to institutional issues:
  - 5: high possibility to delay project by more than six months and impact budget by 10% or more.
  - 4: medium possibility to delay project more than six months and impact budget by more than 5%.
  - 3: medium possibility to delay project less than six months and impact budget by 5%.
  - 2: small possibility of delay in schedule less than six months.
  - 1: Unlikely to impact schedule or budget.
- Implementability – High level planning studies such as a LTCP must formulate plans based on incomplete information. Unexpected factors such as poor soil condition and conflicts with unknown existing infrastructure can impact a project's schedule and budget. Accordingly, implementability was assigned a weighting of 15%. The following ratings were assigned to implementability:
  - 5: high possibility to delay project by more than six months and impact budget by 10% or more.
  - 4: medium possibility to delay project more than six months and impact budget by more than 5%.
  - 3: medium possibility to delay project less than six months and impact budget by 5%.
  - 2: small possibility of delay in schedule less than six months.
  - 1: Unlikely to impact schedule or budget.
- Public Acceptance – Public acceptance of an alternative is largely based on experience which guides anticipated public reaction. These responses can change as demographic and economic changes occur as well as overall societal trends towards the environmental develop. Public acceptance is an important criterion, but ultimately the Villages obligations are driven by the permit requirements, accordingly, public acceptance is assigned a weighting of 15%. The following ratings were applied to the anticipated public acceptance.

- 5: Public would welcome and support proposed plan.
- 4: Public would accept proposed plan, but no provide external support.
- 3: Public objects to proposed plan but takes minimal action.
- 2: Public objects to proposed plan, and actively opposes.
- 1: Strong public opposition, including legal challenges

Each of the six control programs was rated as per the above criteria. To provide a more concise comparison each control program was rated for the level of control corresponding to four overflows in the Typical Year. For Control Program 6, green stormwater infrastructure, the results for 5% of modeled directly connected impervious were presented, which is closest to the estimated maximum amount of green infrastructure that can be formally attributed to the LTCP. It is noted that green infrastructure does not achieve the desired level of control in terms of volume reduction or reduction in CSO frequency. However, it does provide a volume reduction and it is anticipated that if included in the LTCP it would additive to other control programs. The results of the rating process are summarized in Table 7-31.

Table 7-31: Summary Rating of Control Programs

Control Program	Cost	CSO Volume Reduction	CSO Frequency Reduction	Institutional Issues	Implement-ability	Public Acceptance	Weighted Score
1. Eliminate CSO-006A	NA	NA	NA	NA	NA	NA	NA
2. Consolidated Tank Storage	4	5	5	4	3	3	4.0
3. Tunnel	3	5	5	4	2	2	3.5
4. Consoldiated End of Pipe Treatment	4	5	5	2	3	2	3.6
5. Sewer Separation	2	5	5	3	2	2	3.1
6. Green Infrastructure	1	1	1	5	4	5	2.7
Weighting	25%	15%	15%	15%	15%	15%	100%

7.5.2 Regulatory Compliance

Four of the six control plans formulated were sized to provide regulatory compliance with the Presumptive Approach 4 overflows requirement. As discussed, the performance of the Ridgefield Park facilities was coordinated with the rest of the BCUA CSO Group's communities.

8 Summary

The decision to select alternatives will take place during the next phase of the permit from July 1, 2019 to June 1, 2020. The selected plan may include one of the Control Programs evaluated, it may consist of a combination of Control Programs or include items not discussed in this report. The selection will not be just the outcome of an engineering evaluation but may be influenced by the community's ability to afford the alternative, political considerations, environmental justice, public acceptance, the community's long-term planning and policy decision relating to potential future CSO permitting actions. While no decisions are being made at this time, the overall ratings in Table 7-31, indicate that in general options that include consolidation may be preferable to options that address each outfall individually and that tank storage options may be preferable to tunnel storage or end of pipe treatment option.

The remaining CSO overflow volumes are summarized in Table 8-1, the reduction in CSO volume for each control program is summarized in Table 8-2, the frequency of overflows is summarized in Table 8-3, the % Capture is summarized in Table 8-4 and the net present worth costs normalized by gallon of CSO reduction during the Typical Year are summarized in Table 8-5.

Table 8-1: Summary of CSO Volumes for Typical Year

Control Program	2015 Baseline (MG)	Level of Control - Overflows during Typical Year (MG)				
		0	4	8	12	20
1. Eliminate CSO-006A	50.3	NA	NA	NA	NA	NA
2. Consolidated Tank Storage	50.3	0.0	5.7	5.8	9.7	21.5
3. Tunnel	50.3	0.0	4.7	4.7	7.9	11.4
4. Consoldiated End of Pipe Treatment	50.3	0.0	0.2	0.2	0.2	3.0
5. Sewer Separation	50.3	0.0	NA	NA	NA	NA
% Impervious to GI		2.5%	5%	7.5%	10%	
6. Green Infrastructure	50.3	49.9	49.4	48.9	48.3	

Table 8-2: Summary of CSO Volume Reductions for Typical Year

Control Program	2015 Baseline (MG)	Level of Control - Overflows during Typical Year (MG)				
		0	4	8	12	20
1. Eliminate CSO-006A	50.3	NA	NA	NA	NA	NA
2. Consolidated Tank Storage	50.3	50.3	44.5	44.4	40.6	28.7
3. Tunnel	50.3	50.3	45.6	45.6	42.4	38.8
4. Consoldiated End of Pipe Treatment	50.3	50.3	50.0	50.0	50.0	47.2
5. Sewer Separation	50.3	50.3	NA	NA	NA	NA
% Impervious to GI		2.5%	5%	7.5%	10%	
6. Green Infrastructure	50.3	0.3	0.8	1.4	2.0	

Table 8-3: Summary of Frequency of Overflows for Typical Year

Control Program	2015 Baseline	Level of Control - Overflows during Typical Year				
		0	4	8	12	20
1. Eliminate CSO-006A	53	NA	NA	NA	NA	NA
2. Consolidated Tank Storage	53	0	4	4	10	20
3. Tunnel	53	0	4	4	7	10
4. Consoldiated End of Pipe Treatment	53	0	1	1	2	10
5. Sewer Separation	53	0	NA	NA	NA	NA
% Impervious to GI		2.5%	5%	7.5%	10%	
6. Green Infrastructure	53	53	53	53	53	

Table 8-4: Summary of Percent Capture Achieved by Each Control Program

Control Program	2015 Baseline	Level of Control - Overflows during Typical Year				
		0	4	8	12	20
1. Eliminate CSO-006A	69.5%	NA	NA	NA	NA	NA
2. Consolidated Tank Storage	69.5%	100.0%	96.5%	96.5%	94.1%	86.9%
3. Tunnel	69.5%	100.0%	97.2%	97.2%	95.2%	93.1%
4. Consoldiated End of Pipe Treatment	69.5%	100.0%	99.9%	99.9%	99.9%	98.2%
5. Sewer Separation	69.5%	100.0%	NA	NA	NA	NA
% Impervious to GI		2.5%	5%	7.5%	10%	
6. Green Infrastructure	69.5%	69.7%	70.0%	70.3%	70.7%	

Table 8-5: Net Present Worth Costs, Normalized by Gallon of CSO Reduction

Control Program	Cost per Gallon Volume CSO Reduction ($/gal)				
Level of Control	0	4	8	12	20
1) Eliminate Outfall 006	NA	NA	NA	NA	NA
2) Storage (Consolidated)	$1.7	$1.2	$1.2	$1.1	$1.2
3) Tunnel	$2.4	$2.2	$2.2	$2.2	$2.2
4) Treatment (Consolidated)	$1.7	$1.5	$1.5	$1.5	$1.3
5) Sewer Separation	$3.8	NA	NA	NA	NA
	Volume Reduction for Impervious Area Managed (MG)				
	2.50%	5%	7.50%	10%	
6) Green Infrastructure	$9.1	$7.2	$6.3	$5.8	

9 References

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