Development and Evaluation of Alternatives Report

Combined Sewer Management Permit Compliance

City of Elizabeth
Union County, NJ

50 Winfield Scott Plaza
Elizabeth, NJ 07201

NJPDES Permit No. NJ0108782

Joint Meeting of Essex and Union Counties
Union County, NJ

500 South First Street
Elizabeth, NJ 07202

NJPDES Permit No. NJ0024741

Combined Sewer Overflow Long Term Control Program

June 28, 2019

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Certifications

**Combined Sewer Overflow Long Term Control Program
Development and Evaluation of Alternatives Report**

Submitted by the following participating Permittee

City of Elizabeth
NJPDES Permit No. NJ0108782

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 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: ______________________________ 6/28/2019
Daniel J. Loomis, P.E. Date
City Engineer, City of Elizabeth

Combined Sewer Overflow Long Term Control Program
Development and Evaluation of Alternatives Report

Submitted by the following participating Permittee

Joint Meeting of Essex and Union Counties
NJPDES Permit No. NJ0024741

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Permittee: ______________________ 6/28/19
Samuel McGhee — Date
Executive Director, Joint Meeting of Essex and Union Counties

Section 1
Introduction

The City of Elizabeth (City) and the Joint Meeting of Essex and Union Counties (JMEUC or Joint Meeting) are submitting this document to meet certain conditions 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 (CSO) control, referred herein as the NJPDES CSO Permits. As permittees of a hydraulically connected system, the City and JMEUC are cooperating and collaborating on the development of a Long Term Control Plan (LTCP) for CSO control per the permit conditions and are jointly submitting this report for permit compliance. The City and JMEUC are collectively referred herein as the Permittees.

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 municipalities and commissions/authorities that own or operate facilities controlling, transporting, or treating wastewater flows from combined sewer systems. Discharges from the City of Elizabeth's 29 designated CSO outfalls are authorized and regulated under NJPDES Permit No. NJ0108782. While the Joint Meeting does not own or operate CSO control facilities or outfalls, the downstream portion of the JMEUC trunk sewer system receives and conveys combined sewage from the City and the systems are hydraulically connected. As such, the NJDEP revoked and reissued the JMEUC individual Category "A" Permit No. NJ0024741 to incorporate the NJPDES CSO Permit requirements as part of the recent permit actions.

This *Development and Evaluation of Alternatives Report* (DEAR) has been compiled by the City and JMEUC for the required "Evaluation of Alternatives" under Part IV Section G.4 of the City's NJPDES Permit No. NJ0108782 and JMEUC's NJPDES Permit No. NJ0024741. As presented herein, a comprehensive analysis of a wide range of CSO control alternatives has been conducted per the permit requirements to facilitate the selection of a practical and technically feasible Long Term Control Plan. This report documents the process used to develop alternatives and demonstrates that a full range of potential controls with respect to meeting pertinent water quality standards have been analyzed. With this detailed information, the Permittees, in consultation with NJDEP and the public, will be able to select CSO controls that best meet the needs of the public and conform to the various regulatory requirements.

NJDEP has issued similar NJPDES CSO permits to New Jersey entities who own combined sewer systems or who treat combined sewage from these systems with the intent to address combined sewer overflow impacts on the State's waters. The JMEUC and the City are members of the NJ CSO Group and have coordinated with the Group during the preparation of this DEAR, including work related to water quality modeling, CSO control technology descriptions, basis of cost estimates, and reporting on sensitive area assessments. The NJ CSO Group was originally formed to bring together utilities and municipalities that own combined sewers in Northern New Jersey, who all have the common interest of coordinating their activities and responses to local regulatory issues like the pathogen Total Maximum Daily Load (TMDL) program. The group was expanded to facilitate compliance with the NJPDES requirements established in the 2015 CSO permits and the JMEUC and the City are actively participating in the permit compliance efforts of the Group.

1.1 Regulatory Context

In the current NJPDES CSO Permits, the NJDEP has mandated that the permittees prepare a CSO Long Term Control Plan and the NJDEP has incorporated permit conditions that closely reflect the requirements of the National CSO Control Policy established by the United States Environmental

Protection Agency (EPA). 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 evaluates the scope, costs, and performance of possible control alternatives for treating or 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 CSO Permits 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:

- 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. Separate Work Plans were submitted by the Permittees; both were submitted on time and approved by NJDEP.
    - Permittees were required to submit a System Characterization Report within 36 months of the EDP, or a due date of July 1, 2018. Separate System Characterization Reports were submitted on time by the Permittees and approved by NJDEP. These documents serve as the basis for the subsequent development and evaluation of alternatives efforts (documented in this report).
    - 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). The Public Participation Process Report was prepared jointly by the Permittees and submitted on time. The Consideration of Sensitive Areas report was prepared as a cooperative effort of the NJ CSO Group and submitted on time by the

Group. Both reports were approved by NJDEP and contributed to the development and evaluation of alternatives efforts.

- o 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 then a Baseline CMP Report by July 1, 2018. The Permittees collaborated with the NJ CSO Group to satisfy these permit conditions through a regional ambient water quality sampling and testing program and pathogen water quality modeling. Both the Work Plan and Report were submitted on time by the Group and were approved by NJDEP.

Section 1.4, below, provides additional detail on the documents prepared and submitted under Step 1 of the NJPDES CSO permit process.

- 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) per the corresponding conditions prescribed in the permit. Maximizing treatment at the existing POTW treatment plant and cost and performance considerations are also to be addressed in Step 2. This report is being submitted by the Permittees in fulfillment of this permit condition for the City and JMEUC.
- 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.

Based on the National CSO Control Policy, NJDEP has incorporated the following conditions into the NJPDES CSO Permits related to the evaluation of alternatives:

- Section G.4.a stipulates that permittees are to evaluate a reasonable range of CSO control alternatives that will meet the water quality-based requirements of the CWA using either the Presumption Approach or the Demonstration Approach.
- Section G.4.b. states the DEAR is to enable the permittees, in consultation with NJDEP, the public, owners and operators of the entire collection system that conveys flows to the treatment works, to select the alternatives to ensure the CSO controls meet the water quality-based requirements of the CWA, are protective of the existing and designated uses, give the highest priority to controlling CSOs to sensitive areas, and address minimizing impacts from significant indirect user (SIU) discharges.
- Section G.4.c. indicates that permittees are to select either the Demonstration or Presumption Approach for each group of hydraulically connected CSOs and identify each CSO group and its individual discharge locations.
- Section G.4.d. notes that the DEAR is to include a list of control alternative(s) evaluated for each CSO outfall.
- Section G.4.e requires that the permittees evaluate a range of CSO control alternatives predicted to accomplish the requirements of the CWA and use hydrologic, hydraulic and water quality models approved by NJDEP in the evaluation. The models are to simulate the existing conditions and conditions as they are expected to exist after construction and operation of the chosen alternative(s).
- Section G.4.e further notes that the evaluation is to consider the practical and technical feasibility of the proposed CSO control alternative(s), and water quality benefits of constructing and implementing various remedial controls and combination of such controls and activities. It also includes a list of seven (7) control alternatives that, at a minimum, are to be evaluated.

- Section G.4.f describes the criteria of the Presumption Approach, while Section G.4.g lists the criteria of the Demonstration Approach, with each section referring to N.J.A.C. 7:14A-11 Appendix C. These criteria are described in further detail in Section 3 of this report.
- Section G.5.a indicates that the DEAR is to include cost/performance considerations to relate and compare proposed control alternatives evaluated per Section G.4 and help guide selection of controls. The analysis is to consider the diminishing incremental pollution reduction achieved in the receiving water compared to the increased costs as the level of control increases.

Under this regulatory context for the development and evaluation of CSO control alternatives, the permittees are to evaluate a sufficient number of control alternatives to guide the selection of a suitable and cost-effective long term control plan in the next phase of the process.

1.2 Report Objectives

The development and evaluation of alternative 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 specific control alternatives for the subject combined sewer system. 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 DEAR incorporates a comprehensive review and analysis of applicable CSO control strategies based on the information gathered and presented in the System Characterization Reports. JMEUC and the City have developed a thorough understanding of their wastewater collection and treatment systems, including the systems' responses to precipitation events of varying duration and intensity, and the capacity of these systems to capture and treat flows from the combined sewer system (CSS).The hydrologic and hydraulic models approved by the NJDEP have been used to simulate the system performance under the baseline conditions as well as the system 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 control programs for control levels, technical merit, ability to be implemented and costs
- Present cost/performance considerations; and,
- 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 City and JMEUC to advance the LTCP process to the Selection and Implementation of Alternatives step.

1.3 Combined Sewer System and Service Area Overview

The JMEUC owns and operates a wastewater treatment facility which treats wastewater collected in a 65 square mile service area in northern New Jersey. The JMEUC trunk sewer system collects wastewater from this service area, which includes eleven member (owner) communities and four customer communities. Owner communities include all or some parts of East Orange, Hillside, Irvington, Maplewood, Millburn, Newark, Roselle Park, South Orange, Summit, Union, and West Orange. The City of Elizabeth and portions of Livingston, Orange, and New Providence are currently served as customers by the JMEUC. Small portions of two neighboring communities, Berkeley Heights and Linden are also served. Note: Only portions of Newark, Berkeley Heights, Linden, Roselle and Livingston are within the service area of JMEUC.

Figure 1-1 depicts the locations of trunk sewer system, communities served, and the wastewater treatment facility.

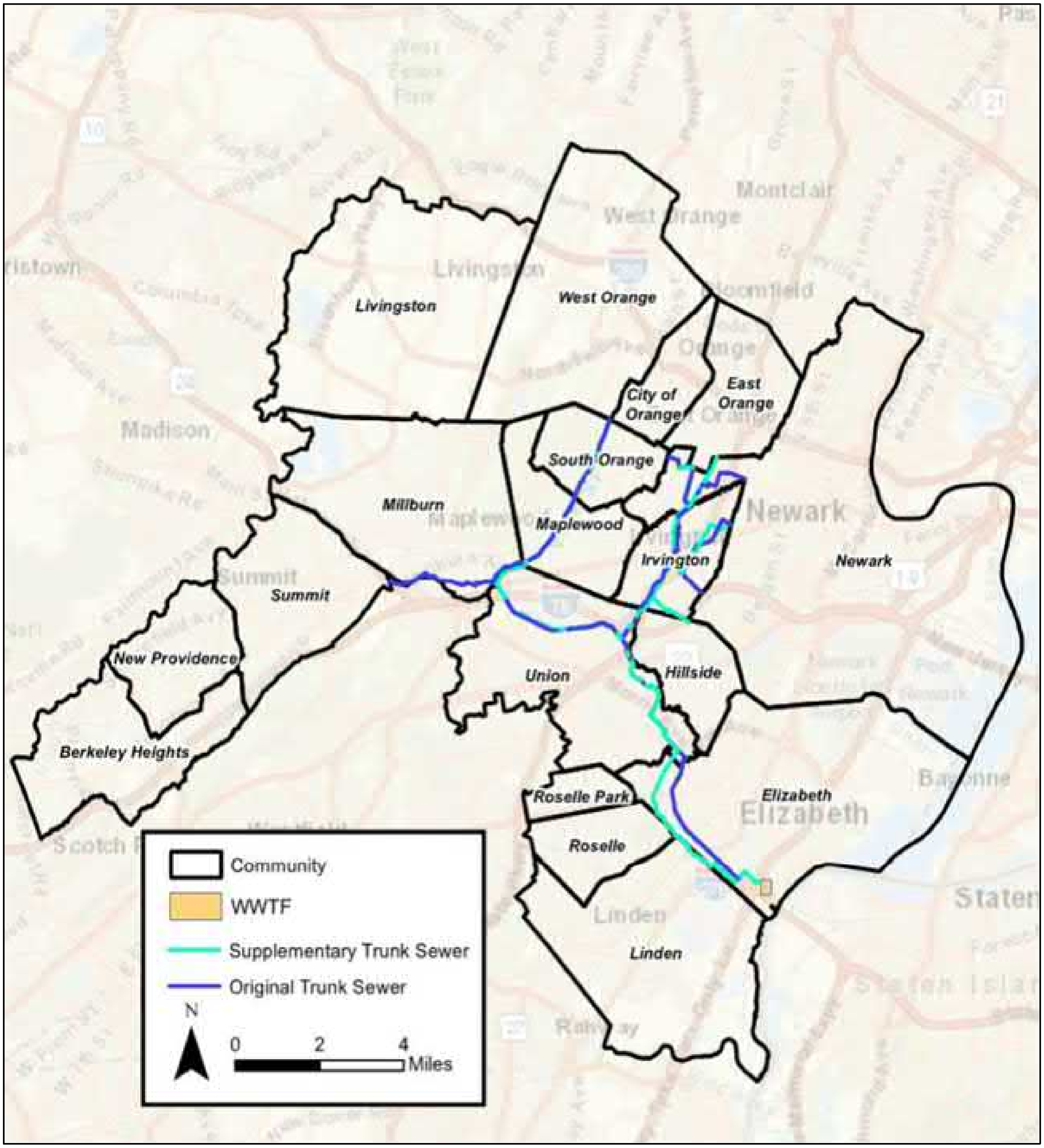
The JMEUC service area is dominated by separate sanitary sewer areas, with the only confirmed combined sewer area in the system located within the City of Elizabeth. The JMEUC has coordinated with Elizabeth, and will continue to coordinate with Elizabeth, to identify portions of Roselle Park and possibly other adjoining towns that flow into Elizabeth that may also be combined, or have their storm sewers connected into Elizabeth's combined or separate sanitary sewers. Similarly, the JMEUC has identified New Jersey Department of Transportation (NJDOT) catch basin connections into the sanitary and/or combined sewer systems in JMEUC's service area.

Separate sanitary sewers owned by each JMEUC member community and the separate sewer areas in the customer communities provide local sewer service, and the largely combined sewer system in Elizabeth provides local sewer service for that community. The JMEUC trunk sewers capture flows from these local sewer systems and provide regional conveyance of all flows to the Edward P. Decher Secondary Wastewater Treatment Facility (WWTF) located in Elizabeth, New Jersey. The JMEUC trunk sewer system totals roughly 43 miles in length, and was developed as a network of twin conduits, referred to as the "Original" and "Supplementary" sewers, aligned more or less in parallel for the full length of the system. The Original Trunk Sewer system was constructed in the early 1900's, and the much larger Supplementary Trunk Sewer in the 1930's.

The JMEUC WWTF is a conventional activated sludge plant rated for an average flow of 85 mgd. The preliminary treatment consists of mechanical coarse screens followed by mechanical fine screens followed by gravity grit chambers. From the grit chambers, the wastewater flows to four rectangular primary clarifiers. The primary clarifier effluent is conveyed to four aeration tanks by five low lift pumps. The aeration tanks are equipped with mechanical aerators. From the aeration tanks, the mixed liquor flows to four circular secondary clarifiers. The secondary effluent is then disinfected using sodium hypochlorite, and then dechlorinated with sodium bisulfite. The treated wastewater is then discharged through one of two outfall conduits into the Arthur Kill (the second conduit is available for emergency bypass of primary effluent), just below Newark Bay. An additional primary effluent emergency bypass outfall is located at the Elizabeth River just above the confluence with the Arthur Kill.

The City of Elizabeth provides wastewater and stormwater collection and conveyance services to about 128,600 people within its municipal boundaries, which encompasses approximately 12.3 square miles in Union County, NJ. This collection and conveyance system consists of an extensive network of sewers, manholes, catch basins, pump stations, overflow control facilities, and drainage conduits, totaling over 210 miles of pipe. The City of Elizabeth does not own or operate any wastewater treatment plant facilities; wastewater flows are conveyed to the JMEUC WWTF. The City owned sewer system assets are operated and maintained through a multi-year service contract with E'Town Services LLC, a subsidiary of American Water.

Much of the City is served by a CSS that collects and conveys sanitary and stormwater flows in the same conduit. The combined sewers are prevalent throughout the northern, western, and southern sections of the City, 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. In other areas of the City, sanitary flows are conveyed in a separate (sanitary) sewer system connected to interceptors, with stormwater runoff conveyed by a separate storm sewer system.



Note: Only portions of Newark, Berkeley Heights, Linden, Roselle and Livingston are within the service area of JMEUC.

Figure 1-1: Municipalities Served by JMEUC

All dry weather sewage from the City owned sewer system is conveyed to and treated at the JMEUC WWTF. Except for flows from sewers directly connected to the Joint Meeting trunk sewers, wastewater is collected and conveyed by two City-owned intercepting sewers serving the easterly and westerly portions of the City, respectively. These intercepting sewers flow to the Trenton Avenue Pumping Station (TAPS), which is the City's main pumping station, and its force main discharges flows to the JMEUC incoming trunk sewer approximately 1,300 feet upstream of the wastewater treatment facilities. The City is a customer of JMEUC, not a member municipality, and is currently contractual limited to an 18 mgd maximum average daily flow and a 36 mgd maximum instantaneous peak discharge from its main wastewater pumping station to the JMEUC treatment works.

1.4 Previous Studies

This report builds on the System Characterization Reports prepared by the Permittees and approved by NJDEP under the first part of the NJPDES CSO Permits. Other prior work plans and reports submitted by the Permittees and through the NJ CSO Group are also referenced. These recent permit submissions and reports include:

- System Characterization Report, prepared by CDM Smith for the Joint Meeting of Essex and Union Counties, dated June 2018, revised December 2018.
- System Characterization Report, prepared by Mott MacDonald for the City of Elizabeth, dated June 2018, revised January 2019.
- System Characterization Work Plan, prepared by CDM Smith for the Joint Meeting of Essex and Union Counties, dated December 2015, revised June 2016.
- System Characterization Work Plan: Quality Assurance Project Plan, prepared by Hatch Mott MacDonald on behalf of the City of Elizabeth, dated December 2015, revised May 2016.
- Public Participation Process Report, completed for the City of Elizabeth and Joint Meeting of Essex and Union Counties, dated June 2018, revised November 2018.
- Identification of Sensitive Areas Report, prepared by the Passaic Valley Sewerage Commission on behalf of participating permittees of the NJ CSO Group, dated June 2018, revised March 2019.
- NJ CSO Group Compliance Monitoring Program Report, prepared by the Passaic Valley Sewerage Commission on behalf of participating permittees of the NJ CSO Group, dated June 2018, revised October 2018.
- Pathogen Water Quality Model (PWQM) Quality Assurance Project Plan (QAPP), prepared by the Passaic Valley Sewerage Commission on behalf of participating permittees of the NJ CSO Group, dated May 2016, revised January 2017.
- Typical Hydrologic Year Report, prepared by the Passaic Valley Sewerage Commission on behalf of participating permittees of the NJ CSO Group, dated May 2018.

Reports from previous permit cycle submissions that were consulted for the cost and performance of CSO control strategies are:

- Long Term Control Plan, Cost and Performance Analysis Report, completed by CDM for JMEUC in March 2007.
- CSO Long Term Control Plan, Cost & Performance Analysis Report, Volume 1, prepared by Hatch Mott MacDonald for the City of Elizabeth, dated March 2007.
- CSO Long Term Control Plan, Cost & Performance Analysis Report, Volume 2 - Technical Guidance Manual, prepared by Hatch Mott MacDonald for the City of Elizabeth, dated March 2007.

Section 2
Sewer System Collection and Treatment Facilities Overview

The Joint Meeting of Essex and Union Counties (JMEUC) provides wastewater conveyance and treatment service to eleven-member communities along with four customer communities within Essex and Union counties. The JMEUC owns and operates the Edward P. Decher Secondary Wastewater Treatment Facility (WWTF) in Elizabeth, New Jersey and the service area includes both separate sanitary sewer systems and combined sewer systems. Flow from upstream communities is conveyed to the WWTF via two trunk sewers (Original and Supplementary) owned and operated by the JMEUC. In the downstream portion of the collection system, the Original and Supplementary Trunk Sewers come together at a junction (herein called Junction J16) at the intersection of Bayway Avenue and Pulaski Street. A twin barrel trunk sewer (the North Barrel and South Barrel) exits J16 with flow being split relatively equally between the two barrels before reaching the WWTF. In addition to member and customer communities flows, flow is received from catch basin connections into the JMEUC trunk sewers from the New Jersey Department of Transportation (NJDOT) drainage system along Elmora Avenue and Bayway between Westfield and Brunswick Avenues in the downstream portion of the JMEUC's service area within Elizabeth. The extents of the JMEUC service area are indicated in Figure 1-1.

The combined sewer area contributing flow to the JMEUC trunk sewers is limited to the City of Elizabeth, which is a customer municipality of JMEUC. The City is a major urban center located in eastern Union County and is situated along the west banks of the Arthur Kill and Newark Bay. It is the state's 4th largest municipality by population with approximately 128,000 inhabitants. The City is bounded by the City of Newark (in Essex County, NJ) to the north, by the Townships of Hillside and Union to the northwest, by the Boroughs of Roselle Park and Roselle to the west, and by the City of Linden to the southwest. Bayonne (in Hudson County, NJ) is located to the east across Newark Bay and the borough of Staten Island (New York) is located across the Arthur Kill to the south.

This section summarizes the key elements of the JMEUC and Elizabeth sewer service areas and systems as background for identifying and reviewing appropriate CSO control strategies. Detailed descriptions of the sewer system facilities, receiving waters, and hydrologic and hydraulic model development are provided in the following System Characterization Reports:

- System Characterization Report, prepared by CDM Smith for the Joint Meeting of Essex and Union Counties, dated June 2018, revised December 2018.
- System Characterization Report, prepared by Mott MacDonald for the City of Elizabeth, dated June 2018, revised January 2019.

2.1 JMEUC Separate Sanitary Sewer Service Area Description

The eleven member communities of the JMEUC along with the customer communities of Livingston, Orange, and New Providence (along with small portions of Berkeley Heights and Linden) are serviced by separate sanitary sewer systems which are owned and operated by each individual community. These systems are tributary to the Original and Supplementary Trunk Sewers owned and operated by the JMEUC, which collect and convey flows from these communities to the WWTF. The total population of the separated sewer service area is estimated to be 327,313 based on American Community Survey 2011-2015 5-year estimates, while the total sewered area of these communities (excluding large parks and other significant open spaces) is estimated to be 29,780 acres or 46.5 square miles.

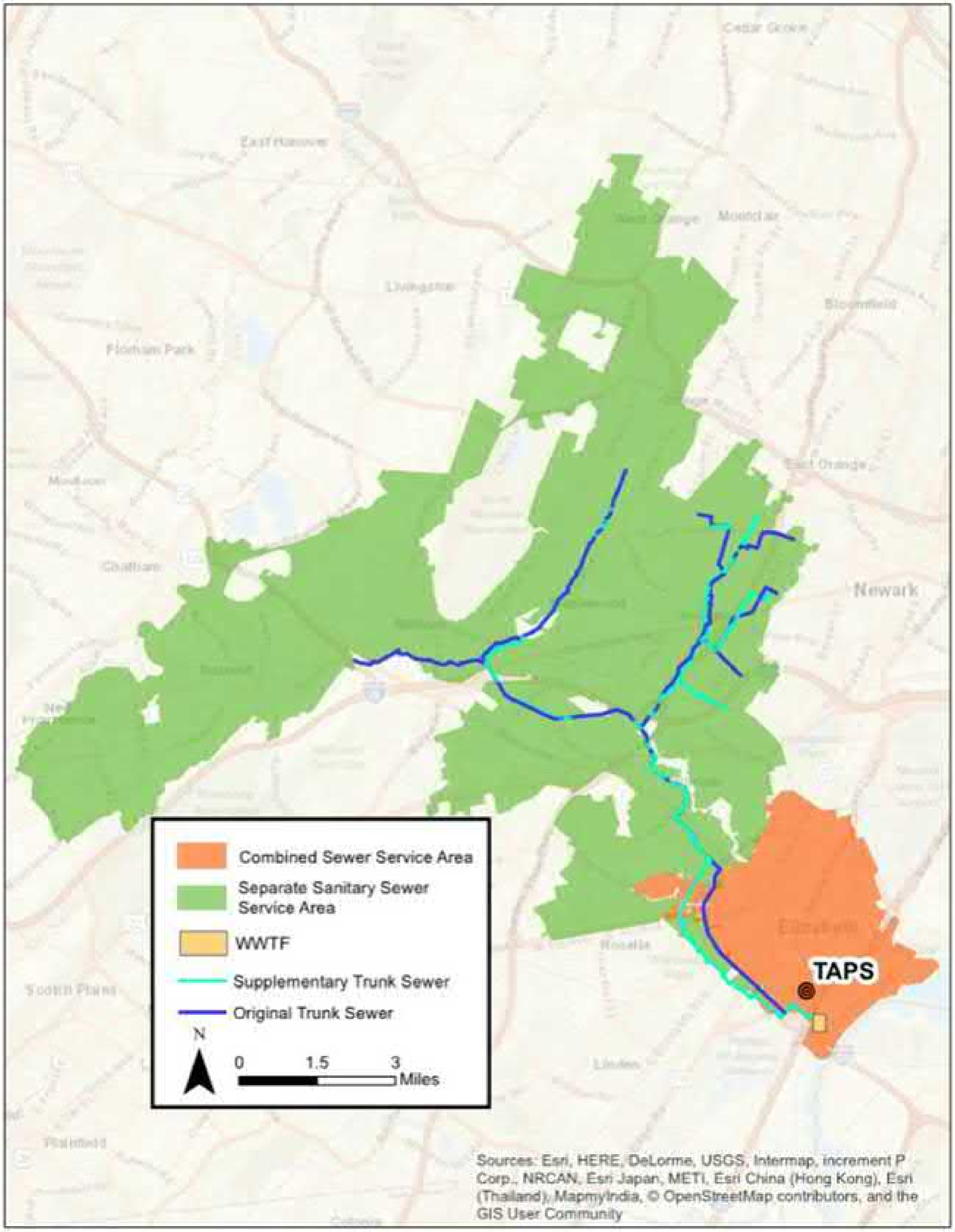


Figure 2-1: Municipalities Served by JMEUC

Over two-thirds of the JMEUC separate sanitary sewer service area is made up of residential property, of which most is either medium or high-density housing. Commercially developed land makes up the next highest land use percentage (15%), while the remaining areas are evenly distributed among wooded, recreational, industrial, and transportation land uses. Population estimates and sewered areas are broken down by community in Table 2-1. Figure 1-1 in Section 1 shows the locations of the communities which make up the separate sewer portion of the JMEUC collection system, along with their locations relative to the JMEUC trunk sewers.

Table 2-1: Separated Sewer Communities Served by JMEUC

Member Community (see footnotes below)	Estimated Population Serviced by the JMEUC	Sewered Area (acres)
East Orange [1]	17,247	570
Hillside	20,415	1,570
Irvington	55,774	1,870
Maplewood	23,156	1,890
Millburn and Livingston	17,322	3,840
Newark[1]	44,284	1,210
Roselle Park [2]	11,735	680
South Orange	16,257	1,670
Summit [3]	31,978	5,700
Union	53,871	5,140
West Orange [4]	40,743	5,440

[1] Population and area values include only the portion of the community serviced by JMEUC. Remainder of community is serviced by Passaic Valley Sewerage Commission.
[2] Population and area values include only the portion of the community serviced by JMEUC. Remainder of community is serviced by Rahway Valley Sewerage Commission.
[3] Population and area values include the customer community of New Providence and portion of Berkeley Heights serviced by the JMEUC.
[4] Population and area values include Customer Community of City of Orange.

2.2 JMEUC Trunk Sewer System

The JMEUC does not own or operate any portion of member or customer community collection systems upstream of the two trunk sewers. The JMEUC trunk sewer system includes the Original Trunk Sewer constructed in the early 1900's and the Supplementary Trunk Sewer constructed in the 1930's. They generally run parallel to one another throughout the service area. In the downstream portion of the collection system, the Original and Supplementary Trunk Sewers come together at Junction J16 at the intersection of Bayway Avenue and Pulaski Street. A twin barrel trunk sewer (the North Barrel and South Barrel) exit J16 with flow being split relatively evenly between the two barrels. Together, the total length of the trunk sewers owned and operated by the JMEUC is approximately 43 miles.

There are approximately 900 manholes which serve as access points to the trunk sewers from the tributary collection systems. The diameters of the trunk sewers range in size from 10" in the most upstream portions of the system in Newark and Irvington, to 81" in the downstream portion of the Supplementary Trunk Sewer. Figure 2-2 through Figure 2-5 show the trunk sewer network and associated pipe shapes and sizes. All pipes within the trunk sewer network are circular except the twin barrel trunk sewer in the downstream portion of the system and a short stretch of rectangular pipe making up the Original Trunk Sewer, as indicated in Figure 2-4.

All flow within the JMEUC trunk sewers is conveyed downstream via gravity, although four pump stations are present immediately upstream of the trunk sewer network. Three of the pump stations convey

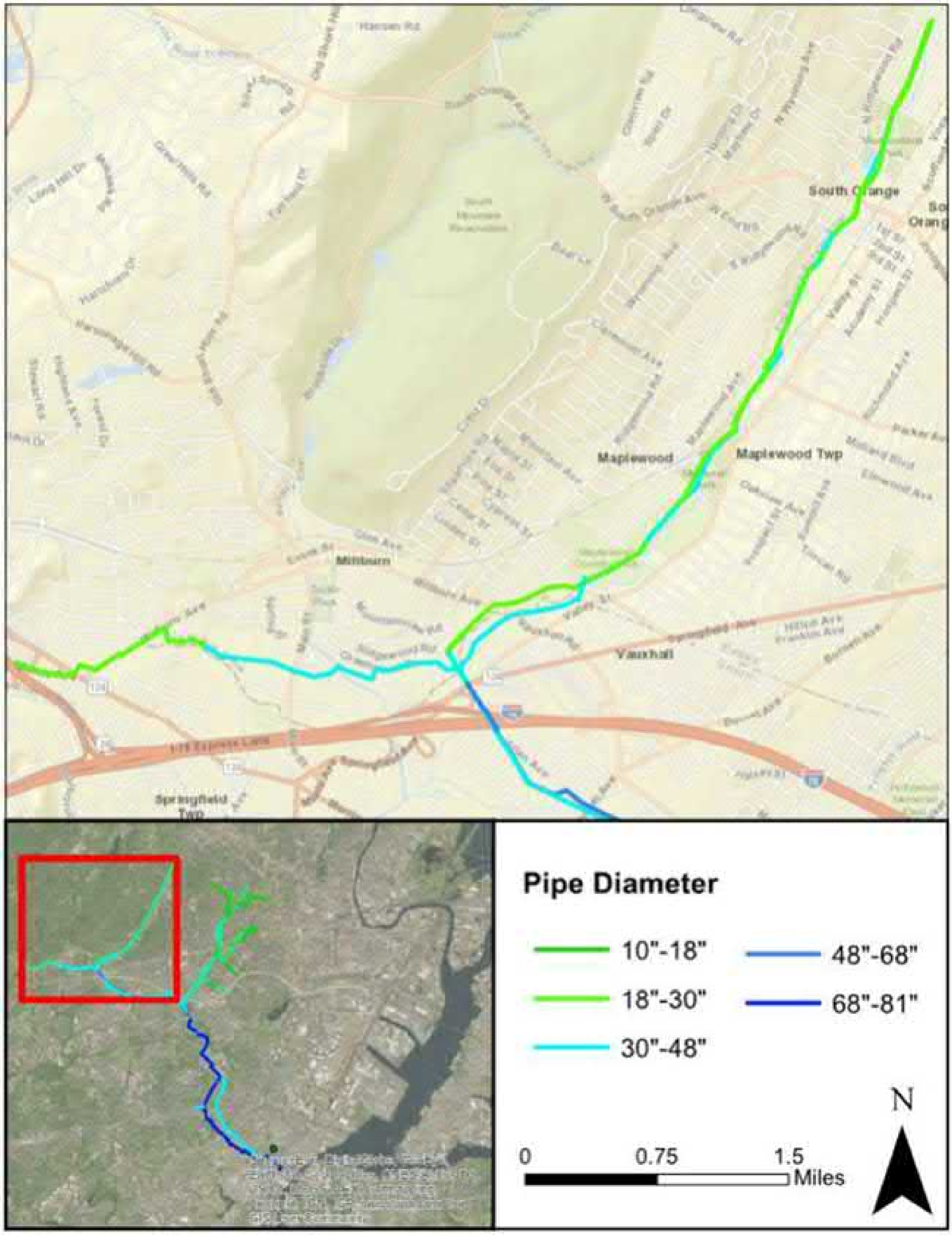


Figure 2-2: JMEUC Trunk Sewer Pipe Sizes and Shapes – Northwest Portion of Service Area

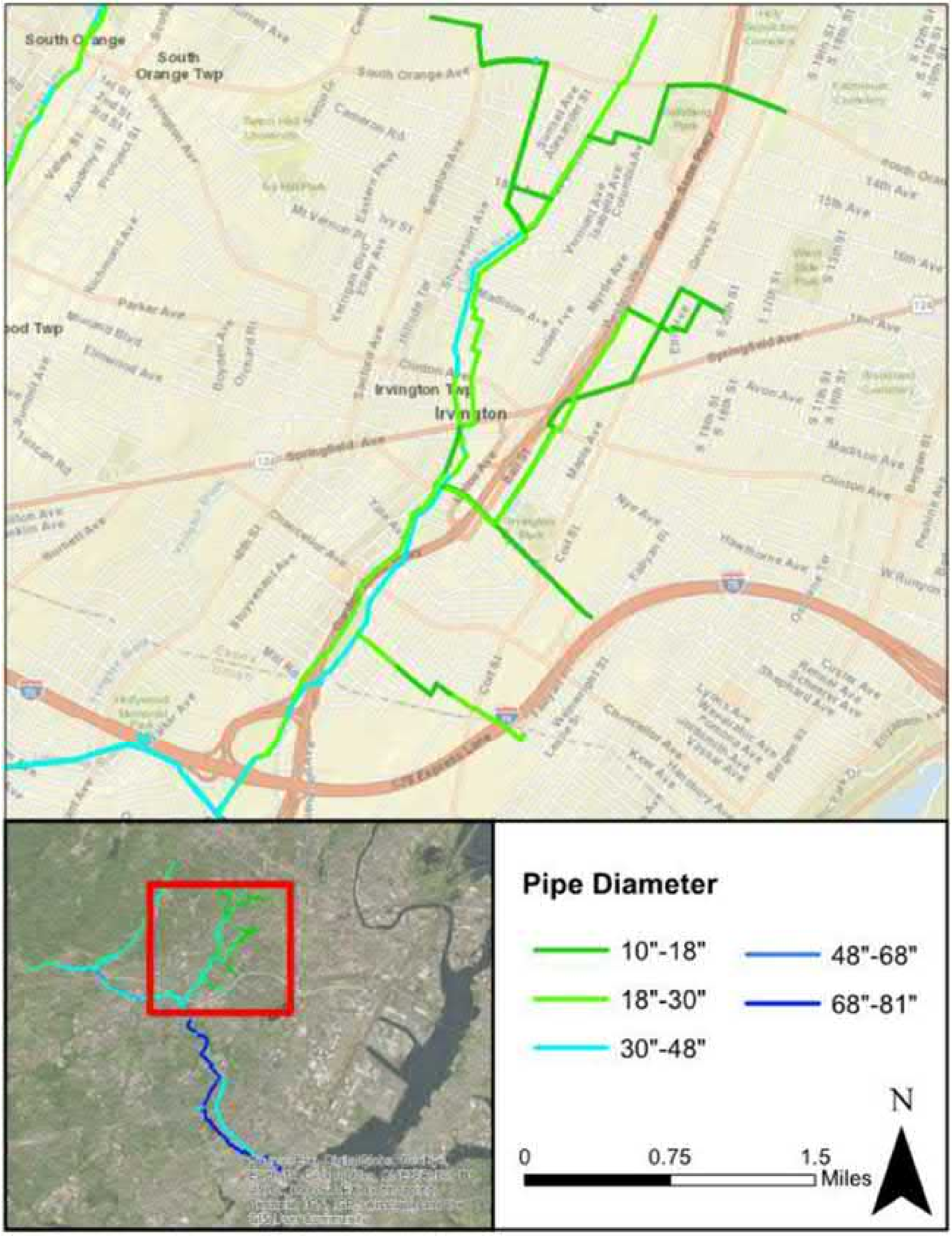


Figure 2-3: JMEUC Trunk Sewer Pipe Sizes and Shapes – Northern Portion of Service Area

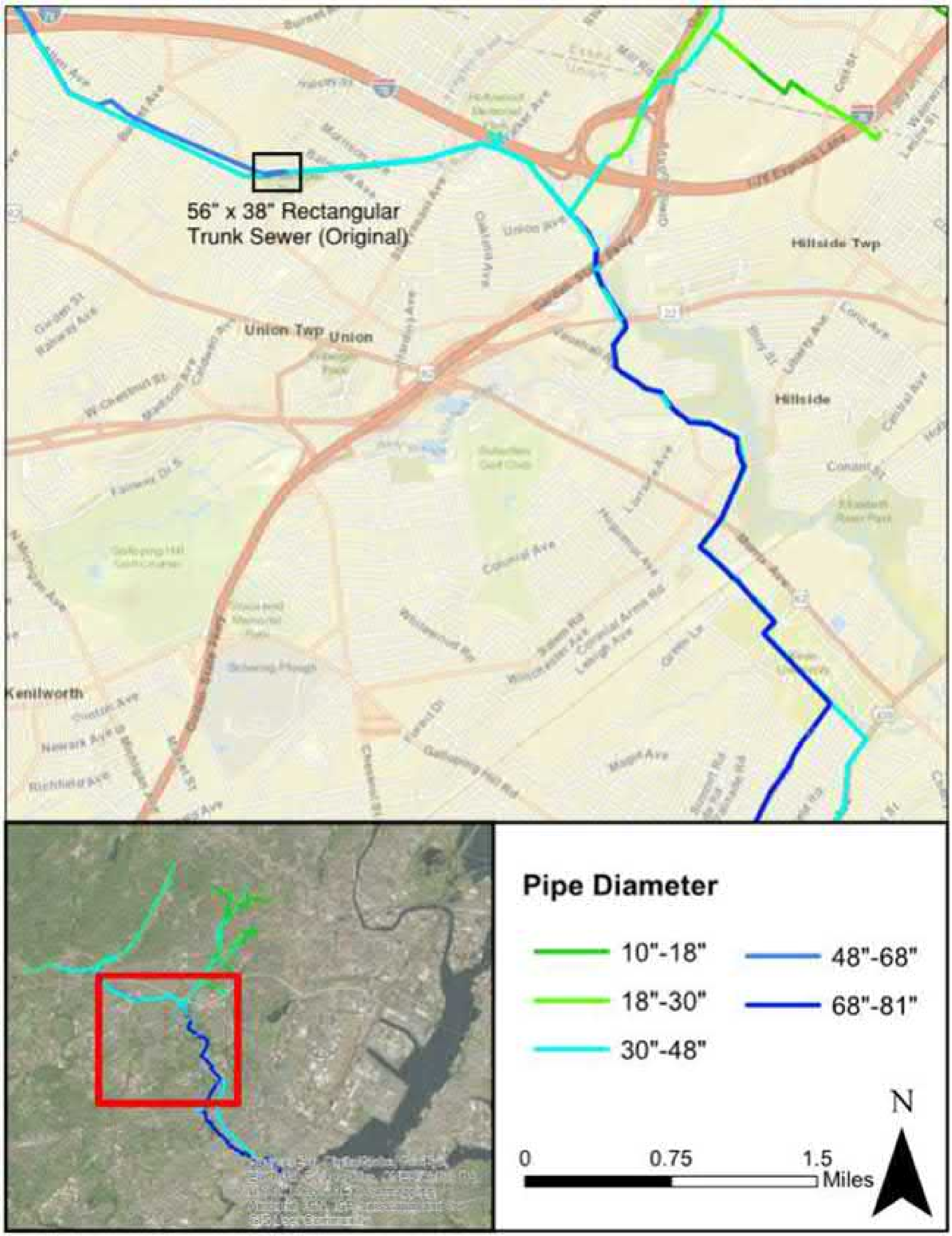


Figure 2-4: JMEUC Trunk Sewer Pipe Sizes and Shapes – Central Portion of Service Area

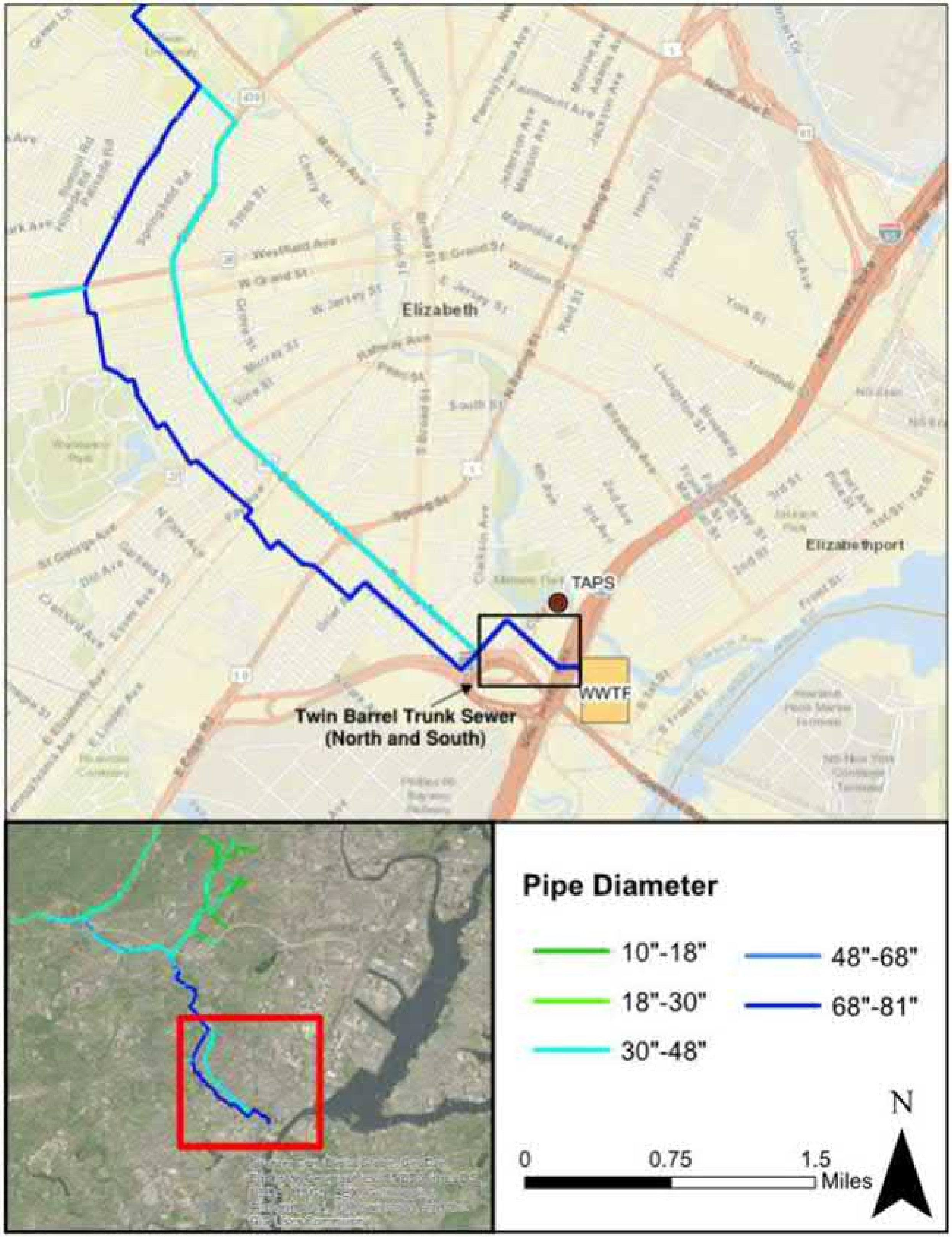


Figure 2-5: JMEUC Trunk Sewer Pipe Sizes and Shapes – Southeast Portion of Service Area

separated wastewater flows to the trunk sewer system, while the Trenton Avenue Pumping Station (Trenton Avenue PS or TAPS) conveys combined flows from the City of Elizabeth to the North Barrel of the twin barrel trunk sewer. There are no constructed relief points to the receiving waters within the trunk sewer system. There are a total of 18 cross connections (relief sewers) and 16 junctions throughout the trunk sewer network which divert and distribute flow among the two trunk sewers to maximize conveyance capacity of the system during wet weather flow (WWF) conditions. These connections and junctions balance flow and head in the system, thereby avoiding the overloading of one trunk while capacity may be available in the other.

The trunk sewer network also includes two inoperable venturi meters and four areas of depressed pipe segments below stream/river crossings. The venturi meters are not currently used to measure flows, but they are still able to convey flows via inverted siphons. Additionally, both venturi meters have bypass structures which add additional localized capacity and allow for some flow to bypass the inverted siphons. There are also four areas of depressed pipe segments under stream/river crossings that can impact the hydraulic conditions in the trunk sewers. At the depressed pipe locations, the pipe maintains its slope and transitions in cross-sectional shape from circular to rectangular and then back to circular.

Historically, the JMEUC has not observed issues with sewer system overflows or flooding and the hydraulic modeling results have indicated no measurable flooding in the JMEUC system during the Typical Year rainfall, as described in the City of Elizabeth and JMEUC System Characterization Reports.

2.3 Edward P. Decher Secondary Wastewater Treatment Facility

The Edward P. Decher Secondary Wastewater Treatment Facility has a rated peak hydraulic capacity of 180 million gallons per day (mgd), although flows reaching 220 mgd may be processed during significant wet weather events. Peak discharge from the WWTF is limited by mean sea level (MSL), with rated capacity of the WWTF dropping to 120 mgd when tides exceed eight feet above MSL (corresponding to 13-year recurrence interval). The plant is rated for average daily influent flows of 85 mgd.

2.3.1 Preliminary Treatment

Flows from the Original and Supplementary Trunk Sewers enter the headworks of the WWTF and are diverted to one of two paired sets of coarse and fine screens. No pumping of the influent is required at the headworks of the WWTF. Flow passes by gravity first through the coarse screens and then through the fine screens. The coarse screens have 3.5-inch clear openings while the fine screens have 0.75 inch clear openings. When both sets of screens are on-line flow is typically split evenly between the paired sets of screens. Effluent flow from the fine screen enters four grit channels, each measuring 9.5 feet wide by seven feet deep by 57 feet long.

2.3.2 Primary Treatment

Flow exiting the individual grit channels is combined at a downstream flume which routes flow to a collection channel immediately upstream of four primary settling tanks (PSTs). The four PSTs have identical geometries (200 feet long by 75 feet wide by 13.8 feet deep). During dry weather flow (DWF) conditions, only two of the four PSTs are on-line. A third PST is brought on-line during WWF events when flows measured directly upstream of secondary treatment exceed 100 mgd. The fourth PST is only brought on-line in emergency situations such as power failure.

The four PSTs have effluent weir lengths of 75 feet each, with effluent flow entering a collection channel before flowing to the primary effluent chamber. Under normal operating conditions, flow exits the primary effluent chamber and enters a six foot by 10 foot box-shaped conduit which conveys flow to the Main Sewage Pumps wet well. The wet well feeds five low lift pumps, all equipped with variable frequency drives. Two pumps are normally in operation at all times, and their pumping rate controlled by the water level of the wet well. When flows discharging from the wet well exceed 100 mgd, a third and occasionally fourth pump are turned on manually to maintain the water level in the wet well. Collectively the five wet well pumps have a capacity of over 200 mgd, enough to maintain proper water levels in the plant during extreme wet weather events.

The primary effluent chamber also has two emergency overflows (one discharging to the Arthur Kill and the other discharging to the Elizabeth River). Activation of these overflows is controlled by the primary effluent chamber water level and by gates in the chamber which are normally closed. These emergency overflows have not activated in many years and any activation of these overflows would most likely be due to downstream mechanical issues as opposed to insufficient downstream capacity.

2.3.3 Secondary Treatment and Disinfection

The WWTF has four aeration tanks, each with a volume of 3.97 million gallons (15.89 million gallons total). Each aeration tank has eight surface aerators rated at 100 horsepower and two-speed operation capable of providing a maximum of 2,360 lb/hour of oxygen per tank. Effluent flows from the aeration tanks enter four final settling tanks (FSTs), each having a diameter of 180 feet and a depth of 15 feet. FST effluent flows are disinfected with sodium hypochlorite in a chlorine contact tank capable of treating a peak hour flow of 73 mgd at the required contact time of 20 minutes. The disinfected effluent is then dechlorinated with sodium bisulfate before being discharged to the Arthur Kill through two outfall conduits.

2.4 City of Elizabeth Sewer System Description

The City's wastewater and stormwater collection and conveyance system consists of a complex network of intercepting sewers, sewer mains, manholes, catch basins, pump stations, overflow control facilities, outfalls, and drainage channels. As shown in Note: Only portions of Newark, Berkeley Heights, Linden, Roselle and Livingston are within the service area of JMEUC.

Figure 1-1, the City of Elizabeth is located at the downstream end of the JMEUC service area. Except for flows from sewers directly connected to the JMEUC trunk sewers, wastewater is collected and conveyed by two (2) City owned intercepting sewers serving the easterly and westerly portions of the City, respectively. These intercepting sewers flow to the Trenton Avenue Pumping Station, which is the City's main pumping station located along the west bank of the Elizabeth River in the southern end of the City and only separated by the New Jersey Turnpike from the Joint Meeting WWTF. As such, the Trenton Avenue PS discharges flows to the North Barrel of the JMEUC twin barrel incoming trunk sewer approximately 1,300 feet upstream of the wastewater treatment facilities.

Figure 2-6, Figure 2-7, and Figure 2-8 depict the location of the major sewer system components in the northwestern, northeastern, and southern sections of the City, respectively. The location of Significant Indirect Users (SIU) within the City are also noted on these figures. In general, these major sewer system facilities include:

- Approximately 159 miles of combined gravity sewer mains and trunks, with an estimated 6,400 manholes and 3,300 inlets and catch basins associated with these lines.
- Approximately 9.5 miles of separate sanitary sewers, with about 310 manholes associated with these lines.
- Approximately 38 miles of separate storm sewers, with an estimated 700 manholes and 1,700 inlets and catch basins associated with these lines.
- Twenty-nine (29) permitted combined sewer overflow (CSO) outfall discharge points, 38 regulator and diversion structures, and associated solids/floatables control facilities and tide gate chambers.
- Two (2) intercepting sewer lines, totaling 6.6 miles: 4.3 miles for the Easterly Interceptor and 2.3 miles for the Westerly Interceptor.

- A total of 9 pumping stations: 3 sewage pumping stations and 6 stormwater pumping stations.
- Stormwater drainage ditches and channels that convey stormwater as well as combined sewer overflows in certain locations to receiving waters.

Statistics on the major components of the Elizabeth sewer system are summarized in Table 2-2. Furthermore, Plate A in Appendix A provides a large format map exhibit of the overall sewer system.

Table 2-2: Major Components of Sewer System

Component	Length/Number (approx.)
Gravity sewer mains (miles)	206.5 total 159.0 combined sewer 9.5 separate sanitary 38.0 separate storm
Manholes (estimated number)	7,410 total 6,400 combined sewer 310 separate sanitary 700 separate storm
Inlets and catch basins (estimated number)	5,000 total 3,300 combined sewer 1,700 separate storm
Interceptor sewers (miles)	6.6 total 4.3 Easterly Interceptor 2.3 Westerly Interceptor
Pump Stations – Sanitary/Combined Sewer	3 Trenton Avenue Pump Station (TAPS) Kapkowski Road Pump Station West Jersey Street Pump Station
Pump Stations – Stormwater System	6 Arch Pump Station Verona-Gebhardt Pump Station South Street Pump Station Mattano Park Pump Station South Second Street Pump Station South First Street Pump Station (operated and maintained by JMEUC)
Siphons	8
Permitted CSO Outfall Discharge Outlets	29
CSO Regulators	39
Solids/Floatable Control Facilities	35

Much of the City has combined sewers that collect and convey sanitary and stormwater flows in the same conduit, but in certain areas, sanitary flows are conveyed in a separate (sanitary) sewer system connected to interceptors and stormwater runoff is conveyed by separate storm sewers and drainage channels. The City has an area of approximately 7,168 acres situated in the JMEUC sewer service area, of which 4,090 acres have combined sewer systems, 2,208 acres have separate sanitary sewers, and 870 acres do not generate sanitary sewage flows, consisting of major highway infrastructure corridors. Furthermore, the combined sewers in certain locations do not have diversions to outfalls for discharge to receiving waters, but rather instead are directly connected to the intercepting sewers. As such, a total area of approximately 3,492 acres is tributary to combined sewer overflow control points and is further delineated as the CSO drainage basins.

Map Document: (P:\340878\GIS\MXD\Alternatives Analysis\SewerSystemComponents_NorthWestElizabeth.mxd) 6/25/2019 11:59:27 AM mar51660

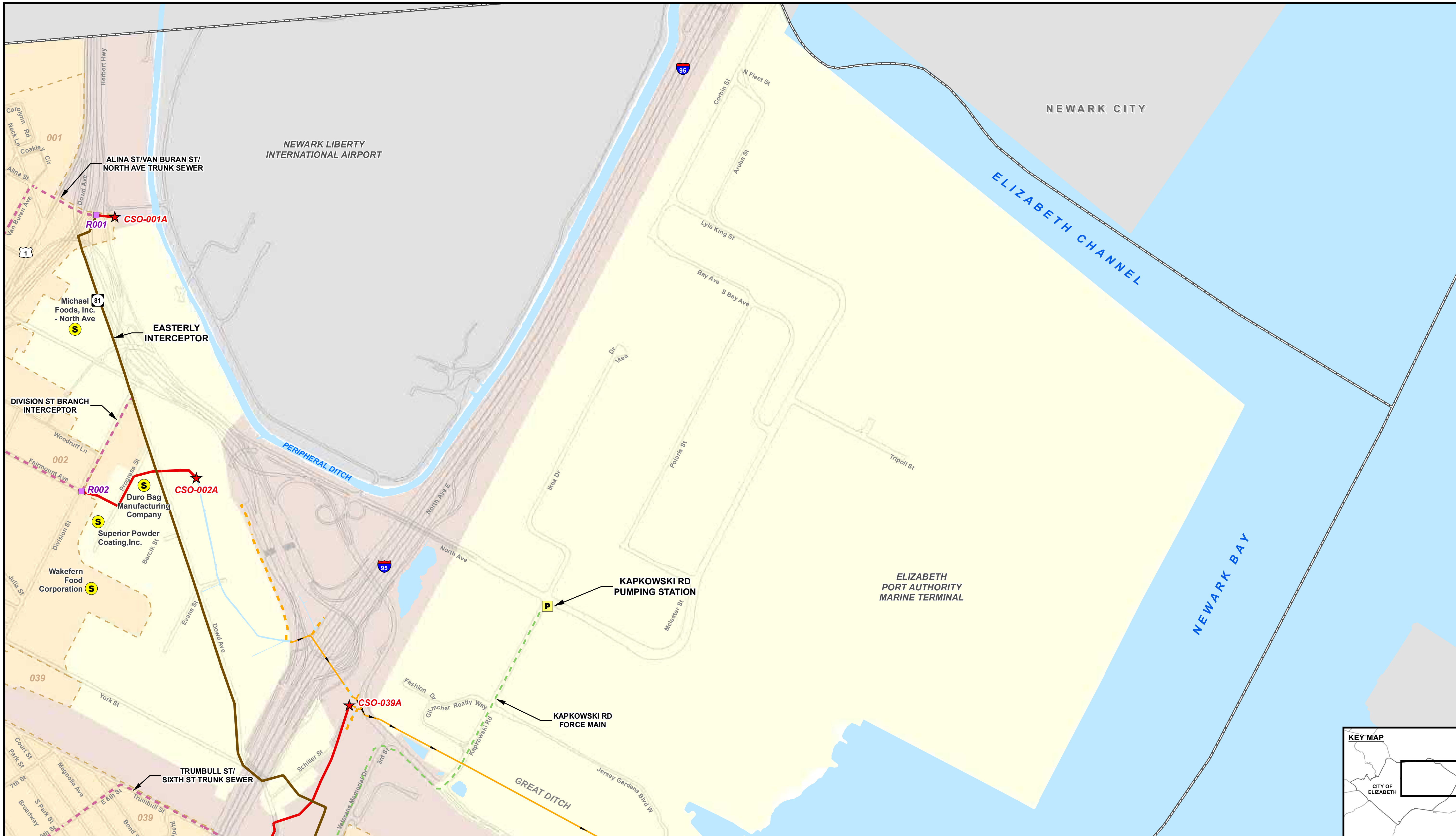
FIGURE 2-6

MOTT MACDONALD
111 Wood Avenue South
Iselin, New Jersey 08830-4112

CITY OF ELIZABETH
UNION COUNTY, NEW JERSEY
CSO LONG TERM CONTROL PROGRAM
ALTERNATIVES EVALUATION REPORT

SEWER SYSTEM COMPONENTS NORTHWEST ELIZABETH

Designed	Drawn	Checked	Approved	Date
AG	MM	AG	AG	6/25/2019



Legend

- S Significant Industrial Users
- STP JMEUC Treatment Plant
- P Pumping Station
- CSO Discharge
- Regulator
- Elizabeth City Hall
- Force Main
- Siphon Main
- Trunk Sewer Alignments
- CSO Outfall

Interceptor Alignment

- Easterly Interceptor
- Westerly Interceptor
- JMEUC Trunk Sewer
- Municipal Boundary
- Regulator Basin

Sewer System Drainage Types

- Combined Sewer System
- Seperated Sewer System
- No Contributing Flow

0 1,200 2,400 Feet

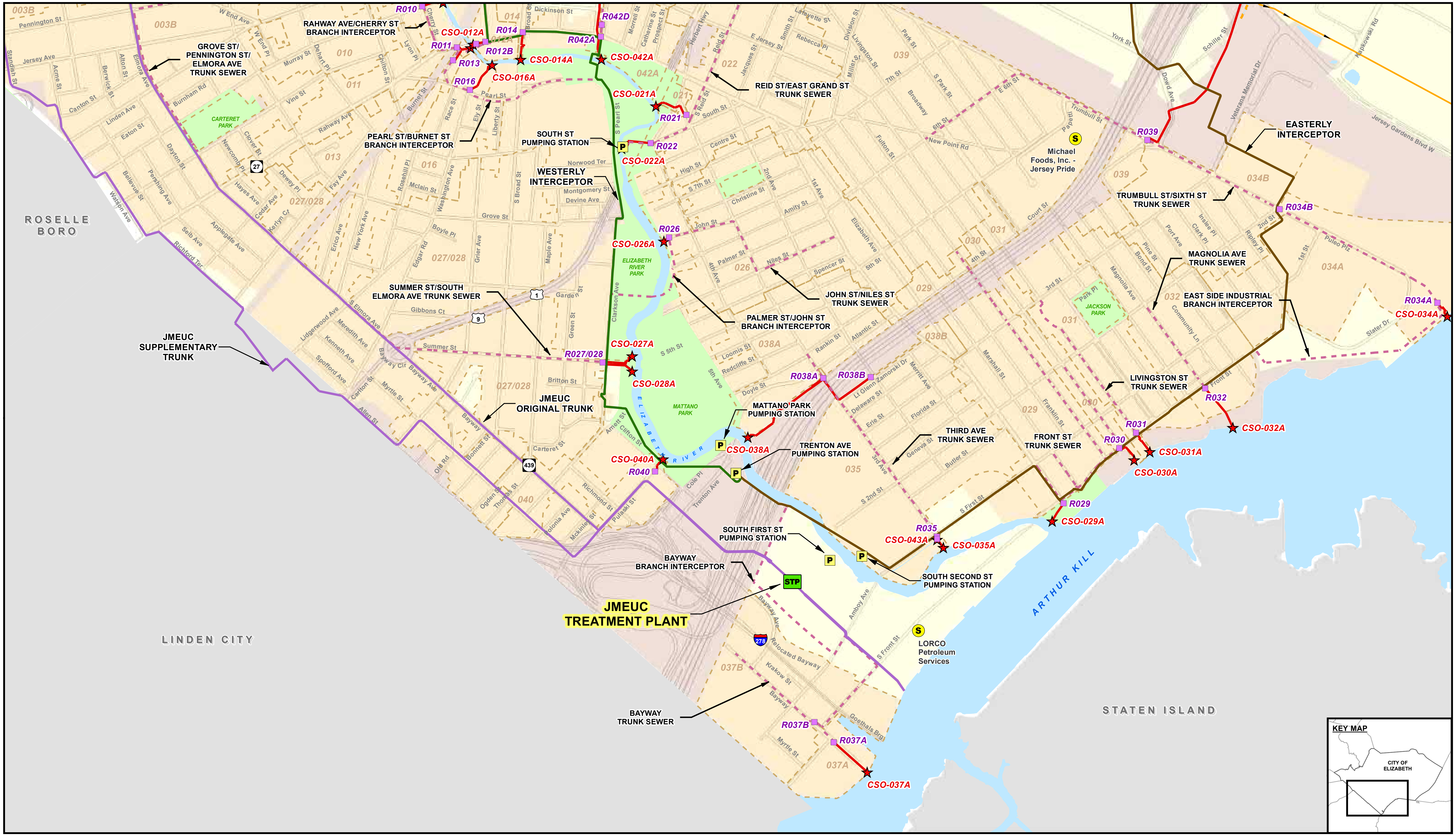
FIGURE 2-7

MOTT MACDONALD
111 Wood Avenue South
Iselin, New Jersey 08830-4112

CITY OF ELIZABETH
UNION COUNTY, NEW JERSEY
CSO LONG TERM CONTROL PROGRAM
ALTERNATIVES EVALUATION REPORT

SEWER SYSTEM COMPONENTS NORTHEAST ELIZABETH

Designed	Drawn	Checked	Approved	Date
AG	MM	AG	AG	6/13/2019



Legend

- Significant Industrial Users
- STP JMEUC Treatment Plant
- P Pumping Station
- CSO Discharge
- Regulator
- Elizabeth City Hall
- Trunk Sewer Alignments
- CSO Outfall
- Force Main Alignment
- Edge Of Pavement

Interceptor Alignment

- Easterly Interceptor
- Westerly Interceptor
- JMEUC Trunk Sewer
- Municipal Boundary
- Regulator Basin

Sewer System Drainage Types

- Combined Sewer System
- Seperated Sewer System
- No Contributing Flow

0 1,200 2,400 Feet

FIGURE 2-8

MOTT MACDONALD
111 Wood Avenue South
Iselin, New Jersey 08830-4112

CITY OF ELIZABETH
UNION COUNTY, NEW JERSEY
CSO LONG TERM CONTROL PROGRAM
ALTERNATIVES EVALUATION REPORT

SEWER SYSTEM COMPONENTS
SOUTH ELIZABETH

Designed	Drawn	Checked	Approved	Date
AG	MM	AG	AG	6/25/2019

On a composite (i.e., percent total) basis using the 2012 NJDEP geographic information system dataset, the overall CSO drainage area is approximately 52.2% high density residential, 8.2% medium density residential, 0.1% low density residential, 17.3% commercial, 11.6% industrial, 3.5% open areas, 3.3% transportation, 1.7% mixed urban, 1.9% other urban, and 0.2% wetlands. The composite impervious cover for the CSO sewershed is about 62%.

As with many other combined sewer systems, the City's combined sewers are predominately vitrified clay pipe (VCP) ranging from 6" to 24" diameter, and larger pipe is constructed of brick or reinforced concrete pipe (RCP). Brick combined sewers are either circular ranging in size between 15" and 84" diameter or egg-shaped ranging in size between 16" wide by 24" high and 60" wide by 90" high, inside dimensions. About 75% of the combined sewer are reported as less than 24" diameter (or minimum internal dimension) and over 10% is greater than 42". Approximately 67% of the combined sewer system is constructed of VCP, 14% of RCP, 9% of brick masonry, and the balance of various other materials.

During wet weather conditions, a certain amount of combined sewage is conveyed through the interceptors to the Trenton Avenue PS and pumped to the JMEUC WWTF for treatment. The daily average flow rate from the TAPS is approximately 15.5 mgd based on records for the last five years. This value fluctuates from year to year based on wet weather conditions as the flow in the City's CSS is comprised of both sewage and stormwater runoff. The City's sewage is predominantly domestic, with some commercial and industrial wastewater contribution.

By the current agreement with the JMEUC, the maximum average daily flow that can be discharged from the Trenton Avenue PS to the JMEUC WWTF is 18 million gallons per day (mgd) and the maximum peak flow is limited to 36 mgd. (The existing ultimate pumping capacity (all pumps running) of Trenton Avenue PS is estimated to be about 55 mgd.) Combined sewage flows in excess of the allowable pumping rate and the conveyance and storage capacities are diverted at regulator structures to the permitted CSO outfalls to the Elizabeth River, Arthur Kill and Newark Bay. Each CSO outfall is equipped with an overflow control facility to collect solids and floatables that would otherwise be discharged to the receiving waters.

Based on population estimates and hydraulic model results, the estimated average dry weather flow from the Elmora sewer area is around two mgd, a significant majority of which drains directly to the Original JMEUC Trunk Sewer. Along with the combined sewer area in the City of Elizabeth, there are also NJDOT catch basin connections to the Original Trunk Sewer which collect storm water along Elmora Avenue and Bayway between Westfield Avenue and Brunswick Avenue.

2.4.1 Flow from Neighboring Communities

As part of the system characterization process, the City reviewed record documents and corresponded with adjacent municipalities to identify the location and flow contribution of inter-municipal sewer connections. Except of the City of Newark, the neighboring communities are reported to have separate sanitary and stormwater collection systems. From this investigation, the major external connection to the City's CSS consists of a 42" diameter storm sewer from the Borough of Roselle Park connecting to the City's combined sewer system in Park Avenue along the municipal boundary at Galloping Hill Road. The other identified inter-municipal connections were found to be associated with small sewers of short lengths, following local topography, and of limited tributary flow.

The 42" Roselle Park storm sewer connection is noted on Figure 2-6 and contributes significant wet weather flow to the upstream end of a large CSO basin. Furthermore, its impact on localized street flooding at the intersection of Park Avenue and Glenwood Road was recognized in a prior study by the City. Roselle Park has delineated a 120-acre drainage area as being tributary to the 42" storm sewer connection to the City combined sewer system. The City has been monitoring the flow from the connection on a continuous basis since December 2017 and is evaluating the conditions for an inter-municipal agreement with Roselle Park for connection at Park Avenue, including a cost structure for a user charges and future construction and capital expenditures. The contributing drainage area to the 42" Roselle Park storm sewer connection has been incorporated into the hydraulic computer model for the Elizabeth CSS.

2.4.2 Permitted CSO Discharge Locations

The City's NJPDES CSO Permit currently includes 29 CSO discharge points:

- 4 CSO outfalls discharge to Newark Bay (2 via the Great Ditch, 1 via the Peripheral Ditch, and 1 directly to the bay);
- 4 CSO outfalls discharge to the Arthur Kill; and
- 21 CSO outfalls discharge to the Elizabeth River.

Several CSO outfalls have been eliminated over the years through outfall consolidation and sewer separation work. Accordingly, the remaining number of CSO outfalls is significantly less than the highest outfall discharge serial number assigned by the CSO Permit. The permitted CSO outfall discharge points are listed in Table 2-3 and shown on Figure 2-6 through Figure 2-8.

Table 2-3: List of CSO Outfall Discharges and Locations

Outfall No.	Outfall Name	Discharge Coordinates		Receiving Stream
		Latitude (degree)	Longitude (degree)	
001A	Airport South Area	40.680754	-74.191792	Peripheral Ditch to Newark Bay
002A	Dowd Avenue	40.671438	-74.188015	Great Ditch to Newark Bay
003A *	Westfield Avenue & Magie Avenue	40.667910	-74.219405	Elizabeth River
005A	Westfield Avenue	40.667885	-74.219236	Elizabeth River
008A	West Grand Street/Price Street	40.666300	-74.218607	Elizabeth River
010A	Murray Street/Cherry Street	40.663122	-74.218836	Elizabeth River
012A	Rahway Avenue	40.661474	-74.217542	Elizabeth River
013A	Rahway Avenue/Burnet Street	40.661598	-74.217420	Elizabeth River
014A	Broad Street Rahway Avenue	40.661050	-74.215169	Elizabeth River
016A	Edgar Road/Pearl Street	40.660860	-74.216519	Elizabeth River
021A *	Spring Street/Third Avenue	40.659355	-74.208766	Elizabeth River
022A	South Street	40.657827	-74.210393	Elizabeth River
026A	John Street	40.654472	-74.208411	Elizabeth River
027A	Summer Street/Arnett Street	40.650336	-74.209934	Elizabeth River
028A	Summer Street/Arnett Street	40.649784	-74.209929	Elizabeth River
029A	South Front Street	40.644317	-74.190050	Elizabeth River
030A *	Front Street/East Jersey Street	40.646520	-74.186165	Arthur Kill
031A	Front Street/Livingston Street	40.646811	-74.185418	Arthur Kill
032A	Front Street/Magnolia Avenue	40.647672	-74.181477	Arthur Kill
034A	Atalanta Place	40.651665	-74.171288	Newark Bay
035A	South Front Street/Third Avenue	40.643376	-74.195218	Elizabeth River
036A *	Orchard Street/Dod Court	40.671036	-74.219232	Elizabeth River
037A	Bayway/South Front Street	40.635265	-74.198874	Arthur Kill
038A *	Third Avenue	40.647386	-74.204464	Elizabeth River

Outfall No.	Outfall Name	Discharge Coordinates		Receiving Stream
		Latitude (degree)	Longitude (degree)	
039A *	Trumbull Street, Fourth Street	40.663314	-74.180887	Great Ditch to Newark Bay
040A	Pulaski Street/Clifton Street	40.646607	-74.208485	Elizabeth River
041A *	Morris Avenue/Sayre Street	40.669631	-74.219365	Elizabeth River
042A	Bridge Street/Elizabeth River	40.661052	-74.211343	Elizabeth River
043A *	Army Corps Flood Control Structure	40.643666	-74.195516	Elizabeth River via ditch

The permitted CSO outfalls are classified as either primary or relief outfalls, with relief outfalls being designated where the sewershed has an interconnection to another downstream sewershed with a subsequent regulator and outfall network. The relief outfalls (annotated with an asterisk in Table 2-3) and the associated sewersheds are as follows:

- Relief Outfall 003A, Westfield Avenue and Magie Avenue, relieving Relief Outfall 041A and Primary Outfall 005A. (Westerly Interceptor.)
- Relief Outfall 021A, Spring Street / Third Avenue, relieving Primary Outfall 022A. (Westerly Interceptor.)
- Relief Outfall 030A, Front Street/East Jersey Street, relieving Primary Outfall 029A. (Easterly Interceptor.)
- Relief Outfall 036A, Orchard Street / Dod Court, relieving Primary Outfall 005A. (Westerly Interceptor.)
- Relief Outfall 038A, Third Avenue, relieving Primary Outfall 035A. (Easterly Interceptor.)
- Relief Outfall 039A, Trumbull Street / Fourth Street, relieving Primary Outfall 034A. (Easterly Interceptor.)
- Relief Outfall 041A, Morris Avenue / Sayre Street, relieving Primary Outfall 005A (Westerly Interceptor.)
- Relief Outfall 043A, Army Corps Flood Control Structure, relieves Primary Outfall 035A (Easterly Interceptor.)

2.4.3 Overflow Regulators

There are currently 38 overflow regulators and diversion structures in the existing system that discharge through the 29 CSO outfalls, as indicated in Table 2-4. Each regulator is associated with a CSO outfall and either the Easterly or Westerly Interceptor sewer service areas. The size of the tributary area to the CSO regulators are also noted in the table.

Table 2-4: List of Overflow Regulators

Outfall No.	Interceptor Service Area	Regulator ID	Location / Street Name	Coordinates		Area (acres)
				Latitude (degree)	Longitude (degree)	
001A	Easterly	R001	Route 1&9 N Ramp from Route 81 West	40.680809	-74.192651	438.9
002A	Easterly	R002	Division St at Fairmount Ave	40.670950	-74.193386	222.9
003A *	Westerly	R003A *	Westfield Ave at Magie Ave and Orchard St	40.666448	-74.228955	220.4
		R003B *	Grove St at W. Grand St	40.664905	-74.229390	118.8

Outfall No.	Interceptor Service Area	Regulator ID	Location / Street Name	Coordinates		Area (acres)
				Latitude (degree)	Longitude (degree)	
005A	Westerly	R005	Westfield Ave at Union St	40.668616	-74.217710	189.2
008A	Westerly	R008	W. Grand St, west of Elizabeth R	40.666282	-74.218750	23.1
010A	Westerly	R010	Murray St at Cherry St	40.662981	-74.219820	76.3
012A	Westerly	R012A	Rahway Ave, east of Elizabeth River	40.661619	-74.217280	See R012B
		R012B	Rahway Ave, east of Elizabeth River	40.661681	-74.216842	9.2
013A	Westerly	R011	Rahway Ave at Burnet St	40.661488	-74.218185	34.1
		R013	Burnet St, south of Rahway Ave	40.661025	-74.218373	23.8
014A	Westerly	R014	South Broad Street at Rahway Ave	40.662033	-74.215064	12.4
016A	Westerly	R016	Pearl St at Washington Ave	40.659955	-74.217582	38.1
021A *	Westerly	R021 *	Third Ave, north of South Reid St	40.659022	-74.207321	2.8
022A	Westerly	R022	South St at Fourth Ave	40.658011	-74.209023	168.3
026A	Westerly	R026	John St at Elizabeth River	40.654604	-74.208163	110.7
027A & 028A	Westerly	R027/028	Summer St, west of Clarkson Ave	40.650097	-74.211322	216.2
029A	Easterly	R029	S. Front St at Elizabeth Ave, Veterans Memorial Waterfront Park	40.644955	-74.189513	76.3
030A *	Easterly	R030 *	Front St, west of E. Jersey Ave	40.646941	-74.186849	19.2
031A	Easterly	R031	Front St at Livingston St	40.647499	-74.186058	59.5
032A	Easterly	R032	Front St at Magnolia Ave	40.649095	-74.182773	65.0
034A	Easterly	R034A	Esmt on 1 Atlanta Plz, east of Puleo Pl	40.652154	-74.171752	102.9
		R034B *	Trumbull St at Second St	40.655549	-74.179215	75.5
035A	Easterly	R035	S. First St at Third Ave	40.643767	-74.195509	120.0
036A *	Westerly	R036A *	N. Broad St at Salem Ave and Pingry Pl	40.675879	-74.213348	See R036B
		R036B *	N. Broad St, north of Pingry Pl	40.676359	-74.213390	209.5
037A	Easterly	R037A	Bayway, south of S. Front St	40.636352	-74.200433	16.2
		R037B	Bayway, north of S. Front St	40.637085	-74.201346	70.2
038A *	Easterly	R038A *	Third Ave, south of Atlantic St	40.649505	-74.200874	58.0
		R038B *	LT Glenn Zamorski Dr at Second St	40.649533	-74.198624	5.8
039A *	Easterly	R039 *	Trumbull St at Fourth Ave	40.658062	-74.185464	244.9

Outfall No.	Interceptor Service Area	Regulator ID	Location / Street Name	Coordinates Latitude (degree)	Longitude (degree)	Area (acres)
040A	Westerly	R040	Pulaski St, west of Clifton St	40.646155	-74.208854	34.9
041A *	Westerly	R041 *	Morris Ave, north of Elizabeth R	40.670003	-74.219117	238.1
042A	Westerly	R042A	Elizabeth Ave at Bridge St	40.661856	-74.211366	23.7
		R042B	E. Jersey St at Winfield Scott Plz	40.664057	-74.211256	25.1
		R042C *	Jefferson Ave at Chestnut St	40.668196	-74.210906	109.9
		R042D *	Winfield Scott Park, north of Elizabeth Ave	40.662288	-74.211381	32.8
043A *	Easterly	R043 *	S. First St at Third Ave	40.643684	-74.195507	See R035

Some regulators serve as relief diversion structures and are connected to sewersheds for other regulators. These relief regulators are indicated with an asterisk in Table 2-4. Key observations associated with the overflow regulators are summarized below:

- Regulators R003A, R003B, and R041 are connected, with the DWF pipe from R003B flowing to R003A, which then in turn connects to the trunk sewer to Regulator R041. Regulators R036A and R036B contribute flow to a separate trunk sewer collecting flow from the Regulator R005 sewershed, which then merges with the trunk sewer from R041 before connecting to R005 and subsequently to the Westerly Interceptor.
- Dry weather flow from Regulator R021 is tributary to the Regulator R022 sewershed.
- Outfalls 027A and 028A have a common tributary area and regulator structure. Regulator R027/028 has two (2) overflow outlets, one that leads to each outfall pipe. The outfall pipes are also interconnected downstream of the regulator.
- Dry weather flow from Regulator R030 connects downstream to the Regulator R029 sewershed.
- Regulators R035, R038A, R038B, and R043 are interconnected, with Regulator R035 having the downstream DWF pipe connection to the Easterly Interceptor. The DWF pipes from Regulators R038A and R038B connect to the trunk sewer within Third Avenue leading to R035, while the R038A and R038B overflow pipes merge prior to discharging through CSO Outfall 038A. Regulator R043 is an emergency relief overflow located on the CSO 035A Outfall.
- Regulator R039 is a relief overflow diversion situated on a trunk sewer within Trumbull Street connecting Regulator R034B. Regulator R034B has a DWF pipe connection to the Easterly Interceptor, while the wet weather flow pipe continues as the trunk sewer and the incoming pipe to Regulator R034A, collecting flow from the R034A drainage basin. As such, R034B is an internal diversion to the interceptor and does not have a designated outfall.
- Regulator R042D provides a relief overflow diversion for the sewershed associated with Regulator R042A, with the DWF pipe continuing through R042D to R042A and then connecting downstream to the Westerly Interceptor. The sewersheds for Regulators R042B and R042C are also interconnected, with the DWF pipe from R042C continuing as a trunk sewer to R042B, from which a dry weather branch sewer extends southerly to the Westerly Interceptor, collecting sanitary flow from lateral connections along the run.

2.4.4 City Interceptor and Trunk Sewers

The City's sewer system tributary to the TAPS is served by the Easterly and Westerly Interceptors. Each interceptor enters the pump station through a 60" diameter RCP. The City interceptors intercept various local trunk and branch sewers. Table 2-5 summarizes certain data for the City interceptors, interceptor branches, and major trunk sewers. The location of the interceptor and main trunk sewers are also noted on Figure 2-6 through Figure 2-8.

Table 2-5: City Interceptors and Major Trunk Sewers

Interceptor Name Branch Interceptor Name Trunk Sewer Name	Sewer Length	Downstream Pipe Size	Downstream Pipe Material	Tributary Length
	(miles)	(inches)	(-)	(miles)
Easterly Interceptor	4.30	60	RCP	58.7
Division Street Branch	0.27	24	RCP	-
East Side Industrial Branch	0.56	18	PCCP	1.43
Bayway Branch	0.93	30	VCP	1.56
Alina St / Van Buren St / North Ave Trunk	1.50	48	RCP	14.1
Fairmount Ave Trunk	0.40	48	RCP	5.56
Trumbull St / Sixth St Trunk	1.48	48 x 72	Brick Egg	12.7
Magnolia Ave Trunk	0.26	30 x 45	Brick Egg	3.00
Livingston St Trunk	0.43	36 x 54	Brick Egg	2.75
Front St Trunk	1.32	44 x 63	Brick Egg	3.41
Third Ave Trunk	0.57	48	RCP	5.09
Bayway Trunk	0.26	72	Brick	1.07
Westerly Interceptor	2.30	60	RCP	78.9
W Jersey St / W Grand St Branch	0.16	12	VCP	1.04
Rahway Ave / Cherry St Branch	0.25	12	VCP	3.68
Pearl St / Burnet St Branch	0.50	12	VCP	1.97
South St Branch	0.08	15	VCP	6.76
Palmer St / John St Branch	0.26	20	VCP	4.54
Westfield Ave / Park Ave Trunk	1.23	54	CCFRPM	8.00
Grove St / Pennington St / Elmora Ave Trunk	0.86	48 x 72	Brick	4.97
Magie Ave Trunk	0.26	18	VCP	0.392
Orchard St / Morris Ave Trunk	0.78	72	RCP	23.4
Union Ave / Newark Ave Trunk	1.24	48 x 72	Brick Egg	15.4
Bridge St / Jefferson Ave Trunk	0.79	42 x 63	Brick Egg	5.22
Reid St / East Grand St Trunk	0.86	48 x 72	Brick Egg	6.64
John St / Niles St Trunk	0.52	36 x 54	Brick Egg	4.28
Summer St / South Elmora Ave Trunk	0.68	60	RCP	7.34

Abbreviations: Brick Egg = Egg-shaped brick masonry sewer; CCFRPM = centrifugally cast fiberglass reinforced polymer mortar; PCCP = pre-stressed concrete cylinder pipe; RCP = reinforced concrete pipe; VCP = vitrified clay pipe.

The Easterly Interceptor is approximately 23,400 feet long, ranges in size from 33" to 60" diameter, and is constructed of reinforced concrete pipe. It starts in the northern portion of the City at Regulator R001, and then flows southeasterly along NJ Route 81 and Dowd Avenue, across the New Jersey Turnpike and

Conrail lines, and through easements to Trumbull Street at Second Street and to Front Street at Port Avenue. The interceptor continues southwesterly along Front Street, northerly along Elizabeth Avenue, and southwesterly again along South First Street. The interceptor then heads northwesterly along the Elizabeth River to the Trenton Avenue Pumping Station. The 60" RCP interceptor reduces to twin 36" ductile iron pipes where it crosses beneath the Elizabeth River near the end of South Second Street.

The Easterly Interceptor receives flows from a sewage service area of 3,690 acres, including 1,570 acres of combined sewers associated with Regulators R001, R002, R029, R030, R031, R032, R034A and B, R035, R037A and B, R038A and B, and R039. It also receives flow from the largest separate sewer areas of the City associated with the Kapkowski Road Pumping Station and along Dowd Avenue. The system tributary to the Easterly Interceptor includes approximately 58.7 miles of sewer main, 2,350 manholes, and 1,070 storm inlets and catch basins.

The Westerly Interceptor serves the northern, central, and western parts of the City, with the main branch beginning at the Union Street, Morris Avenue, and Westfield Avenue intersection, connecting to Regulator R005. The Westerly Interceptor flows southerly along Union Street to West Jersey Street, easterly across the Amtrak railroad lines to Elizabethtown Plaza, and then southerly to Rahway Avenue. The interceptor continues easterly along Rahway Avenue and Elizabeth Avenue to Bridge Street, and then runs southerly across the Elizabeth River to Pearl Street. It then flows southerly along South Pearl Street, through Grove Street to Clarkson Avenue. From Clarkson Avenue at Britton Street, the Westerly Interceptor is mostly routed along the western bank of the Elizabeth River to the Trenton Avenue Pumping Station.

The Westerly Interceptor receives flows from a sewer service area of 2,140 acres, including 1,890 acres of combined sewer system areas associated with Regulators R003A, R003B, R005, R008, R010, R012A, R011, R013, R014, R016, R021, R022, R026, R027/028, R027/028, R036A, R040, R041, and R042A, B, C and D. Approximately 78.9 miles of sewer main, 3,330 manholes, and 1,270 storm inlets and catch basins are estimated to contribute flow to the Westerly Interceptor.

Three (3) branch interceptors, varying in length from 1,400 feet to 4,800 feet, are associated with the Easterly Interceptor and five (5) branch interceptors, varying from 600 feet to 2,600 feet, connect the Westerly Interceptor to various upstream regulators. Seventeen (17) trunk sewers with a total length of about 13.3 miles are listed in Table 2-5 for the City's combined sewer system. Each trunk sewer receives and conveys flows from a relatively large area and has substantial branch sewer connections. Eight (8) trunk sewers contribute flow to the Easterly Interceptor and nine (9) trunk sewers flow to the Westerly Interceptor. Many trunk sewers are egg-shaped or circular brick sewers, ranging in size from 30" wide by 45" high to 60" wide by 90" high.

2.4.5 Pumping Stations

There are 3 pumping stations within the City that handle dry weather sanitary sewage: the Trenton Avenue Pumping Station (TAPS) located at Trenton Avenue and the Elizabeth River; the Kapkowski Road Pumping Station located at the intersection of Kapkowski Road and North Avenue East; and the West Jersey Street Pumping Station located on West Jersey Street between Cherry Street and Price Street. The Kapkowski Road and West Jersey Street pumping stations receive flow from separate sewer systems, but discharge into the combined sewer system for treatment. As previously noted, TAPS is the main pumping station situated at the downstream point of the sewer system and conveys the majority of flows from the City to the JMEUC WWTF, including the tributary flows from the Kapkowski Road and West Jersey Street pumping stations.

Additionally, there are 6 stormwater pumping stations (SWPS) within the City: Arch Stormwater Pumping Station, Verona-Gebhardt Stormwater Pumping Station, and four stations constructed by the Army Corps of Engineers as part of the Elizabeth River Flood Control Project. Due to connections with CSO outfalls, certain stormwater pumping stations have been incorporated in the collection system hydraulic model to characterize the potential influence on the combined sewer system.

2.5 Significant Indirect Users

The NJPDES CSO Permit requires that impacts from significant indirect users (SIU), as defined in the NJPDES Rules (N.J.A.C. 7:14A-1), contributing to the CSOs are minimized. Based on the loading and toxicity of SIU contributions, each SIU is required to incorporate a level of pretreatment prior to discharge to the sewer system. JMEUC monitors SIUs for compliance with pretreatment requirements.

In the Elizabeth System Characterization Report, 17 locations were included on the SIU list. However, further coordination with the JMEUC industrial pretreatment unit determined that the previously provided list covered all industrial users in the City, without limitation to the SIU classification. A facility is classified as a SIU if the permitted discharge is greater than 25,000 gallons per day (gpd) or the equivalent loading for a specific pollutant, or if the facility falls under a federal categorical group. This additional information indicates that eight (8) facilities located in Elizabeth are actually classified as Significant Indirect Users. These facilities are listed in Table 2-6.

Table 2-6: Significant Indirect Users

ID	Name	CSO Basin	Street Address	Flow (mgd)	SIC Code	Pre-treatment
1	Actavis Elizabeth LLC.	None	200 Elmora Avenue	0.054	Manufacturer of Generic Pharmaceuticals - 2834	Yes
2	Duro Bag Manufacturing Company	None	750 Dowd Avenue	0.018	Manufacturing of Paper Bags - 2674	No
3	LORCO Petroleum Services	None	450 S. Front Street	0.063	CWT, Oil Treatment & Recovery - 2992	Yes
4	Mastercraft Metal Finishing	039	801 Magnolia Avenue	0.00008	Manufacturing of Phonographic Masters - 3471	Yes
5	Michael Foods, Inc. - North Ave	None	877 North Avenue	0.109	Egg Processing - 2015	Yes
6	Michael Foods, Inc. - Jersey Pride	039	1 Papetti Plaza	0.110	Egg Processing - 2015	Yes
7	Superior Powder Coating, Inc.	None	600 Progress Street	0.014	Powder Coating of Metal Parts - 3399	Yes
8	Wakefern Food Corporation	002	600 York Street	0.013	Food Warehousing & Distribution - 5140	Yes

Only three (3) of these SIU facilities contribute flow to a sewer tributary to a CSO regulator / diversion structure:

- Mastercraft Metal Finishing, located in CSO Basin 039. The facility electroplates vinyl record masters. The vinyl record masters are silver and nickel plated to form record stampers to make the production vinyl records. The process wastewater flow rate is approximately 80 gpd. Pretreatment consists of chemical precipitation, filtration, neutralization and pH correction.
- Michael Foods, Inc. - Jersey Pride, located in CSO Basin 039. The egg processing performed at the site includes liquid-egg pasteurization, homogenization, storage, and distribution and hard

cook eggs washing, boiling, peeling, and packaging. The process wastewater flow rate is approximately 0.11 mgd. Pretreatment includes flow equalization, settled solids removal, neutralization and pH correction.

- Wakefern Food Corporation, located in CSO Basin 002. The facility warehouses and distributes various food items to supermarkets and seafood cleaning/packaging. The reported average daily process wastewater flow rate is approximately 13,300 gpd. Pretreatment includes flow equalization, sedimentation, grease/sludge removal and pH neutralization.

An analysis of the discharge from these three (3) SIUs for the Typical Year wet weather overflow volumes to evaluate the potential impacts on water quality is provided in Section 7.8.

Section 3
CSO Receiving Waters and Control Objectives

The intent of CSO controls is to improve the water quality of receiving waters by meeting the water-quality based requirements of the Clean Water Act (CWA). In this section, the City of Elizabeth CSO receiving waters are discussed and the CSO control objectives and approaches are presented. Grouping of CSO outfalls by receiving waterbody and by hydraulic connectivity are identified. Information on the existing system CSO performance, sensitive area considerations, and public involvement is also included.

3.1 CSO Receiving Waters

The City of Elizabeth CSO receiving waters are the Elizabeth River, the Arthur Kill and Newark Bay, with the Peripheral Ditch and Great Ditch consisting of stormwater conveyance ditches tributary to Newark Bay noted in NJPDES CSO Permit No. NJ0108782 as receiving streams.

These receiving waters are located within Watershed Management Area (WMA) 7 – Arthur Kill as designated by NJDEP. According to the State of New Jersey "2014 Hazard Mitigation Plan: Appendix P Watersheds" document, water quality in WMA 7 is reported as being reflective of urbanized streams and past industrial uses. Key issues in this watershed are indicated as including point and nonpoint source pollution, habitat destruction, and flood control. Sources of nonpoint pollution can involve construction activities, storm sewers, and urban surface and road runoff and these conditions are noted as having contributed to high stream temperatures, sediment and nutrient loadings, periodic low dissolved oxygen levels and fish kills.

NJDEP has established Surface Water Quality Standards (SWQS), which outline designated uses for the state's surface waters, classify those waters based on their designated uses, and establish water quality criteria for each waterbody classification. The standards are based on both bacterial and physical/chemical standards such as levels of dissolved oxygen, turbidity, nutrients, and pH. 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 receiving waters relevant to the City of Elizabeth are FW2-NT (freshwaters category 2, non-trout supporting) and SE3 (saline estuarine). The NJDEP surface water bacterial quality criteria and designated uses for these waters are shown in Table 3-1.

Table 3-1: Surface Water Quality Standards

Classification	Designated Use(s)	Indicator Bacteria	Criteria (per 100 mL)
FW2-NT (Fresh Water Non Trout)	1. Maintenance, migration and propagation of the natural and established biota; 2. Primary contact recreation; 3. Industrial and agricultural water supply; 4. Public potable water supply after conventional filtration treatment (a series of processes including filtration, flocculation, coagulation, and sedimentation, resulting in substantial particulate)	E. Coli	126 geometric mean, 235 single sample maximum
SE3 (Saline Estuarine Water)	1. Secondary contact recreation; 2. Maintenance and migration of fish populations;	Fecal Coliform	1500 geometric mean

Classification	Designated Use(s)	Indicator Bacteria	Criteria (per 100 mL)
	3. Migration of diadromous fish; 4. Maintenance of wildlife; 5. Any other reasonable uses.		

Under the SWQS, the Arthur Kill and Newark Bay (including the associated Peripheral Ditch and Great Ditch) are classified by NJDEP as SE3 waters, with 4 CSO outfalls discharging to each. The Elizabeth River is divided into two reaches for SWQS classification based on salinity content. The lower reach, from the Broad Street bridge to the mouth, is classified as SE3 and 11 CSO outfalls discharge to this section. The upper reach of the Elizabeth River, from the source to the Bridge Street bridge, is classified as FW2-NT and 10 outfalls discharge to this section. The outfalls can be grouped according to the receiving waters and water quality requirements as listed in Table 3-2 and mapped in Figure 3-1.

Table 3-2: City of Elizabeth Receiving Waters

Waterbody	Reach	Water Quality Classification	Outfalls Discharging in this Reach
Elizabeth River	North of Broad St. bridge	FW2-NT	003A, 005A, 008A, 010A, 012A, 013A, 014A, 016A, 036A, 041A
	Broad St. bridge to mouth	SE3	021A, 022A, 026A, 027A, 028A, 029A, 035A, 038A, 040A, 042A, 043A
Arthur Kill	n/a	SE3	030A, 031A, 032A, 037A
Newark Bay and ditches	n/a	SE3	001A, 002A, 034A, 039A

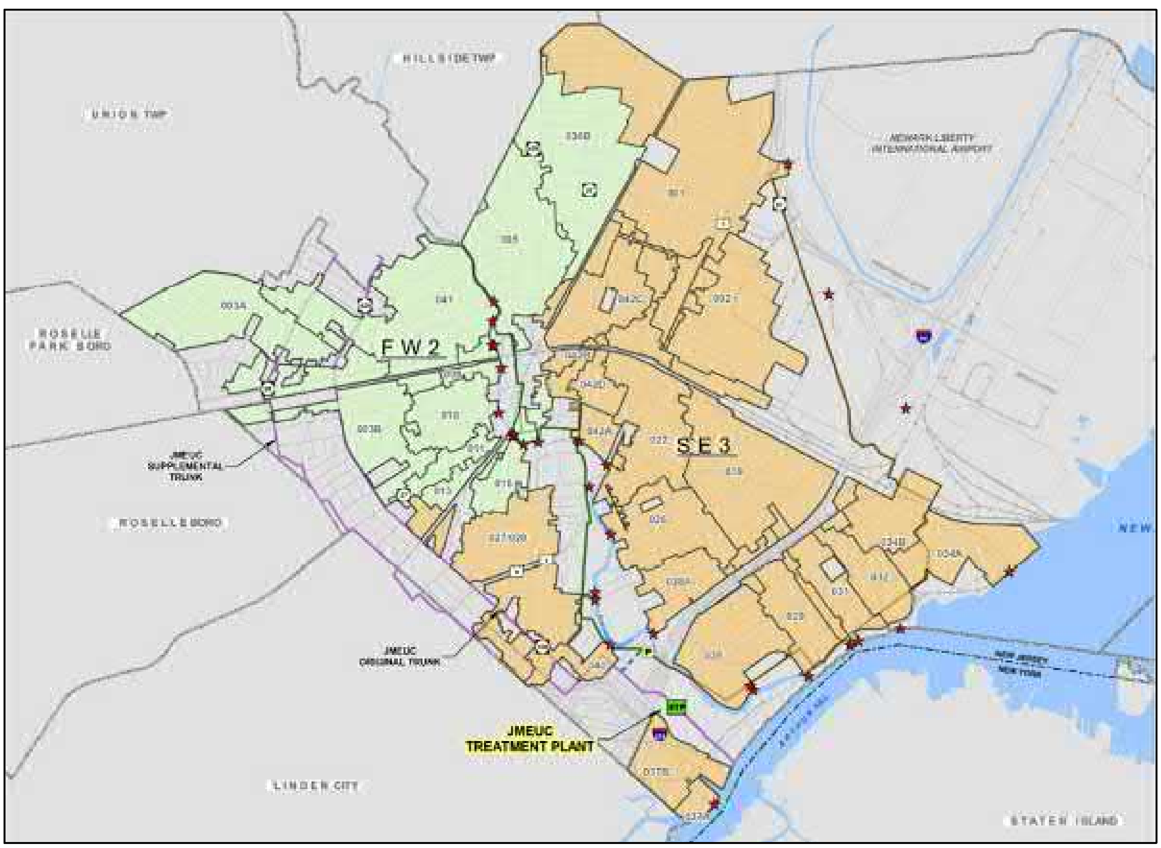


Figure 3-1: Outfall Groupings by Water Quality Classification

The outfalls can also be grouped according to hydraulic connectivity, size, and proximity, as shown in Table 3-3 and Figure 3-2 below. The outfalls are grouped in terms of those tributary to the Westerly Interceptor and the Easterly Interceptor, as well as their geographic proximity to each other and size.

Table 3-3: Outfall Groupings by Interceptor

Overall Grouping	Sub-Grouping	Outfalls
Area A – Easterly Interceptor	A1	001A, 002A
	A2	034A, 039A
	A3	029A, 030A, 031A, 032A
	A4	035A /043A, 038A
	A5	037A
Area B – Westerly Interceptor	B1	003A, 005A, 036A, 041A
	B2	008A, 010A, 013A, 016A
	B3	012A, 014A
	B4	042
	B5	021A, 022A, 026A
	B6	027A, 028A, 040A

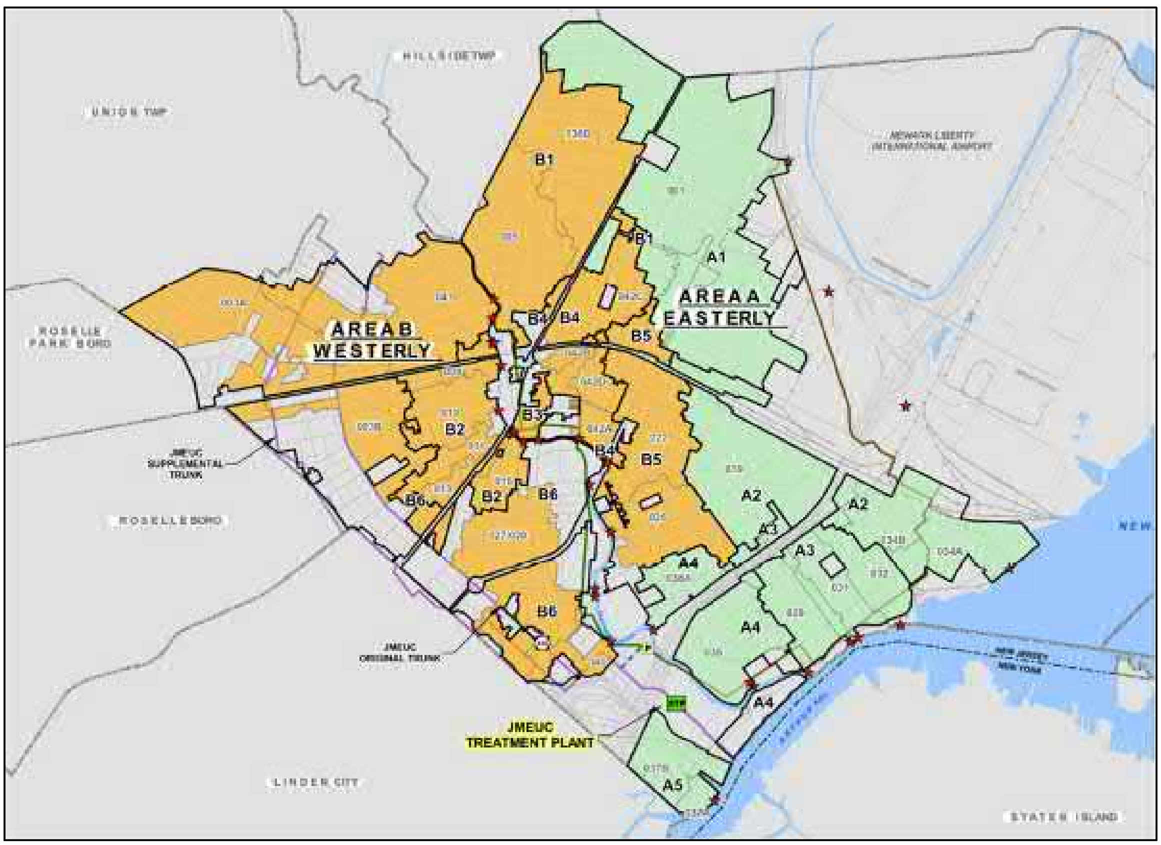


Figure 3-2: Outfall Groupings by Hydraulic Connectivity and Proximity

Relative to pollution from CSO discharges, the three pollutants of concern (POCs) that have been identified for the receiving waters of the NJ CSO Group, which includes the City and JMEUC, are fecal coliform, E. coli, and Enterococcus. The concentrations of these identified POCs are parameters typically associated with CSO discharges. The City of Elizabeth and JMEUC have collaborated with the NJ CSO Group in preparing a Baseline Compliance Monitoring Program (BCMP) that was approved by NJDEP. The BCMP includes a regional ambient water quality sampling and testing program as well as pathogen water quality modeling. The impact of CSO discharges on the receiving waters for the POCs are being further investigated through the receiving water quality monitoring and modeling program with the NJ CSO Group.

The 2014 New Jersey Integrated Water Quality Monitoring and Assessment Report 303(d) list is a catalog of the impaired waters throughout the state of New Jersey. The Elizabeth River below the Elizabeth City corporate boundary appears on the 303(d) list as being impaired for the following pollutants: arsenic, benzo(a)pyrene (PAHs), chlordane in fish tissue, DDT and its metabolites in fish tissue, dieldrin, dioxin, heptachlor epoxide, hexachlorobenzene, lead, mercury in fish tissue, PCB in fish tissue, pH, phosphorus (total), total dissolved solids (TDS). These contaminants primarily impact the designated use of fish consumption for SE3 and FW2 classified waters.

The Interstate Environmental Commission (IEC) is an air and water pollution control agency that serves the Interstate Environmental District within the states of New York, New Jersey, and Connecticut. The Commission serves to coordinate interstate and region-wide water quality programs for the enhancement of environmental conditions in the Tri-State area. The IEC classifies the waters of the Arthur Kill north of Outerbridge Crossing and those of the Newark Bay as Class B-2, meaning that they are suitable for passage of anadromous fish and for the maintenance of fish life in a manner consistent with the criteria established by the general regulations, thus require minimum dissolved oxygen of 3 mg/L.

3.2 CSO Control Objectives

Given the pollutants of concern for the receiving waters, the primary objectives of the CSO long term control program are the reduction of pathogens and CSO volume to receiving waters. 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 sufficient to meet the water-quality based requirements of the CWA. Per the National CSO Control Policy, the LTCP can adopt either the “Presumption” Approach or the “Demonstration” Approach for this purpose.

The “Presumption" Approach refers to a program that is presumed to achieve attainment of water quality standards (WQS). The Presumption Approach requires that the CSO control program meets any of the following three (3) criteria, provided that the permitting authority (i.e., NJDEP), determines that the approach is reasonable in light of the data and analysis conducted in the characterization, monitoring, and modeling of the system and in consideration of sensitive areas:

1. 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.
2. 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.
3. 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 paragraph 2 above.

The “Demonstration” Approach refers to a program that uses a receiving water model to demonstrate compliance with each of the following criteria from the National CSO Control Policy:

1. 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.
2. 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.
3. The planned control program will provide the maximum pollution reduction benefits reasonably attainable.
4. 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.

Section 3.2.1.1 of the EPA document titled “Combined Sewer Overflows: Guidance for Long-Term Control Plan” states:

> *The demonstration approach is particularly appropriate where attainment of WQS cannot be achieved through CSO control alone, due to the impacts of non-CSO sources of pollution. In such cases, an appropriate level of CSO control cannot be dictated directly by existing WQS but must be defined based on water quality data, system performance modeling, and economic factors.*

As discussed with NJDEP regarding the selection of a CSO control approach for each group of hydraulically connected CSOs, each CSO outfall is being analyzed under a range of CSO control levels at this time, including 0, 4, 8, 12, and 20 overflows per year. Various CSO technologies to provide varying levels of control (i.e., 0, 4, 8, 12, and 20 uncontrolled overflows per year, or 85% volume capture of annual wet weather combined sewer flow) have been evaluated for effectiveness. These evaluations address the Presumption Approach requirements, and the range of control levels will enable cost/performance considerations to be incorporated into the final selection of controls.

In order to address the Demonstration Approach requirements, the Pathogen Water Quality Model simulations are being undertaken through the NJ CSO Group to understand the pollutant sources and their relative contributions for the affected study area. Use of the NJ CSO Group water quality model is expected to indicate which level of control evaluated for the CSO outfalls is needed to demonstrate attainment of WQS and designated uses of the corresponding receiving waters. The Pathogen Water Quality Model is also intended to demonstrate the maximum pollutant reduction benefits reasonably attainable for the receiving waters. Final selection of the CSO control approach (either Presumption or Demonstration) will be made when identifying the selected controls for implementation and will be presented in the subsequent Selection and Implementation of Alternatives Report.

3.3 Sensitive Areas

Consistent with the requirements of the National CSO Control Policy, the NJPDES CSO Permits stipulate 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 City 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.

The permittees included in the Sensitive Areas Report are in the process of developing a LTCP which follows the framework established by the EPA. Passaic Valley Sewerage Commission (PVSC) has prepared the Sensitive Areas Report on behalf of the permittees of the NJ CSO Group to identify all sensitive areas that are impacted by CSOs within the NJ CSO Group study area, which includes the receiving surface waters as well as the adjacent waters.

A comprehensive review to identify sensitive areas within the project area was completed. Results from this review can be found in the Identification of Sensitive Areas Report issued last revised and submitted on March 29, 2019 and associated comments and communications filed with NJDEP.

3.4 Existing Conditions System Performance

For the purposes of the Long Term Control Plan formulation, compliance with the regulatory requirements will be based on approved hydrologic, hydraulic, and water quality models to simulate existing conditions and evaluate CSO control alternatives. This includes the use of the approved Typical Year precipitation record consisting of the 2004 calendar year for the Newark Liberty International Airport rain gage. The System Characterization Reports for the City and JMEUC provide details on the development of the hydrologic and hydraulic (H&H) computer model representing the hydraulically connected sewer system and its response to wet weather events and the latest versions of the characterization reports can referenced for a comprehensive presentation of these topics.

This Development and Evaluation of Alternatives Report uses the Innovyze InfoWorks® ICM model described in the approved System Characterization Reports. The model version has been fixed to be consistent with the modeling performed under the characterization. The hydraulic and hydrologic model that was developed and calibrated during the system characterization phase was the main tool for the analysis of CSO management alternatives.

3.4.1 Combined Sewer System Typical Year Performance

With the validated collection system model and the selection of the typical hydrologic period (Typical Year), the existing CSS performance relative to volume, frequency, and duration of overflows on a system-wide, annual average basis has been simulated. The Typical Year simulation for the current existing conditions (2015 baseline) 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 1,068 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 Table 3-4. 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.

Table 3-4 :Typical Year Existing Conditions Annual Total CSO Characterization by Outfall

Outfall No.	Outfall Name	Annual Total: No. Overflow Events	Annual Total: Overflow Volume (MG)	Annual Total: Duration (hours)	Maximum: Peak Flow (mgd)
001A	Airport South Area	42	86.3	432	73.4
002A	Dowd Avenue	35	32.4	224	62.0

Outfall No.	Outfall Name	Annual Total			Maximum
		No. Overflow Events	Overflow Volume (MG)	Duration (hours)	Peak Flow (mgd)
003A	Westfield Avenue & Magie Avenue	43	60.7	285	187.6
005A	Westfield Avenue	54	96.6	593	61.3
008A	West Grand Street/Price Street	36	9.6	302	11.8
010A	Murray Street/Cherry Street	42	17.2	271	31.8
012A	Rahway Avenue	44	5.8	355	3.1
013A	Rahway Avenue/Burnet Street	42	16.9	313	20.9
014A	Broad Street Rahway Avenue	13	1.1	16	6.6
016A	Edgar Road/Pearl Street	46	16.7	367	28.1
021A	Spring Street/Third Avenue	19	1.4	32	6.4
022A	South Street	46	71.3	591	62.0
026A	John Street	53	53.2	613	54.3
027A	Summer Street/Arnett Street	25	27.7	378	42.9
028A	Summer Street/Arnett Street	35	35.4	514	57.0
029A	South Front Street	39	44.7	474	60.4
030A	Front Street/East Jersey Street	11	2.2	19	38.1
031A	Front Street/Livingston Street	35	15.4	266	35.7
032A	Front Street/Magnolia Avenue	26	7.4	83	40.7
034A	Atalanta Place	44	77.7	404	70.3
035A	South Front Street/Third Avenue	35	42.6	307	51.8
036A	Orchard Street/Dod Court	30	43.6	240	61.4
037A	Bayway/South Front Street	44	64.6	463	46.6
038A	Third Avenue	30	8.6	224	40.0
039A	Trumbull Street, Fourth Street	27	9.9	88	18.1
040A	Pulaski Street/Clifton Street	42	16.3	262	20.0
041A	Morris Avenue/Sayre Street	53	191.9	591	146.5
042A	Bridge Street/Elizabeth River	19	11.5	54	58.9
043A	Army Corps Flood Control Structure	3	0.2	1	6.2
System-wide Total		**not appl.**	**1068.5**	**not appl.**	**not appl.**

Outfall No.	Outfall Name	Annual Total			Maximum
		No. Overflow Events	Overflow Volume (MG)	Duration (hours)	Peak Flow (mgd)
System-wide Maximum		**54**	**191.9**	**613**	**187.6**

3.4.2 Percent Capture Calculations

One level of control condition that can be satisfied under the Presumption Approach of the CSO Control Policy is the "elimination or 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." The hydraulic model was used to estimate the percent capture from the CSS under existing conditions for the Typical Year. Wet weather periods for the 2004 Typical Year precipitation record were identified using a 12-hour inter-event time period and rainfall threshold of 0.1" depth in the preceding 12 hours. Approximately 1,500 hours of wet weather flow are defined with these conditions.

The percent capture was calculated using two different approaches: the first is percent capture at the inflow of the Trenton Avenue Pump Station, and the second is percent capture at the inflow of the Joint Meeting WWTF. Table 3-5 summarizes the results from the hydraulic model at the two locations under the Typical Year condition. The results were used to estimate the percent capture, as well as the estimated additional capture volume required to meet the CSO objectives for each calculation method.

Table 3-5: System-Wide Percent Capture Performance

Item	Elizabeth, TAPS	JMEUC WWTF (with upstream systems)
Total Wet Weather Flow (MG)	3,190	6,330
Wet Weather Flow Captured (MG)	2,122	5,262
CSO Volume (MG)	1,068	1,068
% Capture	66.5	83.1
Additional Volume Needed for 85% Capture (MG)	590	119

As presented in Table 3-6, the model results were also evaluated to estimate the percent capture and additional capture volume required to meet the CSO objectives at a range of control levels: 0, 4, 8, 12 and 20 overflows per year. The range of control levels as well as the capture volumes for the two locations can be considered in comparing the control strategies described in Section 7.

Table 3-6: Percent Capture for Range of Control Levels

No. Events / Yr	Additional Capture Volume (MG)	% Capture, TAPS Inflow	% Capture, JMEUC Inflow
0	1,068	100.0	100.0
4	947	96.2	98.1
8	873	93.9	96.9
12	793	91.4	95.7
20	539	83.4	91.6

3.5 Public Involvement

The CSO control alternatives described in this report have been presented to the Supplemental CSO Team for feedback. All comments received have been logged and considered in the evaluation of alternatives process. No additional areas of concern regarding the CSO control objectives have been identified by the Supplemental CSO Team. Further information on the public participation process is included in Section 8.

Section 4
Future Conditions

4.1 Background

Part IV.G.4.e of the NJPDES CSO Permit indicates that permittees are to evaluate “conditions as they are expected to exist after construction and operation of the chosen alternative(s)”. This is to ensure that future changes in the community and sewer system will not reduce the effectiveness of proposed LTCP facilities. An evaluation of anticipated population changes and potential changes to sewer flows was undertaken. Discussions were also held with the City to document planned changes to the sewer system. It has been assumed that the alternatives selected through the LTCP process will be constructed and implemented over a 30-year period. As such, Year 2050 has been selected as the future condition planning year.

It is acknowledged that sea levels have been rising and are expected to continue to rise over the life of the project, but 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 and existing tide levels would provide a conservative estimation of the CSO frequency and volume associated with an alternative.

There is much uncertainty in future projections and as the planning horizon increases, the uncertainty increases as well. As shown below, several appropriate sources can produce differing population projections. However, the goal is to establish suitable future conditions that provide a reasonable estimate of likely future conditions. 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 permittees. If such a situation arises, the LTCP may need to be updated to address the unforeseen conditions.

4.2 City of Elizabeth

4.2.1 Population Growth Projections

As noted in the System Characterization Report, the City of Elizabeth is a fully developed urbanized area, with a population density of 10,144 persons per square mile (ppsm) within approximately 12.3 square miles (7,885 acres) per the 2010 United States Census Bureau data. The western portion of the City is dominated by high density residential, with a mix of other intensity residential and commercial uses. Most commercial uses are grouped around major transportation corridors including the Amtrak / NJ Transit commuter rail line, US Route 1-9, and Broad Street. The eastern border of the City is dominated by transportation uses, such as Newark Liberty International Airport and Port Elizabeth. Some industrial areas are scattered within the northern, central, and southern portions of the City.

A comparison of the 2012 and 2007 composite land use distributions documented in the System Characterization Report found that the distribution has remained mostly unchanged for the period, with a slight decrease in the industrial and other urban categories and a small increase in open areas. The distributions for each CSO basin are also relatively unchanged, along with the impervious coverage values.

Per a 2016 master plan land use element re-examination report, the City has identified certain redevelopment study areas, including potential locations in the Midtown, Elizabethport, and North Elizabeth neighborhoods. Any proposed increase in population from residential development will likely continue as it has in the past by subdivision and infill development, and such growth can reasonably be expected to be minimal. Several population projections were sourced to select a reasonable projection for the future condition baseline.

4.2.1.1 United States Census Bureau

The United States Census Bureau is considered an authoritative source for population data. Data from the 1990, 2000 and 2010 censuses, as well as the American Community Survey (ACS) population estimates through 2017, are shown in Table 4-1.

The Year 2050 population projection was developed by determining the annualized population change from the most recent decennial census populations from 2000 and 2010, which is approximately 0.36% per year. This rate was then used to extrapolate the 2010 population of 124,969 persons to Year 2050, which equates to an estimated future baseline population of 144,240 persons.

As indicated by Table 4-1, the City's population growth has been relatively stable, with an annual population change ranging from -0.85% to 1.03% per year. Given the recent short-term and long-term trends, the City has had a low, steady rate of population growth and it is reasonable to assume it will remain as such.

Table 4-1: Census Bureau Population Data

Year	Population	Annual Percent Change (%/yr)
1990	110,002	-
2000	120,568	0.92%
2010	124,969	0.36%
2011 (ACS estimated)	123,905	-0.85%
2012 (ACS estimated)	124,795	0.72%
2013 (ACS estimated)	125,888	0.88%
2014 (ACS estimated)	126,964	0.85%
2015 (ACS estimated)	127,759	0.63%
2016 (ACS estimated)	128,042	0.22%
2017 (ACS estimated)	129,363	1.03%
2050 (extrapolated)	**144,240**	**-**

Source: https://factfinder.census.gov/faces/nav/jsf/pages/community_facts.xhtml

4.2.1.2 North Jersey Transportation Planning Authority

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

NJTPA completed its latest set of forecasts in 2017. Final forecasts were approved by the NJTPA Board on November 13, 2017 and extend to 2045. The NJTPA employs a Demographic and Employment Forecast Model (DEFM), with the following description of the model published by NJTPA:

> 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:

- o Current land use data (residential, commercial, industrial and vacant land);
- o Composite zoning estimates for density;
- o Highway and transit accessibility;
- o Historical growth; and
- o Known project developments.

The NJTPA forecasts minimal growth (annualized percent population change of 0.68%) for the City of Elizabeth. As noted above, the NJTPA forecasts only extend to 2045 and falls short of the 2050 planning period. The population forecast was extended to 2050 using the same annual growth rate projected in the report, which are summarized in Table 4-2.

Based on the historical growth rate, the NJTPA forecast produces a higher estimate than the extrapolated Census Bureau population estimate for 2050. NJTPA projections are typically based on projected build-out and as such, typically provides a more conservative estimate.

Table 4-2: NJTPA Population Projections

County	Municipality Code	Municipality Name	2015 Population	2045 Population	Annualized % Population Change 2015-2045	2050 Population Extrapolation
Union	3403921000	Elizabeth city	128,900	158,168	0.68%	163,655

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

4.2.1.3 New Jersey Department of Labor and Workforce Development

Population and labor force projections on a county-wide basis have been developed by the New Jersey Department of Labor and Workforce Development (NJ Labor Department) extending to 2034. To obtain an estimated population for 2050, it was assumed that the City of Elizabeth will grow at the same rate as Union County as a whole. Accordingly, since the City currently accounts for approximately 23% of the County population based on the 2017 US Census estimate, for this analysis, it is projected that this ratio will be similar in Year 2050.

The annualized growth rate for the period to 2034 was determined for Union County and then used to project the County's population to 2050. The current ratio of the City population to the County population was then used to obtain a 2050 population estimate for the City of Elizabeth. This yields the estimates shown in Table 4-3.

Table 4-3: New Jersey Labor Department Population Estimates

	US Census Estimate	Projections to July 1				Projected for LTCP
Location	2017	2019	2024	2029	2034	2050
Union	563,900	573,000	588,300	605,600	620,000	671,126
Elizabeth	129,363	-	-	-	-	153,961

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

The NJ Labor Department projection is lower than the NJTPA estimate, but higher than the US Census estimate. This projection assumes the proportion of the County population residing in the City will remain relatively constant, which has been the case in recent decades.

4.2.1.4 Population Summary

The Census Bureau projected population was selected as the basis of the future baseline condition, as it is consistent with recent historical growth in the municipality and is considered a realistic estimate for this long term planning horizon. The City is already fully developed, with limited available space for additional residential development, which corresponds to a relatively low future population growth rate. Furthermore, average per capita sanitary flow rates have been trending downward in general over the past decade due to the adoption of water conservation measures and low-flow plumbing fixtures. An excessively high future population projection applied at current per capita flow rate would over-estimate future base sanitary flows given that water conservation trends are expected to continue. As such, the population for the future baseline condition was increased at annual rate of 0.36% per year, or 15.4% total, from the 2010 population of 124,969 persons to an extrapolated 2050 population of 144,240 persons for the City overall.

4.2.2 Planned Projects

The City continually implements improvement projects associated with its combined and stormwater collection and conveyance systems to address asset replacement and rehabilitation, local street flooding, and hydraulic capacity upgrades. Planned sewer system improvement projects and projects currently under construction have been identified and described below. These projects have been incorporated into the sewer system hydraulic model representing future conditions.

- Trumbull Street Stormwater Control Project - installation of a 1 million gallon underground stormwater storage tank, dewatering pump station and remote level sensing system to address neighborhood flooding and reduce combined sewer overflow. Surface restoration will include a rain garden to address smaller storms and a plaza area with educational signage, lighting and walking paths for the beautification and enhancement of the neighborhood.
- South Street Flood Control Project – separation of existing combined sewers by constructing new separate storm sewers and inlets at various locations to alleviate storm related flooding that occurs in the vicinity of South Street, Fourth Avenue and South Spring Street during heavy rainfall events. Upgrades to the South Street Stormwater Pump Station, restoration of the Elizabeth River Flood Control ponding areas and outlet structures, and repairs to sewers and drainage structures located in the study area will also be implemented under the project.
- Atlantic Street Stormwater Control Project - installation of an underground wet weather detention system in excess of 1 million gallons at Atlantic Street and Third Avenue, to provide combined sewer overflow control for Basin 038 and mitigate street flooding on Third Avenue. The project also includes drainage upgrades to provide additional separate storm drains in the South Second Street area, improvements to the existing South Second Street Stormwater Pump Station, and cleaning of an existing drainage ditch to allow unimpeded flow of runoff from Geneva Street and South Second Street area to the pump station.
- Lincoln Avenue Storm Drainage Improvements Project – construction of approximately 3,000 feet of new storm sewers to replace and augment the existing drainage system on Lincoln Avenue, Melrose Terrace, Decker Avenue and Wilson Terrace. The existing storm sewers on these streets will be upsized and the stormwater runoff redirected east along Lincoln Avenue, north on Cherry Street, and across Morris Avenue to an existing large diameter storm sewer on Trotters Lane for discharge to the Elizabeth River.

4.2.3 Projected Future Wastewater Flows

Base sanitary flows (BSF) are calculated in the hydraulic model for each sub-catchment based on an assigned population and a gallons per capita per day (gpcpd) unit flow rate, per the analysis and calibration process defined in the System Characterization Report. To represent the future estimated population in the hydraulic model, additional population was added to the various sub-catchments. The population increase was distributed equally across the combined sewer area, with the population associated for each sub-catchment increased by a total of 15.4%. Table 4-4 tabulates the additional population and base sanitary flow by CSO basin for the future conditions.

Table 4-4: Additional Base Sanitary Flow by CSO Drainage Basin for Future Conditions

CSO Basin	Estimated Population 2015	Estimated Population 2050	Additional Population	Additional BSF (mgd)
001	11,020	12,718	1,698	0.084
002	4,985	5,753	768	0.049
003	8,355	9,641	1,286	0.026
005	5,043	5,819	776	0.028
008	649	749	100	0.006
010	3,215	3,710	495	0.022
012	130	150	20	0.001
013	1,852	2,137	285	0.011
014	40	46	6	0.001
016	2,066	2,384	318	0.013
021	76	88	12	0.002
022	5,234	6,040	806	0.078
026	5,445	6,283	838	0.081
027/028	7,951	9,175	1,224	0.054
029	2,120	2,446	326	0.033
030	835	964	129	0.013
031	2,583	2,981	398	0.024
032	2,060	2,377	317	0.014
034	3,716	4,288	572	0.052
035 (043)	2,144	2,475	331	0.034
036	6,533	7,538	1,005	0.094
037	1,517	1,750	233	0.024
038	2,165	2,499	334	0.033
039	8,804	10,159	1,355	0.091
040	454	524	70	0.012
041	5,810	6,705	895	0.074
042	6,066	7,001	935	0.043
Total	**100,868**	**116,400**	**15,532**	**0.997**

It is noted that the future projected flow does not include any additional flow for commercial developments. This is because the original modeled flows were developed with any contributing commercial flows accounted in the calibrated base sanitary flow component. As such, the projected flows also include commercial flow, as represented by the ratio of commercial to residential flows occurring under the existing conditions.

A comparison of the estimated CSO performance by outfall associated with the existing and future conditions for the representative hydrologic year is provided in Table 4-5. The projected future baseline conditions impact the combined sewer overflow volumes, with some factors serving to increase overflows and others to reduce overflows. The additional population and associated BSF tends to increase the annual overflow count, volume, and durations, with the largest increase in annual overflow volume in the future condition estimated to be 7.7 million gallons (MG) at Outfall 041A. However, projects currently under construction and planned will reduce the estimated future overflows, with the greatest CSO reduction being related to the Atlantic Street CSO storage facility. It is projected that this project will decrease the annual overflow volume at Outfall 038A by 8.6 MG. Overall, there is a net increase of 3.3 MG, or 0.3%, in the simulated systemwide annual overflow volume from the existing conditions to the future conditions.

Table 4-5: Annual Typical Year CSO Performance for Future Conditions

	Baseline 2015			Baseline 2050			Change		
Outfall No.	# of Events	Volume (MG)	Duration (hours)	# of Events	Volume (MG)	Duration (hours)	# of Events	Volume (MG)	Duration (hours)
001A	41	86.3	425	43	87.8	432	2	1.4	8
002A	34	32.3	224	35	32.5	224	1	0.1	0
003A	42	60.7	285	43	61.1	286	1	0.4	1
005A	52	96.6	651	53	101.3	650	1	4.6	0
008A	35	9.6	302	39	10.0	307	4	0.4	6
010A	41	17.2	271	45	17.3	285	4	0.1	14
012A	43	5.8	355	44	6.1	387	1	0.3	32
013A	41	16.8	313	44	17.0	334	3	0.2	20
014A	13	1.1	16	13	1.1	16	0	0.0	0
016A	45	16.7	367	46	16.5	368	1	-0.2	1
021A	19	1.4	32	16	1.2	32	-3	-0.3	0
022A	45	71.3	1044	46	67.7	1045	1	-3.6	1
026A	51	53.2	666	52	51.7	678	1	-1.5	12
027A	25	27.7	884	25	28.1	899	0	0.4	15
028A	34	35.4	882	37	35.9	906	3	0.5	23
029A	38	44.6	663	39	45.6	690	1	0.9	28
030A	11	2.2	19	11	2.2	19	0	0.0	0
031A	34	15.4	259	35	15.5	260	1	0.2	1
032A	26	7.4	83	26	7.4	83	0	0.0	0
034A	43	77.7	448	44	76.9	476	1	-0.7	28
035A	34	42.6	390	34	40.8	371	0	-1.8	-19
036A	30	43.6	240	30	44.1	243	0	0.5	3
037A	43	64.6	422	44	67.2	465	1	2.6	43
038A	30	8.6	231	3	0.0	7	-27	-8.6	-224
039A	27	9.9	88	27	9.1	88	0	-0.8	0
040A	41	16.3	262	44	16.7	302	3	0.4	41
041A	51	191.9	634	52	199.5	650	1	7.7	16
042A	19	11.5	54	19	11.6	55	0	0.1	0
043A	3	0.2	1	3	0.0	1	0	-0.1	-1
Total	**-**	**1068.5**		**-**	**1071.8**	**-**	**-**	**3.3**	**-**

4.3 Joint Meeting Service Area

4.3.1 Population Growth Projections

The JMEUC service area is made up of eleven member communities along with the customer communities of Livingston, Orange, New Providence, and Elizabeth. With the exception of Elizabeth, all member and customer communities are serviced by separate sanitary sewer systems, owned and operated by each individual community. The JMEUC service area is a fully developed urbanized area, with residential land use making up over two-thirds of the area served by separate sanitary sewers. Section 4.2 addressed the City of Elizabeth service area, and this section addresses the separate sanitary sewer service area.

While uncertainty inherently exists as it pertains to future projections, changes in both land use and population among the JMEUC member and customer communities is expected to be minimal between present day and Year 2050. As part of NJPDES CSO Permit requirements, the JMEUC evaluated potential changes in population and BSF of their service area through Year 2050. The evaluation was completed using several population projection sources, as described below, and coordinated with the similar evaluation completed for the City of Elizabeth.

4.3.1.1 United States Census Bureau

Using United States Census Bureau data, annualized population change (% change per year) between 2000 and 2010 was calculated for each member and customer community serviced by the JMEUC. Using the annualized percent population change and 2010 census data, population projections were extrapolated to Year 2050. Table 4-6 summarizes these population projections for 2050 on a community basis. Note that these populations are total community populations. Some communities are only partially serviced by the JMEUC.

Table 4-6: Population Projections for JMEUC Member and Customer Communities using U.S. Census Bureau Data

Community	2000 Population	2010 Population	Annual Percent Change (%/yr)	2050 Population Projection	Projected Percent Change Between 2010 and 2050
East Orange	69,824	64,270	-0.83%	46,134	-28.22%
Hillside	21,747	21,404	-0.16%	20,085	-6.16%
Irvington	60,695	53,926	-1.18%	33,603	-37.69%
Maplewood	23,868	23,867	0.00%	23,863	-0.02%
Newark	273,671	277,140	0.13%	291,461	5.17%
Roselle Park	13,281	13,297	0.01%	13,361	0.48%
South Orange	16,964	16,198	-0.46%	13,465	-16.88%
Summit	21,131	21,457	0.15%	22,812	6.32%
Union	54,405	56,642	0.40%	66,548	17.49%
West Orange	44,943	46,207	0.28%	51,629	11.73%
Millburn	19,765	20,149	0.19%	21,761	8.00%
Livingston	27,391	29,366	0.70%	38,796	32.11%
Orange	32,868	30,134	-0.86%	21,291	-29.35%
New Providence	11,907	12,171	0.22%	13,287	9.17%
Elizabeth	120,568	124,969	0.36%	144,240	15.42%
Total	**813,028**	**811,197**	**-0.02%**	**822,336**	**1.37 %**

Source: https://factfinder.census.gov/faces/nav/jsf/pages/community_facts.xhtml

Extrapolating to 2050 using U.S. Census Bureau data indicates a small overall increase in the total population for communities making up the JMEUC service area. Using this methodology, some communities are expected to see considerable population growth, while others are expected to see considerable decline. From a regional standpoint however, population growth appears to be essentially flat in communities serviced by the JMEUC.

4.3.1.2 North Jersey Transportation Planning Authority

Using NJTPA 2045 population projections, the JMEUC determined annualized percent changes in population expected for each member and customer community between 2015 and 2045. This annualized percent change was then used to extrapolate the NJTPA 2045 population projections to 2050. These calculations are summarized in Table 4-7. Note that these populations are total community populations. Some communities are only partially serviced by the JMEUC.

Table 4-7: NJTPA Population Projections for JMEUC Member and Customer Communities

County	Municipality Code	Municipality Name	2015 Population	2045 Population	Annualized % Population Change 2015-2045	2050 Population Extrapolation
Essex	3401319390	East Orange city	64,458	71,358	0.34%	72,578
Union	3403931980	Hillside township	21,843	26,058	0.59%	26,836
Essex	3401334450	Irvington township	54,118	59,045	0.29%	59,908
Essex	3401343800	Maplewood township	23,925	27,523	0.47%	28,174
Essex	3401351000	Newark city	282,102	328,809	0.51%	337,314
Union	3403964650	Roselle Park borough	13,595	15,835	0.51%	16,243
Essex	3401369274	South Orange Village township	16,245	18,650	0.46%	19,085
Union	3403971430	Summit city	21,868	26,150	0.60%	26,942
Union	3403974480	Union township	57,712	69,990	0.65%	72,276
Essex	3401379800	West Orange township	46,314	53,287	0.47%	54,547
Essex	3401346380	Millburn township	20,195	22,947	0.43%	23,441
Essex	3401340890	Livingston township	29,449	34,385	0.52%	35,284
Essex	3401313045	City of Orange township	30,200	34,720	0.47%	35,537
Union	3403951810	New Providence borough	12,414	14,799	0.59%	15,238
Union	3403921000	Elizabeth city	128,900	158,168	0.68%	163,655
NA	**NA**	**Total**	**823,338**	**961,724**	**0.52%**	**986,952**

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

The NJTPA forecasts modest annualized growth for all communities making up the JMEUC service area, with the annualized percent change in population between 2015 and 2045 projected to be 0.52%.

4.3.1.3 New Jersey Department of Labor and Workforce Development

The NJ Labor Department projects population change on a county basis. Their current projections extend out to 2034 in five-year increments. Using 2017 U.S. Census estimates and 2034 NJ Labor Department population projections, annualized growth rates for Union and Essex Counties were determined. These annualized growth rates were then applied to 2034 population projections to estimate the 2050 populations of Union and Essex Counties. It was assumed that all member and customer communities would account for the same percent of their county's total population in both 2017 and 2050. Using this assumption and methodology described above, 2050 population projections for each member and customer community were developed, as summarized in Table 4-8 and Table 4-9.

Table 4-8: New Jersey Labor Department Population Estimates

	US Census Estimate	Projections to July 1				Projected for LTCP
County	**2017**	**2019**	**2024**	**2029**	**2034**	**2050**
Union	563,900	573,000	588,300	605,600	620,000	677,889
Essex	808,300	808,300	819,100	829,800	840,100	871,171

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

Table 4-9: Community Population Estimates Using New Jersey Labor Department Projections

Community	County	2017 US Census Estimate	Percent of County's Total Population	2050 Population Estimate
East Orange	Essex	65,378	8.09%	70,463
Hillside	Union	22,069	3.91%	26,530
Irvington	Essex	54,715	6.77%	58,971
Maplewood	Essex	24,706	3.06%	26,628
Newark	Essex	285,156	35.28%	307,336
Roselle Park	Union	13,709	2.43%	16,480
South Orange	Essex	16,503	2.04%	17,787
Summit	Union	22,155	3.93%	26,633
Union	Union	58,499	10.37%	70,324
West Orange	Essex	47,609	5.89%	51,312
Millburn	Essex	20,387	2.52%	21,973
Livingston	Essex	29,955	3.71%	32,285
Orange	Essex	30,731	3.80%	33,121
New Providence	Union	12,716	2.26%	15,286
Elizabeth	Union	129,363	22.94%	155,513
Total		**833,651**		**930,643**

4.3.1.4 Population Summary

To remain consistent with the City of Elizabeth population projections, the Census Bureau projected population was selected as the basis for future baseline conditions of the member and customer communities making up the JMEUC service area. Extrapolation of Census Bureau data projects the population of the JMEUC's service area to remain more or less flat through 2050, with some communities seeing population growth, and others seeing population decline (see Table 4-6).

To model the projected 2050 population of the JMEUC service area, modeled subcatchments were assigned to the respective communities in which they are located. To calculate the projected change in population of each subcatchment, the modeled baseline population of each subcatchment was multiplied by the projected percent change in population (between 2010 and 2050) of the community in which it is located. This projected change in population was then added to the baseline population to arrive at a projected 2050 population for each subcatchment. Table 4-10 summarizes these calculations.

Table 4-10: Modeled 2050 Population by Modeled JMEUC Subcatchment

Subcatchment	Community	Baseline Conditions Population	Projected Change in Population between Baseline and Future Baseline Conditions	Baseline Future Conditions Population
Elmora	Elizabeth	9,250	1,426	10,676
Meter #05A	Union	1,942	340	2,282
Meter #05C	Union	5,299	927	6,226
Meter #05D	Union	2,675	468	3,143
Meter #05E	Union	7,939	1,389	9,328
Meter #05H	Union	9,050	1,583	10,633
Meter #05I	Union	17,277	3,022	20,299
Meter #05J	Union	2,839	497	3,336
Meter #05K	Union	6,090	1,065	7,155
Meter #05L	Union	760	133	893
Meter #17	Newark	10,751	556	11,307
Meter #17E	Newark	3,651	189	3,840
Meter04	Roselle Park	11,735	56	11,791
Meter06	Hillside	18,515	-1,141	17,374
Meter10	Newark	5,214	270	5,484
Meter12	Newark	2,474	128	2,602
Meter13	East Orange	6,657	-1,879	4,778
Meter14	East Orange	10,590	-2,988	7,602
Meter15	South Orange	1,827	-308	1,519
Meter16	Irvington	2,714	-1,023	1,691
Meter18	Newark	22,194	1,147	23,341
Meter21	Maplewood	8,065	-2	8,063
Meter22	Maplewood	3,815	-1	3,814
Meter24	Summit/New Providence	31,978	2,021	33,999
Meter25	Maplewood	6,060	-1	6,059
Meter26/31	Maplewood	5,216	-1	5,215
Meter27	South Orange	6,614	-1,116	5,498
Meter28	South Orange	7,816	-1,319	6,497
Meter29	West Orange	35,489	4,163	39,652
Meter32C	Millburn/Livingston	6,379	510	6,889
Meter32D	Millburn/Livingston	7,032	563	7,595
Meter32E	Millburn/Livingston	3,911	313	4,224
Meter34	Hillside	1,900	-117	1,783

Subcatchment	Community	Baseline Conditions Population	Projected Change in Population between Baseline and Future Baseline Conditions	Baseline Future Conditions Population
Meter9	Irvington	1,901	-716	1,185
Meter9A	Irvington	50,479	-19,026	31,453
Meter9A-Up	Irvington	680	-256	424
Meter30	West Orange	5,254	616	5,870
Total		**342,032**	**-8,512**	**333,520**

From Table 4-10, it can be seen that the projected population of the JMEUC service area is expected to decline by roughly 8,500 persons or 2.5%. This slight decrease is in contrast to the marginal increase of the full community populations in 2050 (-2.5% vs. 1.37%) and can be explained by the fact that some communities are only partially serviced by the JMEUC while others are completely serviced by the JMEUC. For example, the Township of Irvington is completely serviced by the JMEUC and is projected to see a substantial decrease in population through 2050 based on U.S. Census Bureau data. Given that the Township makes up a relatively large portion of the JMEUC service area, this projected population decline has a noticeable impact on the projected population change of the JMEUC service area (causing the projected population to fall from 353,518 persons to 333,520 persons with its inclusion).

4.3.2 Planned Projects

Excluding Elizabeth (see Section 4.2.2), there are currently no future planned improvement projects expected that would significantly impact observed flows through the JMEUC trunk sewer system. As noted in Section 7, JMEUC member and customer communities are expected to continue to implement I/I improvements as part of ongoing best practices for sewer system maintenance, which can be expected to reduce flow to the JMEUC WWTF. Otherwise, any improvement projects along the JMEUC trunk sewers or undertaken by upstream member and customer communities will likely have only localized impacts that will not noticeably alter wet weather flows at the WWTF.

4.3.3 Projected Future Wastewater Flows

To represent 2050 population projections in the hydraulic model, populations of the subcatchments representing the separate sewer service area of the JMEUC were updated to those in Table 4-10. These population changes resulted in changes to the BSF of each subcatchment, as seen in Table 4-11.

Table 4-11: Change in Modeled Subcatchment BSF of the JMEUC separate sewer service area

Subcatchment	Community	Baseline Population	Estimated Population 2050	Population Change	Change in BSF (mgd)
Elmora	Elizabeth	9,250	10,676	1,426	0.143
Meter #05A	Union	1,942	2,282	340	0.020
Meter #05C	Union	5,299	6,226	927	0.056
Meter #05D	Union	2,675	3,143	468	0.028
Meter #05E	Union	7,939	9,328	1,389	0.083
Meter #05H	Union	9,050	10,633	1,583	0.095
Meter #05I	Union	17,277	20,299	3,022	0.181
Meter #05J	Union	2,839	3,336	497	0.030
Meter #05K	Union	6,090	7,155	1,065	0.064

Subcatchment	Community	Baseline Population	Estimated Population 2050	Population Change	Change in BSF (mgd)
Meter #05L	Union	760	892.924	133	0.008
Meter #17	Newark	10,751	11,307	556	0.008
Meter #17E	Newark	3,651	3,840	189	0.003
Meter04	Roselle Park	11,735	11,791	56	0.003
Meter06	Hillside	18,515	17,374	-1,141	-0.063
Meter10	Newark	5,214	5,484	270	0.012
Meter12	Newark	2,474	2,602	128	0.006
Meter13	East Orange	6,657	4,778	-1,879	-0.077
Meter14	East Orange	10,590	7,602	-2,988	-0.191
Meter15	South Orange	1,827	1,519	-308	-0.020
Meter16	Irvington	2,714	1,691	-1,023	-0.063
Meter18	Newark	22,194	23,341	1,147	0.052
Meter21	Maplewood	8,065	8,063	-2	0.000
Meter22	Maplewood	3,815	3,814	-1	0.000
Meter24	Summit	31,978	33,999	2,021	0.172
Meter25	Maplewood	6,060	6,059	-1	0.000
Meter26/31	Maplewood	5,216	5,215	-1	0.000
Meter27	South Orange	6,614	5,498	-1,116	-0.108
Meter28	South Orange	7,816	6,497	-1,319	-0.055
Meter29	West Orange	35,489	39,652	4,163	0.241
Meter32C	Millburn	6,379	6,889	510	0.023
Meter32D	Millburn	7,032	7,595	563	0.078
Meter32E	Millburn	3,911	4,224	313	0.014
Meter34	Hillside	1,900	1,783	-117	-0.010
Meter9	Irvington	1,901	1,185	-716	-0.189
Meter9A	Irvington	50,479	31,453	-19,026	-0.761
Meter9A-Up	Irvington	680	423.708	-256	-0.010
Meter30	West Orange	5,254	5,870	616	0.030
Total		**342,032**	**333,520**	**-8,512**	**-0.197**

As a whole, BSF from the JMEUC service area (excluding portions of Elizabeth tributary to the TAPS) decreased by 0.197 mgd. In total, BSF to the WWTF is projected to increase by roughly 0.8 mgd when including the roughly 1 mgd of BSF from the City entering the JMEUC system through the TAPS. This projected increase in flow is insignificant given the total flows to and existing capacity of the WWTF. As the table above shows, while overall changes in flow are not significant, several subcatchment-specific changes are modestly significant and the effect of these changes on the hydraulic performance of the system was investigated with the sewer system model, as described below.

As is the case with Elizabeth, future projected flows from the JMEUC service area do not explicitly include any future commercial or industrial developments. Given the service area is highly developed and largely residential, it is reasonable to assume that significant commercial and industrial development is unlikely.

As such, the roughly 0.2 mgd decrease in BSF implicitly includes any changes in commercial and industrial activity that may occur within the JMEUC service area over the planning period.

It should also be noted that the projections of future flow do not include reductions in per capita BSF that are likely to occur in the future. It has been observed on both local and national levels that the installation of low water use plumbing fixtures is reducing per capita BSF rates. As these fixtures continue to be installed as part of home remodeling and redevelopment projects, per capita BSF rates will continue to decrease. Because these reductions are difficult to accurately predict and quantify, they have been neglected but provide an additional margin of safety in the prediction that future BSF will not materially increase over the planning period.

As indicated in Section 7.2.2 of the JMEUC SCR, isolated areas within the JMEUC system are predicted to experience surcharge conditions during wet weather under baseline conditions, including two locations where the maximum simulated hydraulic grade line (HGL) reaches the manhole rim elevation at a few simulation timesteps. At these locations, the model predicts flood volumes of 0.00 MG under current baseline conditions. Future baseline condition model results indicate that projected subcatchment-specific changes in population and BSF will not measurably impact the baseline HGL or zero flood volumes at these locations. At one location, there is a reduction in BSF that drops the maximum HGL to below the modeled rim elevation. At the other location, the HGL and zero flood volume remain unchanged. In the remaining portions of the JMEUC system, no appreciable increases in HGL and no flooding are predicted to occur as a result of the subcatchment-specific population and flow changes in the JMEUC service area for 2050.

Section 5
Screening of CSO Control Technologies

5.1 Introduction

This section focuses on the technology screening process for the evaluation of CSO control alternatives per the requirements of the NJPDES CSO Permits. 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 Permit. This screening of technologies complies with the requirements of the National CSO Control Policy Section II.C.4 and is consistent with the EPA's "Guidance for Long Term Control Plan".

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.

The evaluation of seven (7) CSO control alternatives is mandated in Part IV.G.4.e of NJPDES CSO Permit. 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 all of the control alternatives explored in the subsequent subsections fall within the list. The seven (7) control alternatives listed in the Permit, and the corresponding section in which they are discussed herein, are:

1. Green infrastructure. Refer to Section 5.2.5.

2. Increased storage capacity in the collection system. Refer to Section 5.4.1.

3. 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. Refer to Section 5.6.

4. 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 and/or treatment capacity at the STP, and feasibility of implementing in the entire system or portions thereof. Refer to Section 5.3.

5. Sewer separation. Refer to Section 5.7.

6. Treatment of the CSO discharge. Refer to Section 5.8.

7. 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. Refer to Section 5.6.

The evaluation consists of two steps: a screening of alternatives at a high level, followed by a more detailed evaluation of the performance and costs of the alternatives, which is presented in Section 7. The screening of alternatives is summarized in this section.

The screening takes place on several levels. In some cases, a general category may be screened in or out based on its applicability to the City or JMEUC. 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.4.a, which calls for permittees to "evaluate a reasonable range of CSO control alternatives".

The screening is based on the requirement to "evaluate the practical and technical feasibility of the proposed CSO control alternative(s)" 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 – Per the requirements of Part IV.G.4.a and G.4.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, it cannot be evaluated. Many such practices are already in place under other requirements, such as the Tier A Municipal Separate Storm Sewer System (MS4) NJPDES General Permit. These practices would be considered part of the baseline conditions 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 City and JMEUC to implement. For example, the character of a residential neighborhood should not be displaced to make room for a storage or treatment facility. 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.

With respect to water quality, control technologies are screened for their effectiveness in addressing pollutants of concern and CSO control goals to achieve compliance with the Clean Water Act. These goals focus on reducing (1) the volume of CSO discharged to the receiving waterbodies, and (2) the associated water quality impacts, especially pathogen concentrations for:

- Fecal coliform;
- Enterococcus; and
- Escherichia coli reduction

Details on each CSO control technology are presented below and the above criteria are subsequently applied in the screening process to determine the suitability of the control to the subject combined sewer system. A matrix table is included in Section 5.9 to summarize the results of the alternatives screening process.

5.2 Source Control

The United States Environmental Protection Agency (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 combined sewer system (CSS). However, source controls typically require a high level of effort to implement on a scale that can achieve a measurable impact.

The City of Elizabeth 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 / basin type 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.

5.2.1 Stormwater Management

Stormwater management controls consist of measures designed to capture, treat, or delay stormwater runoff prior to entering the CSS.

5.2.1.1 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. 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. Given these potential public safety issues and the heavy traffic, local climate, and combined sewer overflow conditions, catch basin modifications for street and parking storage is considered not appropriate for the City’s CSO control program.

5.2.1.2 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.

It is the City’s current practice and design standard to require catch basins with hoods and collection sumps for heavy solids. Grates and curb pieces meeting NJDEP standards, thereby providing floatables control, are also being used. Furthermore, solids/floatables (S/F) control facilities have been constructed and are being maintained along the CSO outfalls, per previous and current permit requirements. These current practices will continue, but catch basin modification will not be considered further for the alternatives evaluation.

5.2.1.3 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 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. 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.

For leaching catch basins to be considered a feasible control technology, the surrounding soils must be of suitable permeability and the surrounding average groundwater level must be sufficiently below the basin. Leaching catch basins are not considered practical because they offer no filtering of the urban runoff prior to infiltrating below the catch basin. The recommended 15-20 ratio of impervious area to infiltration area would also be very high for typical inlets and would not be practical. Further, based on the information from the soil survey information, the City is primarily characterized as urban land, with soils of low permeability and low available water storage. Accordingly leaching catch basins will be removed from consideration for the alternatives evaluation.

5.2.2 Public Education and 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. 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.

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. The specific public outreach alternatives identified are summarized below for completeness. However, as the impact of public outreach cannot be quantified, it is removed from future consideration for the alternatives evaluation. Nevertheless, it is anticipated that public outreach will continue under future iterations of the Permit as a key element of the LTCP implementation.

5.2.2.1 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 City 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.

5.2.2.2 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 City 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.

5.2.2.3 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 City 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.

5.2.2.4 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.

5.2.2.5 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, or overflows from combined sewer systems. 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. While the City is instituting revisions to its ordinances for FOG source control from food service establishments, this measure is considered to be a program enhancement and will not be considered further for the alternatives evaluation.

5.2.2.6 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. Accordingly, garbage disposal restrictions will be removed from future consideration for the alternatives evaluation.

5.2.2.7 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.

The City of Elizabeth 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.

5.2.2.8 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.

The City of Elizabeth currently enforces fertilizer and pesticide management ordinances as required under the stormwater management permits. 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.

5.2.2.9 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 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 City 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.

5.2.3 Ordinance and Rules Enforcement

Per New Jersey statutes and regulations on land development and stormwater management, the City currently regulates and enforces ordinances related to the following source control measures. Additionally, JMEUC maintains strict control over discharges to its sewer system with published Rules and Regulations that protect public health and the integrity of its facilities.

5.2.3.1 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 has 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. 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. As an existing practice to be continued, this source control measure will be removed from future consideration for the alternatives evaluation since it is already in place.

5.2.3.2 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. The City and various state agencies enforce existing regulations to control illegal dumping and will continue doing so. As an existing practice to be continued, this source control measure will be removed from future consideration for the alternatives evaluation since it is already in place.

5.2.3.3 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. As an existing practice to be continued, this source control measure will be removed from future consideration for the alternatives evaluation since it is already in place.

5.2.3.4 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. As an existing practice to be continued, this source control measure will be removed from future consideration for the alternatives evaluation since it is already in place.

5.2.3.5 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.

The City 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 be removed from future consideration for the alternatives evaluation since it is already in place.

5.2.3.6 JMEUC Sewer Use Regulations

The JMEUC Rules and Regulations set forth uniform requirements for dischargers into the Joint Meeting wastewater collection and treatment systems, and enable the Joint Meeting to protect the public health in conformity with all Applicable Laws relating thereto. The objectives of these Rules and Regulations are:

(1) to prevent the introduction of pollutants into the Joint Meeting Treatment Works, which will interfere with the normal operation of the Treatment Works or contaminate the resulting sludge;

(2) to prevent the introduction of pollutants into the Treatment Works which do not receive satisfactory treatment by the Treatment Works or which pass through the system into receiving waters or the atmosphere or otherwise would be incompatible with the Treatment Works; and

(3) to improve the opportunity to recycle and reclaim wastewater.

These Rules and Regulations provide for the regulation of discharges into the Joint Meeting Treatment Works through the issuance of Industrial User Permits. All Users of the Joint Meeting Treatment Works, whether issued an Industrial User Permit or not, are subject to and must comply with the requirements of the written rules and regulations of the Municipality and Joint Meeting.

5.2.4 Good Housekeeping

5.2.4.1 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 City has a street sweeping program already in place per the stormwater management permits. As such, this measure will not be evaluated further.

5.2.4.2 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. The City has an existing and satisfactory leaf collection program that will be continued in the future. As such, this measure will not be evaluated further.

5.2.4.3 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. The City has an existing and satisfactory recycling program that will be continued in the future. As such, this measure will not be evaluated further.

5.2.4.4 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. Local ordinances are unlikely to produce additional benefits beyond the current program, and such actions cannot be quantified in terms of reduction to CSO pollutant loadings. As such, this measure will not be considered further.

5.2.4.5 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. 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. Local ordinances are unlikely to produce additional benefits beyond the current program, and such actions cannot be quantified in terms of reduction to CSO pollutant loadings. As such, this measure will not be considered further.

5.2.5 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 be 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 sufficient opportunities to install green infrastructure may not exist or be practical, resulting in the Permittee 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.
1. The green infrastructure must be under the control of JMEUC or the City to ensure that it remains in place and that maintenance occurs.
    - Practices evaluated will be sited on land owned by the City including:
        - o Municipal owned right-of-ways
        - o 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 manage stormwater generated within the park.
   - Property of affiliated public entities, like municipal school districts, housing and parking authorities
2. 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 7.6 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.

5.2.5.1 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 5-1.

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

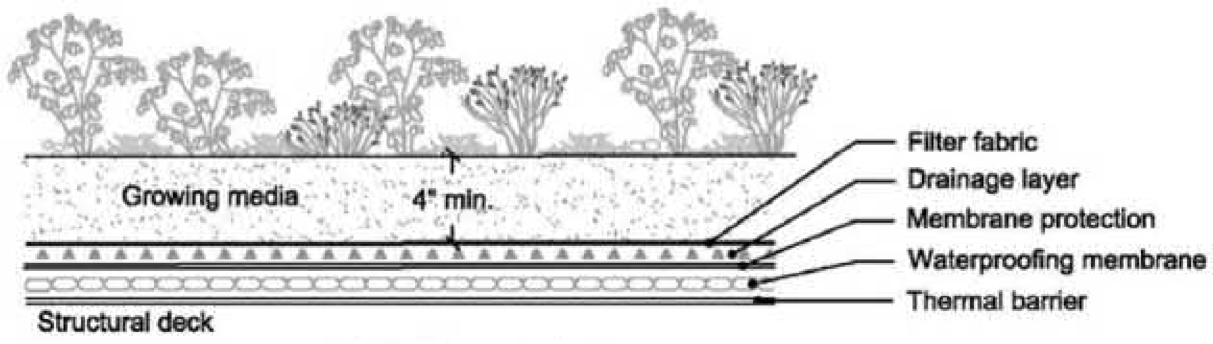


Figure 5-1 Example Green Roof Section

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. However, 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.

Green roofs could be implemented in the City 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. While the City could investigate retrofitting of existing building, given the associated technical challenges and risks, it is not practical or prudent to evaluate retrofit of existing buildings for green roofs in the context of the LTCP.

5.2.5.2 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 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 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 measurable impact.

The City 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 City's control, and anticipated redevelopment patterns. Accordingly, green and blue roofs will not receive further consideration as part of the alternatives analysis.

5.2.5.3 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 5-2, 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.


Figure 5-2 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.

5.2.5.4 Permeable Pavements

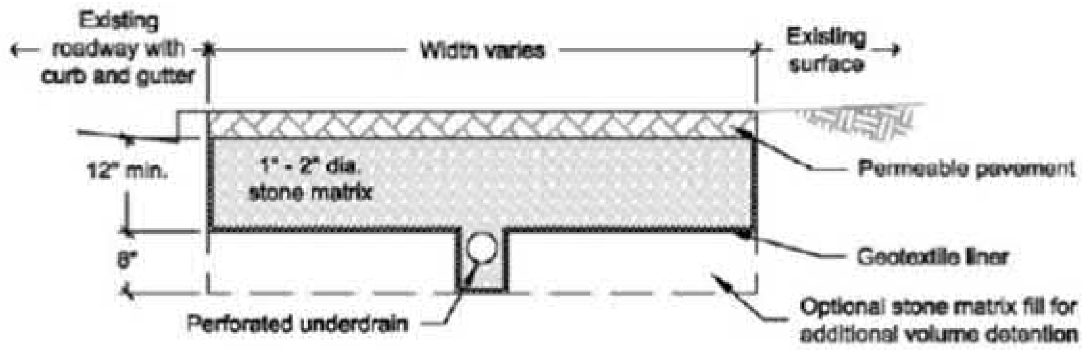
Permeable pavements promote runoff infiltration and rely on a permeable substrate (engineered soils) to store runoff and remove pollutants. There are different types of permeable pavements, most commonly constructed with asphalt, concrete, or pavers. Permeable asphalt and concrete are similar to traditional 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. Figure 5-3 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.

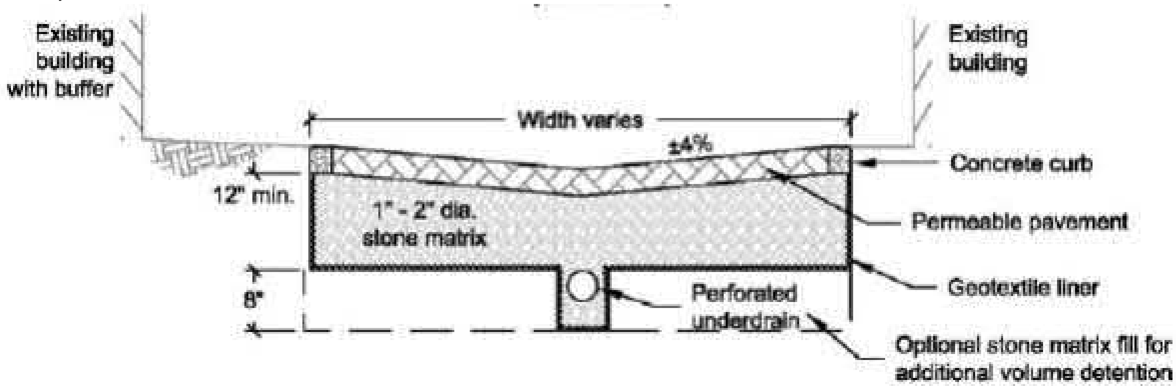
Permeable pavement is typically recommended for low traffic areas; thus, it may be feasible to re-pave municipal parking areas with permeable pavement, specifically the parking stalls and not the travel lanes. This is a common approach and is reflected in Example 1 of Chapter 9.7 of the New Jersey Stormwater 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 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. Since the groundwater level may be shallow, the groundwater separation criterium may greatly limit locations where permeable pavement can be implemented.

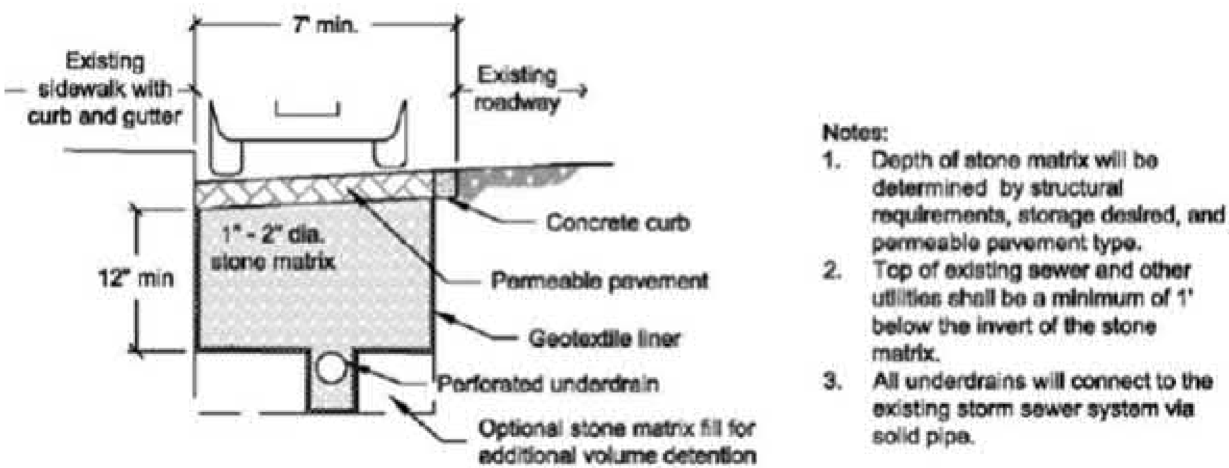
Parking lanes within the City offer a large area to implement permeable pavement. It is noted that there is a high demand for street parking in the City, and the temporary unavailability of parking associated with installation of the permeable pavement make this area less favorable. There are also numerous utilities in the parking lanes which could be very difficult to work around or relocate.



Example Permeable Sidewalk Section



Example Permeable Alley Section



Example Permeable Parking Lane Section

Figure 5-3 Example Permeable Pavement Sections

Sidewalks offer a reasonable 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. 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.

5.2.5.5 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 5-4. 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 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. 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.

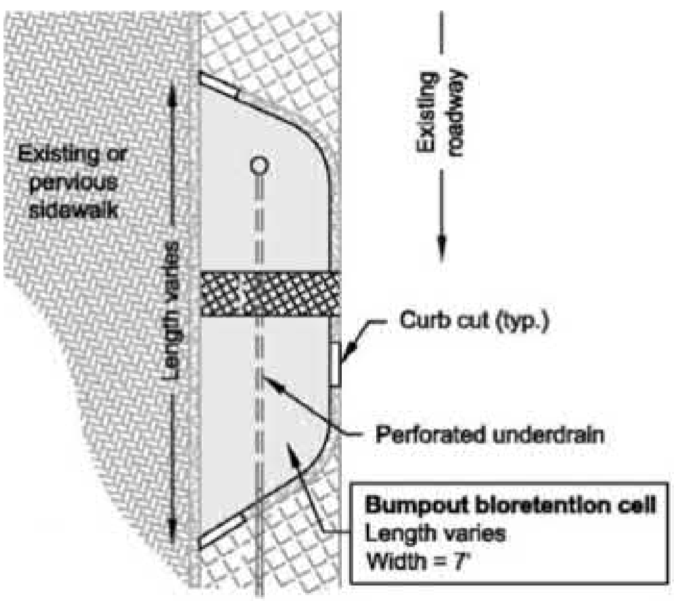


Figure 5-4 Example Bumpout Planter Box Layout

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. 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.

5.2.5.6 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 5-5. 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. 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 7.6.

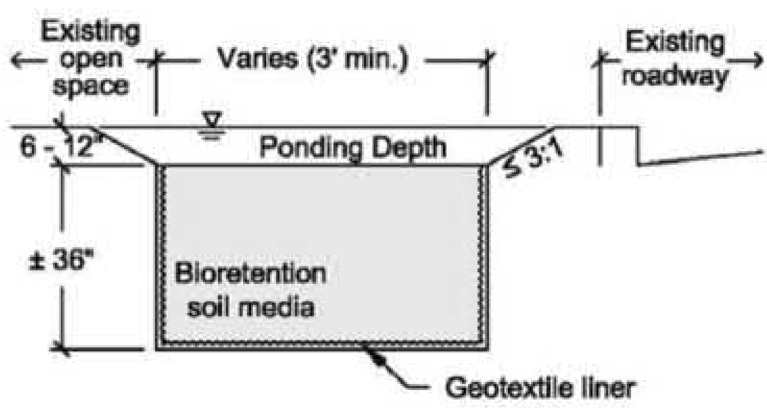


Figure 5-5 Example Bioswale Detail

5.2.5.7 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 design is found in Figure 5-6. 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 City. 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 City may elect to site additional rain gardens within available municipal owned land and continue to promote them on private property.

As rain gardens 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 rain gardens based on future site-specific selections.

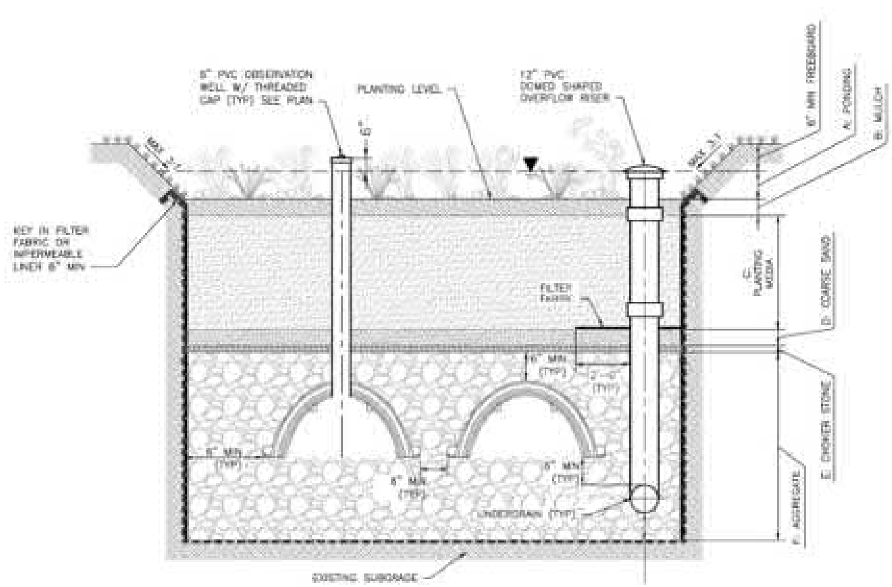


Figure 5-6 Example Rain Garden Section

5.3 Infiltration and Inflow Control

5.3.1 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, and the system is designed to capture inflow. Sanitary sewer systems are not designed to capture inflow, although design standards often recognize that completely excluding inflow is extremely difficult and make an allowance for modest rates 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 rate and 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 various member and customer communities within the JMEUC service area have implemented I/I reduction programs and significant data is available to review I/I impacts. Given the NJPDES CSO Permit requirements, the achievable levels of infiltration and inflow reductions within the JMEUC service area will be considered further for the alternatives evaluation.

5.3.2 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 goals of the LTCP.

The City and JMEUC maintains their collection systems regularly and are 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.

5.3.3 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 City performs sewer cleaning and flushing regularly and as needed. The current program is considered to be implemented at a satisfactory level and will be continued. As such, this measure will not be considered further.

5.3.4 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. The City 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.

5.4 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 direct impact on water quality, but do have the potential to reduce the volume of CSO events.

5.4.1 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.

5.4.1.1 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.

Additional conveyance will be considered in greater detail for the alternatives evaluation as a potential primary and complimentary technology for CSO control. For example, additional conveyance from the existing Trenton Avenue Pump Station to a certain flow rate and time interval during wet weather events can be achieved without wastewater treatment facility (WWTF) upgrades. Opportunities for additional conveyance in certain interceptor sections to increase the use of existing downstream interceptor capacity also may be appropriate to decrease the size of primary controls for certain outfalls.

5.4.1.2 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.

Regulator modifications will 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. For example, modifying regulators to direct more flow to the interceptors will likely be appropriate to fully utilize downstream conveyance and decrease the size of primary controls for certain outfalls.

5.4.1.3 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 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. For example, it may be possible to consolidate a number of outfalls to a single location where storage or treatment facilities are to be located.

5.4.1.4 Real Time Control

Real Time Control (RTC) is an 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 and/or conveyance 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 (rainfall) and/or flow forecast data, is an optional feature, but is complex and requires sophisticated operational capabilities.

RTC involves the installation of mechanical controls, which require upkeep and maintenance, and can only reduce CSO volumes where in-system storage capacity or additional conveyance 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 City. However, RTC may serve as a complementary technology used with a primary alternative, such as additional conveyance from the Trenton Avenue Pump Station.

5.5 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.

5.5.1 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.

5.5.1.1 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 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. Considering the required volumes for the CSO control levels, parallel pipeline storage is not practical and is eliminated from further consideration.

5.5.1.2 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.

While there are many challenges associated with constructing a tunnel in City of Elizabeth, because of the large storage volume provided, relatively lower 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.

5.5.2 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.

5.5.2.1 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 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 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.

While siting of storage tanks within a densely populated areas can be challenging, there are some potential areas available in the City as well as options for consolidated storage. Storage tanks provide effective reduction of CSO volumes and can provide for full treatment of stored flow by allowing the retained volume to be conveyed to the treatment plant, provided that adequate interceptor sewer conveyance capacity and treatment process capacities are available. Accordingly, storage tanks will be considered for evaluation as a feasible alternative for significant CSO control.

5.5.2.2 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 as conveyance and treatment capacities are restored. This would limit the peak wet weather flow to the WWTF and reduce the potential for industrial pollutants to discharge at CSO outfalls during wet weather events. 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 further elsewhere in this report.

5.6 Sewage Treatment Plant Expansion or Storage

5.6.1 Additional Treatment Capacity

CSOs can potentially be reduced by increasing the treatment capacity of the existing wastewater treatment plant. Other technologies, especially increased conveyance capacity, may be needed to make use of this increased treatment capacity by providing more flow to the plant instead of flow to CSO outfalls.

5.6.2 Wet Weather Blending

Blending is the practice of allowing portions of the wet weather peak flow to bypass certain treatment facilities at the plant. The practice of bypassing at the plant would only be used in accordance with a NJPDES permit to specifically authorize bypass. In blending, wet weather flows are typically routed through primary treatment, allowed to bypass secondary treatment (and tertiary treatment, if provided) 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 more costly components. Bypassing the peak wet weather flows from a combined sewer system, which typically increase quickly to very high rates of diluted flow, around the biological processes used in secondary treatment also avoids upsets to those biological processes. As noted above, other technologies, especially increased conveyance capacity, may be needed to make use of the increased wet weather peak flow capacity potentially achieved by blending by providing more flow to the plant instead of flow to CSO outfalls.

5.7 Sewer Separation

5.7.1 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 City of Elizabeth is a highly urbanized area and there is limited opportunity for infiltration of storm runoff from roof leaders to pervious areas. The City prohibits direct connection of roof leaders to the combined sewer system for new construction and renovations, which should reduce the number of connections over time. However, 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. Roof leader disconnection may be coupled green infrastructure technologies, but it has limited application and will not be considered further in the evaluation.

5.7.2 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 City 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 City's combined sewer system and will not be considered further in the alternatives evaluation.

5.7.3 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 while the new pipes are installed and new connections are made. The City has completed numerous sewer separation projects, often associated with flood relief and property redevelopment programs, which has resulted in the elimination of some CSO outfalls. However, these projects in most cases have only partially separated the storm runoff from the larger CSO basin and many CSO outfalls also have storm drain connections downstream of the regulator.

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 estimate highly uncertain.

In spite of its many challenges, sewer separation is a primary technology that could completely eliminate combined sewer overflows. For this reason, this alternative is maintained for future consideration in the alternatives evaluation.

5.8 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.

Satellite end of pipe treatment has been used successfully in other places, and the potential exists for installing end-of-pipe treatment in the City. 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 6.1 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.

5.8.1 Treatment – CSO Facility

5.8.1.1 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 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.

5.8.1.2 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 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.

Static bar screens are being used in the City at certain regulator locations and in conjunction with netting chambers for solids and floatables control. 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 disinfection for bacteria reduction.

5.8.1.3 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. As noted above, the City currently operates 27 CSO netting chambers. 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.

5.8.1.4 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 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 City already has netting facilities and static bar screens for solids and floatables control, containment booms would provide no additional benefit and will not be considered further in the LTCP.

5.8.1.5 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.

As the City already has netting facilities and static bar screens for solids and floatables control, baffles would provide no additional benefit and will not be considered further in the LTCP.

5.8.1.6 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 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 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. Chlorine-based disinfection processes typically require use of a dechlorination process prior to discharge to protect aquatic life. 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 alone does not provide solids removal, solids removal would need to be accomplished with a separate technology and varying levels of solids removal are required prior to disinfection, depending upon the specific disinfection process. While other disinfection technologies exist, peracetic acid is considered as an appropriate disinfection approach for the alternatives evaluation.

5.8.1.7 High Rate Physical/Chemical Treatment

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 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 biochemical 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. As such, these processes may not add significant value towards the LTCP primary goal of bacteria reduction compared to disinfection alone or with less complex and costly solids removal technologies. 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.

5.8.1.8 High Rate Physical Treatment

The Fuzzy Filter® by Schreiber or the WesTech WWETCO FlexFilter™ is an innovative filtration technology that uses a compressible filter media, allowing a much smaller footprint than conventional filtration, with typical 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. Similar to 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.

5.9 Summary

Table 5-1, Table 5-2, and Table 5-3 provide a summary of the control technologies considered in this section, with the results from the preliminary screening indicated.

Table 5-1 Source Control Technology Screening Summary

Technology Group	Practice	Primary Goals		Implementation & Operation Factors	Consider Combining w/ Other Technologies	Being Implemented	Recommendation for Alternatives Evaluation	Notes
		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	Not suitable.
	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	Continue current practice.
	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	Not suitable for soils or groundwater conditions.
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	Minimal benefits, already being implemented.
	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	Already being implemented.
	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	Already being implemented.
	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	Already being implemented.
	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	Already being implemented.
	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	Minimal benefit and unenforceable.
	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	Already being implemented.
	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	Minimal benefit and unenforceable.
	Hazardous Waste Collection	Low	None	The N.J.A.C prohibits the discharge of hazardous waste to the collection system.	Yes	Yes	No	Already being implemented.
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	Already being implemented.
	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	Already being implemented.
	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	Already being implemented.
	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	Already being implemented.
	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	Already being implemented.

Technology Group	Practice	Primary Goals		Implementation & Operation Factors	Consider Combining w/ Other Technologies	Being Implemented	Recommendation for Alternatives Evaluation	Notes
		Bacteria Reduction	Volume Reduction					
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	Already being implemented.
	Leaf Collection	Low	None	Requires additional seasonal labor. Leaf collection maximizes flow capacity and removes nutrients from the collection system.	Yes	Yes	No	Already being implemented.
	Recycling Programs	None	None	Most Cities have an ongoing recycling program.	Yes	Yes	No	Already being implemented.
	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	Minimal benefits.
	Industrial Spill Control	Low	None	JMEUC has established a pretreatment program for industrial users subject to the Federal Categorical Pretreatment Standards 40 CFR 403.1.	Yes	Yes	No	Already being implemented.
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	Not practical
	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	Not practical
Green Infrastructure Buildings	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	Not feasible
Green Infrastructure Impervious Areas	Permeable Pavement	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	Advance to evaluation
	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	Incorporated into evaluation as bioswales
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	Advance to evaluation; representative technology
	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	Incorporated into evaluation as bioswales

Table 5-2 Collection System Technology Screening Summary

Technology Group	Practice	Primary Goals		Implementation & Operation Factors	Consider Combining w/ Other Technologies	Being Implemented	Recommendation for Alternatives Evaluation	Notes
		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	Yes	Further analysis for feasibility.
	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	Minimal benefits
	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	Already being implemented.
	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	Already being implemented.
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	Not likely to be effective
	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	Not likely to be effective
	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	Advance to evaluation
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	Pump station focus
	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	As part of other alternatives
	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	As part of other alternatives
	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	Yes	As part of other alternatives

Table 5-3 Storage and Treatment Technology Screening Summary

Technology Group	Practice	Primary Goals: Bacteria Reduction	Primary Goals: Volume Reduction	Implementation & Operation Factors	Consider Combining w/ Other Technologies	Being Implemented	Recommendation for Alternatives Evaluation	Notes
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	Not cost effective
	Tunnel	High	High	Requires small area at ground level relative to storage basins; disruptive at shaft locations; increased O&M burden.	No	No	Yes	Advance to evaluation
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	Advance to evaluation
	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	Review impacts from SIUs
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	Not effective alone
	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	Not effective alone, include as part of other alternatives
	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	Already being implemented.
	Contaminant Booms	None	None	Difficult to maintain requiring additional resources. Contaminant booms will only address floatables.	Yes	No	No	Not effective
	Baffles	None	None	Very low maintenance; easy to install; requires proper hydraulic configuration; long lifespan. Baffles will only address floatables.	Yes	No	No	Not effective
	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	Advance to evaluation
	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	Advance to evaluation
	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	Consider alternate technology
Treatment-WWTP	Additional Treatment Capacity	High	High	May require additional space; increased O&M burden.	No	No	Yes	Advance to evaluation
	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	Yes	Advance to evaluation
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	Review impacts from SIUs

Section 6
Evaluation Criteria and Performance Considerations

This section describes the criteria and factors considered in assessing the CSO control strategies identified for evaluation. Beyond cost and performance considerations, other factors are reviewed to provide information and insight on the feasibility of a control alternative. The degree to which an alternative is practicable and constructible given actual local conditions is an essential decision factor in the alternatives evaluation process. There may be significant issues associated with the physical implementation and construction of a control strategy that are not represented in its modeling results and preliminary cost estimates and these issues could have serious implications for the fulfillment of the LTCP objectives if not appropriately considered.

6.1 Preliminary Siting Analysis

The EPA document "Combined Sewer Overflows: Guidance for Long-Term Control Plan" (EPA 832-B-95-002 September 1995) lists preliminary siting considerations as a screening mechanism for evaluating CSO control alternatives and recommends 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 be controlled
- Environmental, political, or institutional issues related to locating the facility on the site.

In order to identify potential sites in the vicinity of CSS regulators and outfalls where CSO control measures might be installed based on the criteria above, the following publicly available geographic information system (GIS) information was utilized:

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

This information was layered into a GIS environment and analyzed to identify candidate sites for point and consolidated CSO control facilities around the City of Elizabeth. In the initial analysis, residential areas, transportation corridors and water bodies were eliminated as it was reasoned that these areas would not be suitable as primary candidates given the extensive disturbance that would be required for the facilities. The overall land use of Elizabeth is shown in Figure 6-1, with residential, transportation corridors and water bodies subtracted out. The remaining shaded areas were evaluated for potential sites by visual inspection.

Available aerial photography was used to review the remaining areas around the CSO outfalls and associated regulators. Sites were prioritized based on proximity to outfalls, public ownership or vacant land, and under-utilized locations such as parking areas or abandoned sites. It is noted that the desktop study was based on limited existing data and reasonable inferences were made where appropriate. Table 6-1 below summarizes the characteristics that were considered to identify potential CSO control facility locations in the collection system.

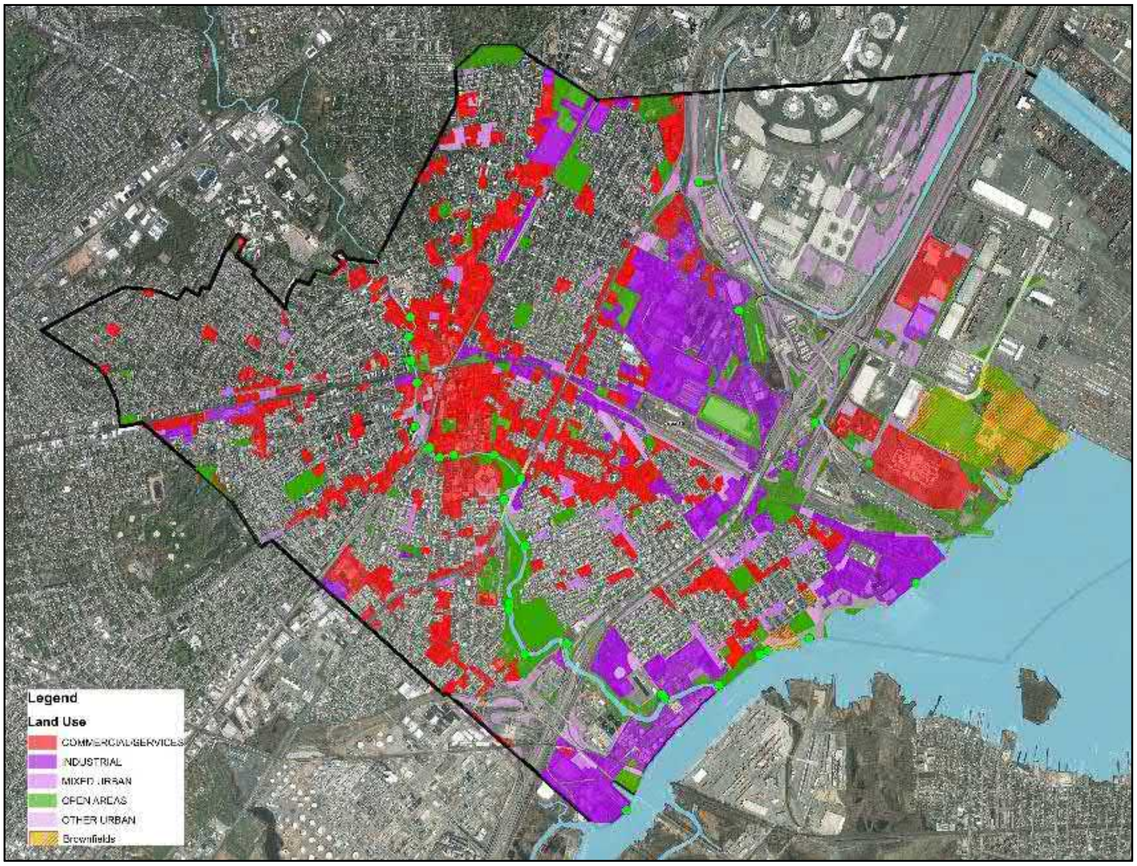


Figure 6-1: Elizabeth Non-Residential Land Use Map

Table 6-1: Siting Criteria Table

Favorable	Unfavorable
Open paved or grass areas, vacant land	Buildings / Structures
Industrial, Commercial, Open Space	Green Acres, Residential, Transportation Corridors
Publicly owned	Privately owned
Small elevation change to outfall or regulator	Large elevation change to outfall or regulator
Close to outfall or regulator	Far from outfall and regulator
No known soil or groundwater contamination	Known contaminated site or brownfield site

By performing a desktop analysis of the City using available GIS information, over 80 sites were identified by the project team as potential locations for control facilities near CSO outfalls. Many of these sites are associated with multiple property parcels. The area surrounding each outfall and regulator was reviewed and multiple possible sites in each basin were drawn. Generous consideration was made to include possible under-utilized locations, with the intent to maximize the initial review of unoccupied space and avoid the future need to acquire occupied residential and commercial property.

Using the site evaluation criteria described above as well as input from the City on planned developments, property owners, easement requirements and potential disruption to the community, the sites were ranked in order of favorability using the following ranking:

- Good (e.g., site is favorable and likely a good candidate for a CSO control facility).
- Fair

- Low
- Very Low (e.g., unavailable or severe site constraints and likely unacceptable for acquisition).

The complete listing of these potential sites is provided in Table 6-2. Figure 6-2, Figure 6-3, and Figure 6-4 depicts the locations of the sites for the northwest, northeast, and south areas of the City, respectively. Based on the initial evaluation by City representatives, only 11 of the 85 potential sites, or 12.9%, were considered well suited for a relatively smooth easement acquisition and facility siting. Another 23.5% of the sites were rated with a fair probability for potential siting, while 52.9% and 10.6% were identified with low and very low ratings as suitable locations.

Many of the low and very low ranking locations were noted as having major redevelopment projects currently underway, with plans for construction approved or under review with the City planning board. Given the wide spatial distribution of the CSO outfalls, there are significant competing interests for potential sites given that the City has several ongoing redevelopment programs focusing on economic initiatives. Other sites are indicated as likely to be highly disruptive to the existing business operations associated with the parking locations.

This preliminary analysis to identify potential open or under-utilized sites for CSO control facilities showed that a very limited amount of such space is available within the City. The outfall by outfall investigation noted that the type and amount of real estate surrounding each outfall is nearly fully occupied and highly constrained. Insufficient open and under-utilized land appears to be available for control measures related to the broadly distributed CSO outfall locations and significant acquisition of occupied commercial, residential, and other urban land will likely be required to implement CSO control facilities sited within the City. Extensive business and resident displacement, lost property taxes, and neighborhood disruptions would likely be associated with the procurement of such land for CSO facility siting. These considerations and the estimated costs for obtaining land rights to construct the CSO facilities will impact the assessment of the control strategies.

6.2 Institutional Issues

Institutional issues refer to permitting requirements, likelihood of receiving permits, timeline to receive permits, regulatory compliance in terms of water quality improvements, and ownership of the site (public versus private). Regulatory considerations such as Green Acres, flood hazard area, and wetlands 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 JMEUC interceptors and WWTF.

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 may take place within the waterfront development area, which extends 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 (i.e., 90-day construction permits).

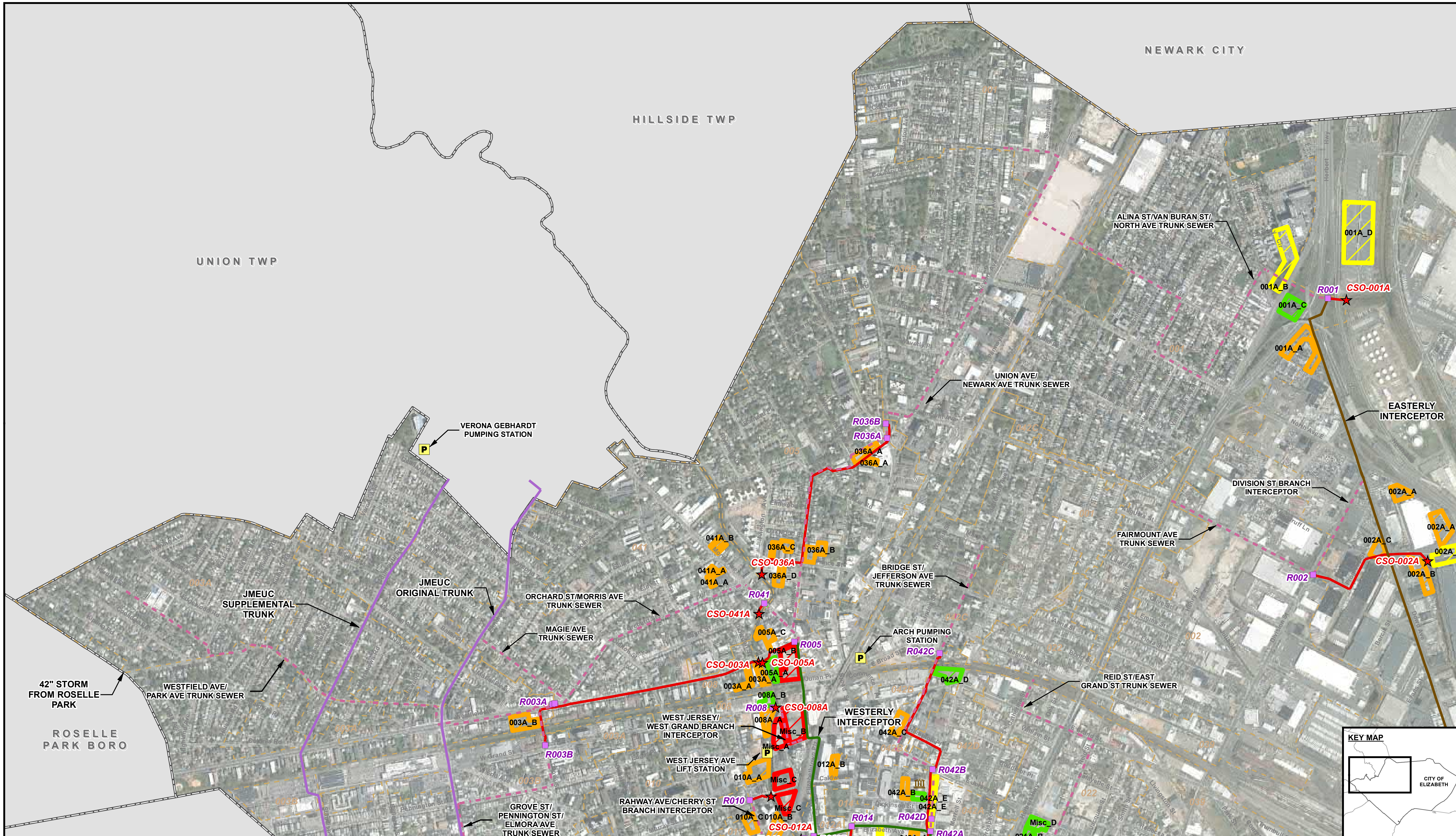
Table 6-2: Preliminary Siting Analysis Results

CSO Basin	Site ID	General Location	Site Area (acres)	Ownership Type	Predominant Land Use	Green Acres	100-yr Flood Hazard	Brownfield Site	Known Contamination	Favorability Rating
001	001A_A	Meadow St	2.06	Private	Commercial	No	No	No	Yes	Low
	001A_B	Spring St	2.96	Private	Commercial	No	No	No	No	Fair
	001A_C	Spring St	1.39	Public-NJDOT	Other Urban	No	No	No	No	Good
	001A_D	Newark Airport	6.71	Public-Port Authority	Transportation	No	No	No	No	Fair
002	002A_A	North Ave E	2.44	Private	Industrial	No	Yes	No	No	Low
	002A_B	Dowd Ave	1.38	Private	Industrial	No	No	No	No	Low
	002A_C	Dowd Ave	0.86	Private	Industrial	No	No	No	No	Low
	002A_D	Dowd Ave	1.35	Private	Open Areas	No	Yes	No	No	Fair
003	003A_A	Westfield Ave	0.47	Private	Open Areas	No	Yes	No	No	Low
	003A_B	Westfield Ave	1.20	Private	Commercial	No	No	No	No	Low
005	005A_A	Harrison St	0.60	Public-City	Open Areas	No	No	No	No	Good
	005A_B	Crane St	2.08	Public-City	Open Areas	No	No	No	No	Very Low
	005A_C	Westfield Ave	0.67	Private	Residential High Density	No	No	No	No	Low
008	008A_A	W Grand St	0.66	Private	Commercial	No	No	No	No	Low
	008A_B	W Grand St	0.47	Public-County	Industrial	No	Yes	No	No	Good
010	010A_A	W Jersey St	1.42	Private	Other Urban	No	Yes	No	No	Low
	010A_B	Cherry St	0.92	Public-Amtrak	Transportation	No	Yes	No	No	Low
	010A_C	Cherry St	0.64	Private	Commercial	No	No	No	No	Low
012	012A_A	Rahway Ave	0.60	Public-County	Commercial	No	No	No	No	Low
	012A_B	W Jersey St	0.42	Private	Commercial	No	No	No	No	Low
013	013A_A	Pearl St	0.55	Private	Residential High Density	No	No	No	No	Low
	013A_B	Rahway Ave	0.33	Public-County	Residential High Density	No	Yes	No	No	Low
	013A_C	Rahway Ave.	0.63	Public-Amtrak	Commercial	No	No	No	No	Low
	013A_D	Rahway Ave	0.29	Private	Commercial	No	No	No	No	Fair
014	014A_A	S Broad St	0.42	Public-City	Commercial	No	No	No	No	Low
	014A_B	S Broad St	0.42	Private	Commercial	No	No	No	No	Fair
016	016A_A	Pearl St	0.40	Private	Residential High Density	No	No	No	No	Low
	016A_B	S Broad St	1.17	Private	Commercial	No	No	No	No	Fair

CSO Basin	Site ID	General Location	Site Area (acres)	Ownership Type	Predominant Land Use	Green Acres	100-yr Flood Hazard	Brownfield Site	Known Contamination	Favorability Rating
	016A_C	Pearl St	0.60	Private	Commercial	No	No	No	No	Fair
021	021A_A	N Spring St	0.48	Public-NJDOT	Commercial	No	No	No	No	Good
	021A_B	N Spring St	0.50	Private	Residential High Density	No	No	No	No	Low
	021A_C	Elizabeth Ave	0.78	Public-City	Open Areas	No	No	No	No	Fair
022	022A_A	South St.	0.33	Private	Industrial	No	No	No	No	Low
	022A_B	South St	1.55	Private	Industrial	No	No	No	Yes	Fair
	022A_C	Centre St	0.77	Private	Other Urban	No	Yes	No	No	Good
	022A_D	Elizabeth Ave	0.36	Public-City, Parking Authority	Commercial	No	No	No	No	Fair
026	026A_A	Palmer St	0.18	Public-City	Commercial	No	No	No	No	Fair
	026A_B	S Seventh St	1.01	Public-County	Residential High Density	Yes	Yes	No	No	Fair
	026A_C	Amity St	0.37	Public-County	Open Areas	No	Yes	No	No	Fair
027/028	027A/028A_A	Clarkson Ave	0.95	Public-Board of Ed	Transportation	No	Yes	No	No	Low
	027A/028A_B	Clarkson Ave	0.93	Public-County	Open Areas	Yes	Yes	No	No	Fair
029	029A_A	Elizabeth Ave	4.13	Private	Industrial	No	Yes	No	No	Fair
	029A_B	Front St	1.12	Public-Board of Ed	Commercial	No	No	Yes	No	Very Low
030	030A/031A_A	Front St	0.81	Public-City	Open Areas	Yes	No	No	No	Fair
	030A/031A_B	Front St	0.68	Public-City	Other Urban	Yes	Yes	Yes	No	Fair
	030A/031A_C	Livingston St	1.23	Public-City	Open Areas	No	No	Yes	No	Very Low
	030A/031A_D	First St	0.36	Private	Commercial	No	No	No	No	Low
032	032A_A	Front St	2.80	Public-City	Other Urban	No	No	No	No	Very Low
	032A_B	Front St	0.58	Public-City	Residential High Density	No	No	No	No	Fair
	032A_C	First St	2.03	Private	Industrial	No	No	No	No	Good
034	034A_A	Atalanta Plaza	1.28	Private	Industrial	No	No	No	No	Low
	034A_B	First St	3.51	Private	Transportation	No	No	No	No	Low
035	035A/043A_A	S First St	5.47	Private	Transportation	No	Yes	No	No	Low
	035A/043A_B	S First St	1.17	Private	Industrial	No	Yes	No	No	Low
	035A/043A_C	S First St	0.80	Private	Industrial	No	Yes	No	No	Low

CSO Basin	Site ID	General Location	Site Area (acres)	Ownership Type	Predominant Land Use	Green Acres	100-yr Flood Hazard	Brownfield Site	Known Contamination	Favorability Rating
	035A/043A_D	Butler St	1.49	Private	Industrial	No	No	No	No	Low
036	036A_A	Westminster Ave	0.93	Private	Commercial	No	No	No	No	Low
	036A_B	Prince St	0.96	Private	Commercial	No	No	No	No	Low
	036A_C	Prince St	0.66	Private	Commercial	No	No	No	No	Low
	036A_D	Union Ave	0.56	Private	Commercial	No	No	No	No	Low
037	037A_A	S Front St	1.88	Private	Industrial	No	Yes	No	No	Low
	037A_B	S Front St	0.88	Private	Industrial	No	Yes	No	No	Low
038	038A_A	Fifth Ave	1.26	Public-County	Open Areas	Yes	No	No	No	Low
	038A_B	Atlantic St	0.32	Private	Other Urban	No	No	No	No	Very Low
	038A_C	Lt G Zamorski Dr	1.16	Public-City	Other Urban	No	No	No	No	Good
	038A_D	Atlantic St	1.68	Public-City	Transportation	No	Yes	No	No	Low
039	039A_A	Schiller St	2.67	Public-NJDOT	Transportation	No	No	No	No	Low
	039A_B	Schiller St	0.82	Private	Industrial	No	No	No	No	Low
040	040A_A	Richmond St	0.49	Public-Board of Ed	Other Urban	No	No	No	No	Very Low
	040A_B	Trenton Ave	1.48	Public-State	Other Urban	No	No	No	No	Fair
	040A_C	Trenton Ave	0.78	Public-County	Open Areas	Yes	Yes	No	No	Low
	040A_D	Trenton Ave	1.31	Public-State	Industrial	No	Yes	No	No	Low
041	041A_A	Morris Ave	0.35	Private	Residential High Density	No	No	No	No	Low
	041A_B	Morris Ave	0.33	Private	Residential High Density	No	No	No	No	Low
042	042A_A	Elizabeth Ave	0.84	Private	Commercial	No	No	No	No	Low
	042A_B	Winfield Scott Plz	0.28	Public-County	Commercial	No	No	No	No	Good
	042A_C	Jefferson Ave	0.75	Private	Commercial	No	No	No	No	Low
	042A_D	E Grand St	1.02	Private	Open Areas	No	No	No	No	Good
	042A_E	Martin L King Jr Plz	0.70	Public-City	Open Areas	No	No	No	No	Fair
Mis	Misc_A	Price St	0.99	Public-City	Commercial	No	No	No	No	Very Low
	Misc_B	Union St	2.30	Public-City	Commercial	No	No	No	No	Very Low
	Misc_C	Murray St	1.95	Public-City	Other Urban	No	No	No	No	Very Low

CSO Basin	Site ID	General Location	Site Area (acres)	Ownership Type	Predominant Land Use	Green Acres	100-yr Flood Hazard	Brownfield Site	Known Contamination	Favorability Rating
	Misc_D	E Jersey St	0.71	Private	Commercial	No	No	No	No	Good
	Misc_E	First St	2.15	Private	Industrial	No	No	No	No	Good
	Misc_F	S First St	2.21	Private	Industrial	No	No	No	No	Low



Legend

- STP JMEUC Treatment Plant
- P Pumping Station
- CSO Discharge
- Regulator
- Elizabeth City Hall
- Trunk Sewer Alignments
- Force Main Alignment
- CSO Outfall
- Municipal Boundary
- Regulator Basin

Interceptor Alignment

- Easterly Interceptor
- Westerly Interceptor
- JMEUC Trunk Sewer

CSO Control Sites

Site Suitability

- Good
- Fair
- Low
- Very Low

0 1,200 2,400 Feet

Map Document: (P:\340878\GIS\MXD\Alternatives Analysis\Siting_NorthWestElizabeth.mxd)
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FIGURE 6-2

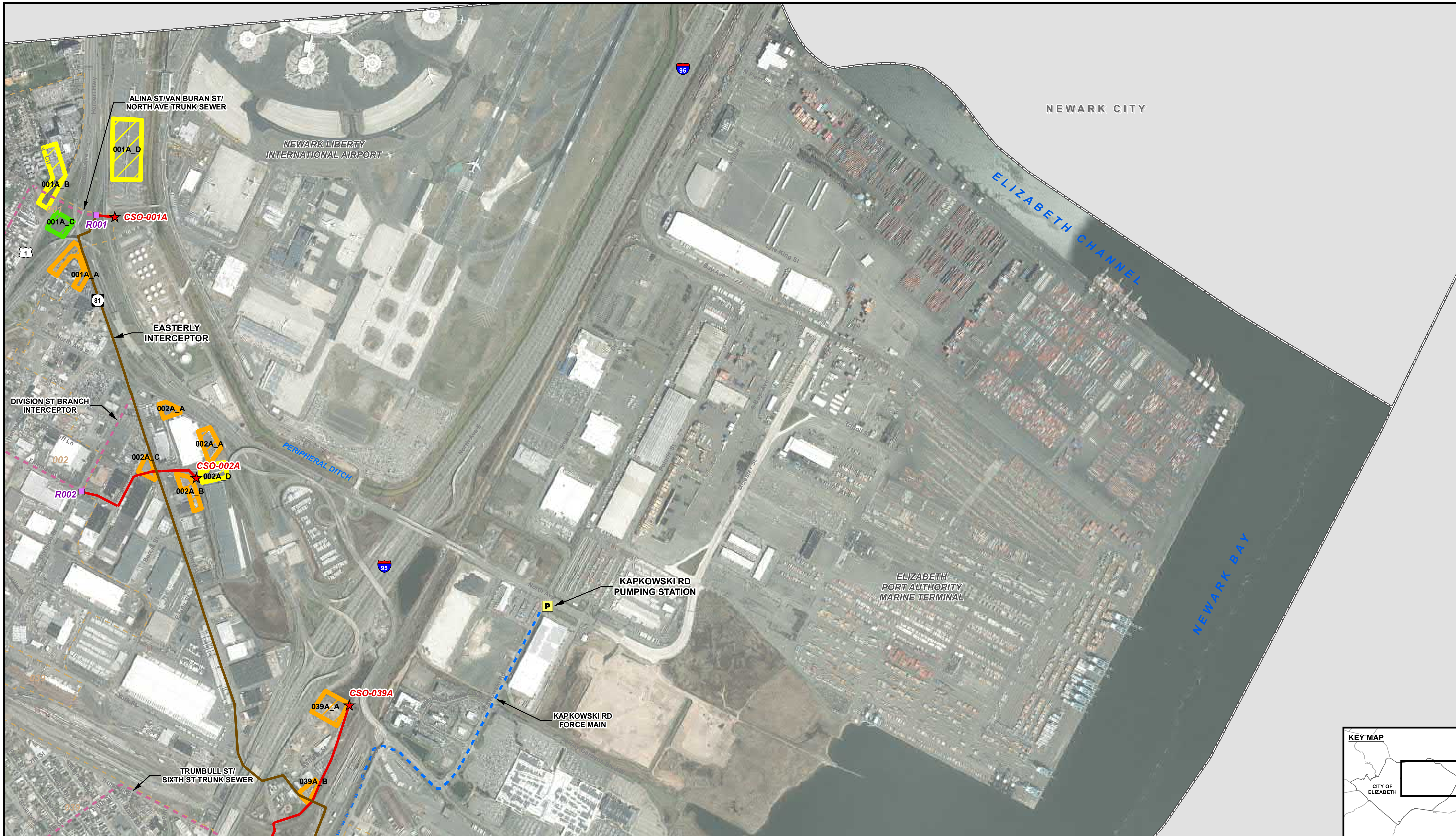
MOTT MACDONALD

111 Wood Avenue South
Iselin, New Jersey 08830-4112

CITY OF ELIZABETH
UNION COUNTY, NEW JERSEY
CSO LONG TERM CONTROL PROGRAM
ALTERNATIVES EVALUATION REPORT

PRELIMINARY SITING ANALYSIS
NORTHWEST ELIZABETH

Designed	Drawn	Checked	Approved	Date
AG	MM	AG	AG	6/13/2019



Legend

STP JMEUC Treatment Plant	Trunk Sewer Alignments	**Interceptor Alignment**	**CSO Control Sites Site Suitability**
P Pumping Station	Force Main Alignment	Easterly Interceptor	Good
CSO Discharge	CSO Outfall	Westerly Interceptor	Fair
Regulator	Municipal Boundary	JMEUC Trunk Sewer	Low
Elizabeth City Hall	Regulator Basin		Very Low

0 1,200 2,400 Feet

Map Document: (P:\340878\GIS\MXD\Alternatives Analysis\Siting_NorthEastElizabeth.mxd)
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FIGURE 6-3

MOTT MACDONALD
111 Wood Avenue South
Iselin, New Jersey 08830-4112

CITY OF ELIZABETH
UNION COUNTY, NEW JERSEY
CSO LONG TERM CONTROL PROGRAM
ALTERNATIVES EVALUATION REPORT

PRELIMINARY SITING ANALYSIS
NORTHEAST ELIZABETH

Designed	Drawn	Checked	Approved	Date
AG	RJL	AG	AG	6/13/2019

Legend

- STP JMEUC Treatment Plant
- P Pumping Station
- CSO Discharge
- Regulator
- Elizabeth City Hall
- Trunk Sewer Alignments
- Force Main Alignment
- CSO Outfall
- Regulator Basin
- Municipal Boundary

Interceptor Alignment

- Easterly Interceptor
- Westerly Interceptor
- JMEUC Trunk Sewer

CSO Control Sites Site Suitability

- Good
- Fair
- Low
- Very Low

0 1,200 2,400 Feet

Map Document: (P:\340878\GIS\MXD\Alternatives Analysis\Siting_South.mxd) 6/13/2019 10:15:33 AM mac82712

MOTT MACDONALD
111 Wood Avenue South
Iselin, New Jersey 08830-4112

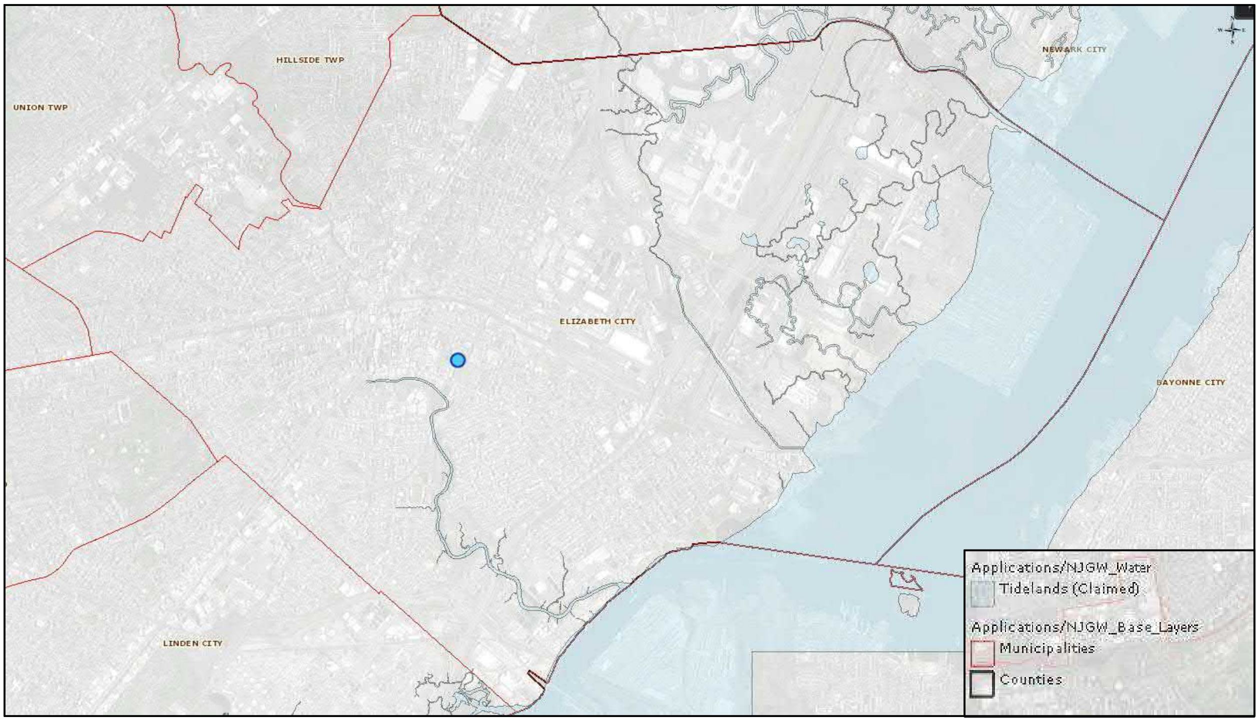
CITY OF ELIZABETH
UNION COUNTY, NEW JERSEY
CSO LONG TERM CONTROL PROGRAM
ALTERNATIVES EVALUATION REPORT

FIGURE 6-4
PRELIMINARY SITING ANALYSIS SOUTH ELIZABETH

Designed	Drawn	Checked	Approved	Date
AG	MM	AG	AG	6/13/2019

FIGURE 6-4

- 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 in Elizabeth is tidal, much of the work will be eligible for permit-by-rule, however certain facilities may require individual permits. While most areas within Elizabeth 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.
- Treatment Works Approval – 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 the 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 Management Rules (N.J.A.C. 7:8) primarily concern themselves with stormwater quantity, quality and recharge. Since Elizabeth 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 not be an issue since Elizabeth is highly urbanized. 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 total suspended solids (TSS) removal. Proposed changes to the Stormwater Management Rules include this NJDEP policy on sewer separation and add green stormwater infrastructure requirements to 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. Coordination with USACE would be required for any work along the Elizabeth River Flood Control Project, which includes the earthen levee and concrete flume sections. Plan approval from USACE would need to be obtained and to ensure suitable restoration of any ponding areas impacted by any proposed CSO-related improvements.
- 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 that 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 Tidelands Resource Council in a process that take several months, and in case of granting riparian rights, the appraised market value must be paid to the State. Figure 6-5 provides mapping of claimed tidelands as retrieved from the NJ-Geo Web application maintained by NJDEP. It appears that the State may have tidelands claims along the Elizabeth River and Arthur Kill, it is not known if any of these claimed tidelands have been granted in the past.



Source: NJDEP, NJ-Geo Web (retrieved May 2019)

Figure 6-5: New Jersey Claimed Tidelands, Elizabeth City

- Soil Conservation District (SCD) Certification – The New Jersey Department of Agriculture implements the New Jersey Soil Erosion and Sediment Control Act (N.J.S.A. 4:24-39). The program is administered by the local soil conservation district in this case the Somerset-Union 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.
- 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, and road opening permits. It is assumed that since the LTCP will be conducted by the City or JMEUC, they will likely facilitate obtaining these approvals. As such, local permits will not be considered a major obstacle.
- Green Acres – 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, it is suggested that Green Acres sites be considered to address the stormwater within the Green Acres property only through green infrastructure which is allowable under the current regulations. NJDEP may be 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 as well as State highways in Elizabeth which could be impacted by construction.

- Railroad Occupancy – A number of industrial rail lines as well as tracks associated with NJ Transit and Amtrak are located in Elizabeth. Agreements to acquire or occupy rail rights-of-way are difficult, expensive and time consuming to obtain.

6.3 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 City and JMEUC, as applicable, will be responsible for the operation and maintenance for LTCP facilities. Therefore, the permittees must be able to acquire (purchase) the property on which the facilities are sited 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 will 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 constructability (e.g., tunnel excavation) and technology performance (e.g., infiltration rates).
- Compatibility with existing infrastructure – Any know existing structures or utilities that would need to be relocated or decommissioned should be considered. 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.

6.4 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 required construction methods can serve to reduce or augment these concerns. For example, an alternative with significant pile driving needs 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, 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 seen as 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 perceived as interfering with the business operations or an anticipated redevelopment program may not be accepted in a commercial area. Communities also will likely object to facilities perceived to produce additional odor, noise, or other operational issues.
- Traffic impacts – Traffic impact may occur during construction and after construction. During construction, consideration must be given to the location and length of time of the impacts. For

impacts that may persist after construction, public acceptance could depend in 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 – The selection of projects should demonstrate that the impacts are not skewed towards areas of 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. As such, projects that directly impact community resources by taking part of the community property, impede its access or function, or affect emergency service response or key routes to hospitals should be avoided.

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, the degree of potential public opposition and associated risks to project implementation should be considered.

6.5 Performance Consideration

There is no true indication that a proposed technology will produce the desired results until it is implemented. This uncertainty can be greatly mitigated through the selection of the technology. Some considerations are:

- Past performance – The technology should be well tested with a history of successful applications to CSO and reliable data supporting its performance.
- Performance Flexibility – CSO flows are known for rapid changes in both quantity and quality. The selected technology should be able to accommodate the design conditions as well as 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 therefore 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 preplanned conditions.
- Reliability – While a technology may be effective, it must function consistently to be successful. CSO flows create a harsh environment for equipment. Wastewater process equipment typically functions under continual use, whereas CSO discharges are intermittent, which can lead to equipment operational issues, such as seizing between uses.

6.6 Levels of Control

The magnitude of the facilities in terms of either (1) CSO volume managed or (2) allowable frequency of uncontrolled events is the primary driver of both cost and effectiveness. Accordingly, a procedure was developed to achieve the desired control objectives, in this case limiting the uncontrolled overflows to 0, 4, 8, 12 or 20 during the Typical Year. The permit requires the levels of control to be established on the basis of the hydraulically connected system. Prior to the evaluation, it was necessary to determine for the CSS what storm events must be controlled for each level of control. Since the LTCP may incorporate 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 (i.e. uncontrolled events). For example, for a single outfall, to achieve 4 overflows with storage, the fifth largest storm would need to be stored and ultimately sent to the WWTF for treatment. However, since sewersheds respond to precipitation differently due to sewer system characteristics such as land use, size, and shape, and each storm has unique hydrologic characteristics such as duration and peak intensity, the top 4 storms may not produce the same responses at every sewershed and the storms categorized as top 4 for a sewershed may differ from sewershed to sewershed.

For this evaluation, 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 system-wide by overflow volume are listed in Figure 6-6, 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 system-wide goal of four overflows to meet the objectives of the overall LTCP.

	Rank	Event No.	Total CSO Vol (MG)	Event Start	Event End
Top 4 Storm Events by Overflow	1	44	145.0	9/28/2004 9:15	9/29/2004 5:08
	2	41	89.6	9/8/2004 4:30	9/9/2004 20:19
	3	43	63.2	9/18/2004 7:00	9/18/2004 13:58
	4	32	62.1	7/18/2004 16:30	7/18/2004 23:43
Top 8 Storm Events by Overflow	5	27	54.3	6/25/2004 17:00	6/25/2004 23:18
	6	51	54.0	11/28/2004 7:00	11/28/2004 15:29
	7	30	43.8	7/12/2004 11:30	7/13/2004 6:46
	8	19	42.6	5/12/2004 15:30	5/12/2004 20:40
Top 12 Storm Events by Overflow	9	6	40.5	2/6/2004 8:00	2/6/2004 23:15
	10	33	40.2	7/23/2004 11:45	7/23/2004 23:33
	11	14	39.5	4/12/2004 18:30	4/14/2004 18:34
	12	34	34.1	7/27/2004 16:15	7/28/2004 1:44
Top 20 Storm Events by Overflow	13	15	31.2	4/26/2004 2:30	4/27/2004 1:56
	14	39	29.9	8/14/2004 22:45	8/16/2004 9:11
	15	29	26.7	7/5/2004 2:45	7/5/2004 15:14
	16	40	25.8	8/21/2004 13:15	8/21/2004 17:50
	17	47	23.4	11/4/2004 14:15	11/4/2004 23:58
	18	52	19.3	12/1/2004 4:45	12/1/2004 14:43
	19	48	19.2	11/12/2004 9:15	11/13/2004 5:13
	20	18	18.6	5/10/2004 23:45	5/11/2004 3:29

Figure 6-6: Top 20 Overflow Events for Existing Conditions Typical Year

6.7 Basis of Cost Estimates

The LTCP development process requires that the permittees evaluate a variety of CSO control alternatives and part of this analysis is the evaluation of costs for each alternative, at different CSO control levels. The sections below outline the basis and assumptions upon which the cost estimates have been developed.

6.7.1 Cost Estimating Approach

Cost estimates for the CSO control programs have been developed for the alternatives comparison. The costs provided are meant to provide an order of magnitude estimate and are considered Class 5 estimates as defined by the Association for the Advancement of Cost Engineering International (AACE International). Class 5 estimates are prepared for any number of strategic business planning purposes, such as long-range capital planning, project screening, and feasibility review. Due to the low level of project definition and information available for the estimate preparation, the accuracy range of Class 5 estimates is classified as -50% to +100%. A 50% contingency is typically applied to such construction cost estimates to reflect the planning nature of the project data and engineering.

The estimates have been developed specifically for the configurations of the alternatives that have been described. It is noted that modifications to these alternatives or their configurations may impact the cost. The information and costs presented in this report are for planning purposes only, and all assumptions and information must be verified in subsequent stages.

The program costs are presented as follows:

- Capital cost – including construction costs with contingency, land acquisition, and non-construction project costs.
    - Construction costs – based on reference cost curves, guidance manual, past project experience, and specific technology cost estimates. These costs are intended to include contractor's general conditions, overhead, and profit. A 50% construction cost contingency has been applied. Specific costs for special site considerations, such as removal of contaminated material and utility relocations, generally were not included.
    - Land acquisition costs – based on the area required for facilities construction.
    - Non-construction costs – an allowance equal to 25% of the total construction and land acquisition costs has been applied for planning, engineering services during design and construction, permitting, legal and administrative expenses.
- Annual operations and maintenance (O&M) costs – annual costs for labor, power, chemicals, parts, equipment overhauls, and other supplies and services to operate and maintain the facilities.
- Life-cycle costs expressed as total present worth (TPW) – for a period of twenty years, i.e. to 2040, with an 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 single cost for reducing CSO events to zero per year. In addition to itemized cost items, the costs are presented as dollars per CSO gallon captured or controlled ($/gal) to provide a point for comparison between alternatives.

References used to prepare the cost estimates are based on various baseline years. For consistency, the costs presented have been escalated to 2019 dollars.

6.7.2 Total Present Worth Calculations

To be consistent with other permittees in the NJ CSO Group, guidance distributed by the Passaic Valley Sewerage Commissioners (PVSC) to the NJ CSO Group was used to develop total present worth values to combine estimated annual O&M costs and capital costs for each control technology. A discount rate of 2.75% was used based on the 2018 Rate of Federal Water Projects noted by the Natural Resources Conservation Service (NRCS) Economics group, Department of the Interior, with an analysis period of 20 years. The following equation was then utilized to calculate the uniform payment series present worth (P/A) factor to convert from annual O&M costs to present worth.

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

This P/A factor was multiplied by the annual O&M costs and added to the total capital costs to obtain the total life cycle cost, or total present worth, for the 20-year period. For the given life cycle and interest rate, the P/A factor is 15.2. Salvage value was assumed to be $0, as it is assumed there is no resale value for CSO control technologies considered.

6.7.3 PVSC Updated Technical Guidance Manual

In 2004, NJDEP re-issued a New Jersey Pollutant Discharge Elimination System (NJPDES) 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. That analysis required the permittees to evaluate alternatives at each CSO point, including various treatment methods. To assist the NJ CSO Group permittees 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 costs. The original TGM was released in 2007.

The NJPDES CSO permits issued in 2015 require the permittees to continue the CSO LTCP development process and perform a CSO control alternatives evaluation to assist in selecting a control plan for implementation. While much of the information in the original TGM is still viable, a decade has passed since it was developed. As such, PVSC updated the TGM to reflect new information, updated costs, and new permit requirements such as the evaluation of green infrastructure. PVSC has distributed this updated TGM to the NJ CSO Group members for reference and use as appropriate. A copy of the Updated Technical Guidance Manual is provided as Appendix B.

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. PVSC included estimates for annual O&M costs for certain technologies in the TGM based on manufacturer recommendations and previous project experience. The TGM was used to develop capital costs for:

- 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 at each outfall location. Costs were developed for the different control levels.
- 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 at each outfall location. 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%, 10%, and 15% 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 a bioswale and permeable pavement which were provided in the TGM.
- Sewage treatment plant expansion for CSO treatment and disinfection – estimated based on cost curves provided in the PVSC TGM. Costs were developed for the upper limit of possible additional CSO treatment.

The TGM did not include cost information on sewer separation, in-pipe storage, and inflow/infiltration, and as such, other resources were consulted to develop cost estimates for these technologies. The TGM also had limited application for costing a tunnel for the City's required depth and configuration, thus additional supplementary cost information was obtained. Consolidation piping costs were added for applicable control programs based on general quantity estimates. Specific considerations and supplements from reference documents were used to fill in any gaps or assumptions from the TGM.

6.7.4 Other Cost Estimating Resources

For some of the CSO control alternatives, additional resources were consulted to develop a more detailed and comprehensive cost understanding. These are described as follows:

- Tunnel: The PVSC TGM provides a generalized cost curve for a storage tunnel of a given depth, however a tunneling specialist was consulted to obtain a more specific cost estimate. The conceptual-level Class 5 cost estimate was prepared as a contractor would prepare a bid estimate, based on labor, materials, equipment and supplies. The estimate was based on tunneling experience, local current prevailing wages and fringe benefits, other similar projects in the United States, and 2019 prices for items such as the tunnel boring machine (TBM) and concrete segments. The estimated construction costs include contractor direct and indirect costs to acquire a TBM and other equipment and construction of work shafts, drop structures, tunnel bore, precast concrete segments, lining, and dewatering pump station.
- Sewer separation: The approach for estimating the total projected conceptual cost of sewer separation was to derive unit costs (i.e., cost per basin area and cost per linear foot of sewer) for a number of drainage areas, and then to apply average unit costs to each CSS regulator basin. A proposed sanitary sewer system layout was developed for a number of 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 costing the new separate sanitary sewers for the representative basins. Treatment of stormwater runoff was not included in the cost estimate. The resulting unit costs for sewer separation construction, including contingency, was estimated at $285,000 per acre and $1,800 per linear foot of sewer main.
- Inflow/infiltration – Previous project experience from other locations were applied to develop costs for comparison on maximum achievable I/I reduction for the JMEUC service area. Details on the costing methods are provided in the relevant control program evaluation section.

6.7.5 Annual Operation and Maintenance Costs

Annual O&M costs for screening, pretreatment, and disinfection treatment technologies were estimated based on values included in the PVSC Updated TGM document. The document indicates that information on routine operation and maintenance activities was obtained from manufacturers for each treatment technology. The power requirements and estimated annual hours of equipment operation were applied to an estimated cost of power is $0.14 per kilowatt-hour. Tables are provided in the TGM for annual costs related to maintenance labor, chemical, filter media replacement, and other parts and supplies.

Operation and maintenance costs for green infrastructure, pump stations, storage tanks, tunnels, and conveyance pipelines/sewer separation were also coordinated with information prepared by PVSC. For green infrastructure, the annual O&M cost criteria of $8,000 per impervious acre managed is used per discussions with PVSC. While PVSC has suggested calculating operating costs and maintenance costs separately for the other technologies based on multiples of a continuous operating post and a percent of construction costs, respectively, a percent of construction cost to cover both operating and maintenance costs was selected for this study. Furthermore, given the scale of the required control facilities for this CSS, the percentages applied are lower than the percentages suggested by PVSC. Table 6-3 summarizes the applied annual O&M cost factors for these technologies.

Table 6-3: Operation & Maintenance Cost Factors for Certain Technologies

Item	Unit	Cost Basis (per year)
Green Infrastructure	Per impervious acre managed	$8,000
Pump Station	% of construction cost	1.0%
Storage	% of construction cost	1.0%
Tunnels	% of construction cost	0.5%

Conveyance Pipelines/Sewer Separation	% of construction cost	1.0%

6.7.6 Land Acquisition Costs

Significant property acquisition is anticipated to be required to construct certain CSO control programs due to the low site availability surrounding the CSO outfalls. Based on the urbanized environment and tax assessment data for properties in the area adjoining the regulators and outfalls, it is assumed that properties on average can be acquired for $80 per square foot for all locations. This approach provides a consistent basis of cost. The actual acquisition cost will depend on the existing owner's willingness to sell, with additional legal costs incurred if it is necessary to acquire the property through condemnation. The site history of contamination and future plans for development will also factor in the final price of acquisition.

6.7.7 Construction Cost Index

The costs were indexed to the Engineering News Record (ENR) Construction Cost Index (CCI) for January 2019, with a corresponding national ENR-CCI value of 11,205.

Section 7
CSO Control Programs Evaluation

In this section, the potentially applicable combined sewer overflow (CSO) control technologies as screened in Section 5 are formulated into control programs and are evaluated per the criteria described in Section 6. The control programs evaluated include strategies for each CSO basin as well as alternatives for system-wide improvements. The discussion herein describes the alternative CSO control programs and the estimated implementation costs for comparison. An analysis of the potential impact of significant indirect users (SIU) discharges is also presented.

The seven (7) CSO control programs evaluated are:

1. Complete sewer separation.
2. Satellite CSO treatment facilities.
3. Pump station and sewage treatment plant (STP) expansion
4. Satellite storage facilities.
5. Tunnel storage and secondary controls.
6. Green infrastructure.
7. Infiltration/Inflow (I/I) reduction

Part IV.G.4.d. of the Elizabeth CSO permit requires a list of control alternatives evaluated for each CSO, as such the table below summarizes the level of detail to which each control program has been evaluated, either on an outfall-by-outfall basis, or as a system-wide basis. As demonstrated below, Control Programs 3, 6, and 7 (STP expansion, green infrastructure and I/I reduction, respectively) were evaluated on a system-wide basis, while Control Programs 1, 2, 4, and 5 (sewer separation, satellite treatment, satellite storage and tunnel storage, respectively) were evaluated based on siting, sizing, level of control, etc. required for specific outfalls in the CSS system. It is noted that Control Program 5 Tunnel Storage incorporates tank storage for Outfalls 001A and 002A and sewer separation for Outfall 037A. Each of the control programs are presented in detail in the subsequent sections.

Table 7-1: Control Program Level of Evaluation, by CSO Basin or Systemwide

CSO Basin	Control Program						
	1	2	3	4	5	6	7
	Sewer Separation	Satellite Treatment	STP Expansion	Satellite Storage	Tunnel Storage	Green Infrastructure	I/I Reduction
System-wide	N/A	N/A	✓	N/A	N/A	✓	✓
001	✓	✓	N/A	✓	✓ (Tank storage proposed for this basin as part of overall tunnel storage alternative)	N/A	N/A
002	✓	✓	N/A	✓	✓ (Tank storage proposed for this basin as part of overall tunnel storage alternative)	N/A	N/A
003	✓	✓	N/A	✓	✓	N/A	N/A
005	✓	✓	N/A	✓	✓	N/A	N/A
008	✓	✓	N/A	✓	✓	N/A	N/A

CSO Basin	Control Program						
	1	2	3	4	5	6	7
	Sewer Separation	Satellite Treatment	STP Expansion	Satellite Storage	Tunnel Storage	Green Infrastructure	I/I Reduction
010	✓	✓	N/A	✓	✓	N/A	N/A
012	✓	✓	N/A	✓	✓	N/A	N/A
013	✓	✓	N/A	✓	✓	N/A	N/A
014	✓	✓	N/A	✓	✓	N/A	N/A
016	✓	✓	N/A	✓	✓	N/A	N/A
021	✓	✓	N/A	✓	✓	N/A	N/A
022	✓	✓	N/A	✓	✓	N/A	N/A
026	✓	✓	N/A	✓	✓	N/A	N/A
027	✓	✓	N/A	✓	✓	N/A	N/A
028	✓	✓	N/A	✓	✓	N/A	N/A
029	✓	✓	N/A	✓	✓	N/A	N/A
030	✓	✓	N/A	✓	✓	N/A	N/A
031	✓	✓	N/A	✓	✓	N/A	N/A
032	✓	✓	N/A	✓	✓	N/A	N/A
034	✓	✓	N/A	✓	✓	N/A	N/A
035	✓	✓	N/A	✓	✓	N/A	N/A
036	✓	✓	N/A	✓	✓	N/A	N/A
037	✓	✓	N/A	✓	✓ (Sewer separation proposed for this basin as part of overall tunnel storage alternative)	N/A	N/A
038	✓	✓	N/A	✓	✓	N/A	N/A
039	✓	✓	N/A	✓	✓	N/A	N/A
040	✓	✓	N/A	✓	✓	N/A	N/A
041	✓	✓	N/A	✓	✓	N/A	N/A
042	✓	✓	N/A	✓	✓	N/A	N/A
043	✓	✓	N/A	✓	✓	N/A	N/A

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7.1 Control Program 1: Complete Sewer Separation

7.1.1 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 City of Elizabeth from being a CSO community.

The benefits of this alternative include:

- 100% CSO elimination, although the discharge of urban storm runoff through the existing outfalls would remain.
- The majority of the work remains in public right-of-way and minimal additional easement and land acquisition would be required.
- Opportunity for renewal of other municipal utilities and road reconstruction.

The challenges include:

- Highly disruptive to roads and traffic, broadly affecting residents and businesses.
- Scale of construction (i.e., over 100 miles of roads would be affected).
- Reconnection of every building sewer sanitary sewer lateral on each street would be required.
- Private property infiltration and inflow sources would have to be separated from the existing building sewers connected to the new sanitary sewer main. Coordination with private property owners and site access would be necessary to identify these I/I sources, and extensive private property disruption could be required to separate drainage from sewage on the property.
- Additional maintenance costs for new sanitary sewer collection system.
- Treatment of the separated stormwater discharge from the outfalls likely will be required in the future.

The sewer separation alternative has been evaluated on a sewershed-by-sewershed basis, however the overall objective under this control program consists of full sewer separation system-wide.

7.1.2 Analysis

New sanitary sewers would be constructed parallel to existing combined sewers, which are located within existing right-of-way, easements, and publicly owned property. Thus, siting for this alternative is straight-forward as the alignment for the new infrastructure would primarily fall within the public right-of-way . Existing easements may need to be modified or expanded to incorporate the construction and maintenance requirements for the new sewers. Additional easements would only be required where the proposed sewers must be routed outside the existing right-of-way due to special circumstances.

Sewer separation has been implemented in parts of the City of Elizabeth and other cities. Portions of the City that have undergone some degree of sewer separation include the Midtown Elizabeth Redevelopment area, Jefferson Avenue and Hampton Place area, and Norwood Terrace and Montgomery Street area. After the new sanitary sewer is installed, it will be able to convey the sanitary flows through the system. The associated operation and maintenance are familiar to sewer department personnel and the system performance is reliable.

7.1.3 Institutional Issues

The institutional issues surrounding sewer separation are typical of large-scale construction projects in an urban area. Construction of the facilities associated with this control program will require planning and land use permits, including some or all of the following, depending on specific locations of sewer separation projects:

- Waterfront Development Permit
- Flood Hazard Area Permit
- USACE Nationwide 404 Permit
- Local Permits
- NJDEP Treatment Works Approval

These permits are standard permits, so they would not be expected to greatly extend the project schedule or add excessive risk to the project.

In addition, it is noted that separating stormwater flow from sanitary 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 current NJDEP permit approval practices, total suspended solids (TSS) removal requirements have been applied to sewer separation projects where modifications to the stormwater outfalls are proposed. Recently proposed stormwater regulations include increased treatment requirements for creating separately sewered areas that would greatly increase the costs and impacts of performing separation.

7.1.4 Implementability

In terms of land acquisition, this alternative ranks highly, given that new public infrastructure would be located within the existing right-of-way and little to no acquisition of private land would be required. 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 City and lengthy traffic detours over the course of construction. Assuming this alternative will be implemented over the course of 30 years, about 116 acres, 3.4 miles of sewers, or 50 blocks of roads would need to be addressed each year. In addition to the approximately 100 miles of sewers that would have to be installed, reconnection and possibly redirection of the sanitary service lateral from each residence and business will also be a very extensive undertaking. As mentioned previously, the sewer separation program may involve modifying existing outfalls for the separate stormwater sewer system, which would probably have to be controlled and treated in the future due to anticipated stormwater regulations, thereby adding more cost and complexity to the solution.

By creating a new separate sanitary sewer system, the volume of I/I from private properties connected will have a large impact on the hydraulic design and operation of the sanitary sewer system. The level of sewer separation that can be achieved is affected by the amount of private inflow and infiltration that can be directed away from the sanitary sewer. Identification of private inflow sources requires inspections of private properties and cooperation from property owners. The redirection of sanitary flow or storm flow from existing buildings and site improvements to the proper public sewer main will involve construction work on private property, which creates significant implementation challenges and potential liabilities related to site restoration, property damage, and runoff infiltration, surface accumulation, or flooding.

Given the magnitude of impacted roadways, the sewer separation program will be affected to a greater extent than other programs by typical utility construction issues, such as handling of excavated materials, disposal of contaminated soils, and relocation of conflicting or impacted utilities. Much of the roadway and sidewalk areas would be disturbed by excavation for the new sewer construction, service lateral reinstatement, and utility relocations. Final restoration work typically includes full width milling and paving, concrete sidewalk, driveway apron, and curb replacement, and pavement marking. These construction items add significant time and costs to sewer separation projects.

7.1.5 Public Acceptance

The construction required for sewer separation is large and invasive, making public acceptance of the program a significant concern. Installation of a new sanitary sewer system and connections will result in road closures and 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. Negative resident and business reactions related to impacts on school bus routes, public transportation, increased commuting time, lost business revenue, and private property damages would likely accompany the massive and extended construction program. Moreover, the public may not prefer this alternative because of its high cost.

Following construction, the public reaction to sewer separation may be more favorable because the resulting facilities would be underground and consist of typical municipal sewer infrastructure.

7.1.6 Performance Summary

The Typical Year CSO performance of Control Program 1 and comparison to the 2015 baseline performance is summarized in Table 7-2.

Table 7-2: Control Program 1 – Sewer Separation, Performance Summary

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
001A	42	86.3	432	0	0.0	0	-42	-86.3	-432
002A	35	32.4	224	0	0.0	0	-35	-32.4	-224
003A	43	60.7	285	0	0.0	0	-43	-60.7	-285
005A	54	96.6	593	0	0.0	0	-54	-96.6	-593
008A	36	9.6	302	0	0.0	0	-36	-9.6	-302
010A	42	17.2	271	0	0.0	0	-42	-17.2	-271
012A	44	5.8	355	0	0.0	0	-44	-5.8	-355
013A	42	16.9	313	0	0.0	0	-42	-16.9	-313
014A	13	1.1	16	0	0.0	0	-13	-1.1	-16
016A	46	16.7	367	0	0.0	0	-46	-16.7	-367
021A	19	1.4	32	0	0.0	0	-19	-1.4	-32
022A	46	71.3	591	0	0.0	0	-46	-71.3	-591
026A	53	53.2	613	0	0.0	0	-53	-53.2	-613
027A	25	27.7	378	0	0.0	0	-25	-27.7	-378
028A	35	35.4	514	0	0.0	0	-35	-35.4	-514
029A	39	44.7	474	0	0.0	0	-39	-44.7	-474
030A	11	2.2	19	0	0.0	0	-11	-2.2	-19
031A	35	15.4	266	0	0.0	0	-35	-15.4	-266
032A	26	7.4	83	0	0.0	0	-26	-7.4	-83
034A	44	77.7	404	0	0.0	0	-44	-77.7	-404
035A	35	42.6	307	0	0.0	0	-35	-42.6	-307
036A	30	43.6	240	0	0.0	0	-30	-43.6	-240
037A	44	64.6	463	0	0.0	0	-44	-64.6	-463
038A	30	8.6	224	0	0.0	0	-30	-8.6	-224
039A	27	9.9	88	0	0.0	0	-27	-9.9	-88
040A	42	16.3	262	0	0.0	0	-42	-16.3	-262
041A	53	191.9	591	0	0.0	0	-53	-191.9	-591
042A	19	11.5	54	0	0.0	0	-19	-11.5	-54
043A	3	0.2	1	0	0.0	0	-3	-0.2	-1
Total	**-**	**1068.5**	**-**	**-**	**0.0**	**-**	**-**	**-1068.5**	**-**

MG = million gallons

7.1.7 Cost Summary

The total capital costs for sewer separation by CSO outfall basin are presented in Table 7-3. These values, in 2019 dollars, include construction costs, with contingency, and an allowance for engineering, legal and other non-construction costs. No land acquisition costs or treatment of stormwater runoff costs were added. The construction costs shown in the table were calculated using the basin areas and a unit cost of $285,000 per acre. The construction costs calculated based on the sewer length within each basin were similar overall, but were slightly higher for certain high-density residential basins and lower for other predominantly industrial land use basins. The cost differences between the 2 methods for the basins balanced out in the aggregate, with a total capital cost using sewer lengths estimated at $1.23 billion, versus $1.24 billion based on basin area.

Table 7-3: Control Program 1 – Sewer Separation Costs by CSO Basin

Outfall No.	Total Capital Cost ($ Million)	Annual O&M Cost ($ Million)	Total Present Worth ($ Million)
001A	$156.4	$1.251	$175.4
002A	$79.4	$0.635	$89.1
003A	$120.9	$0.967	$135.6
005A	$67.4	$0.539	$75.6
008A	$8.2	$0.066	$9.2
010A	$27.2	$0.217	$30.5
012A	$3.3	$0.026	$3.7
013A	$20.6	$0.165	$23.1
014A	$4.4	$0.035	$4.9
016A	$13.6	$0.109	$15.2
021A	$1.0	$0.008	$1.1
022A	$60.0	$0.480	$67.2
026A	$39.4	$0.315	$44.2
027A	$77.0	$0.616	$86.4
028A	Included with 027A		
029A	$27.2	$0.217	$30.5
030A	$6.8	$0.055	$7.7
031A	$21.2	$0.170	$23.8
032A	$23.2	$0.185	$26.0
034A	$63.5	$0.508	$71.3
035A	$42.8	$0.342	$48.0
036A	$74.6	$0.597	$83.7
037A	$30.8	$0.246	$34.5
038A	$22.8	$0.182	$25.5
039A	$87.2	$0.698	$97.8
040A	$12.5	$0.100	$14.0
041A	$84.8	$0.679	$95.1
042A	$68.2	$0.546	$76.5
043A	Included in 035A		
Total	**$1,244**	**$9.954**	**$1,396**

Table 7-3 also provides the cost analysis including the annual operating and maintenance (O&M) costs and the total present worth (TPW) calculated for a 20-year analysis period, according to the parameters outlined in Section 6.7. The TPW for sewer separation varies from $1.1 million for Outfall 021A to $175.4 million for Outfall 001A. The TPW for complete sewer separation of all outfalls under this control program is $1.396 billion.

The overall Class 5 (+100%, -50%) cost estimate for Control Program 1 are summarized in Table 7-4. As complete sewer separation is proposed, with the removal of all sanitary sewage from the outfall discharges, the equivalent number of CSO overflow events upon completion of the program is 0 and values for the other control levels are not applicable. The corresponding CSO volume reduction against the 2015 baseline is indicated and the estimated control program cost in terms of TPW value is normalized to a cost per gallon abated to facilitate comparison to other alternatives.

Table 7-4: Control Program 1 – Sewer Separation, Cost Summary

Control Level, Equivalent to Noted Overflows per Year	0	4	8	12	20
Total Capital Cost ($ Million)	$1,244	NA	NA	NA	NA
Annual O&M Cost ($ Million)	$9.95	NA	NA	NA	NA
Total Present Worth ($ Million)	$1,396	NA	NA	NA	NA
CSO Volume Abated (MG)	1,068.5	NA	NA	NA	NA
Cost per Gallon Abated ($/gal)	$1.31	NA	NA	NA	NA

NA = Not applicable. MG = million gallons.

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7.2 Control Program 2: Satellite CSO Treatment Facilities

7.2.1 Description

This control program consists of siting a treatment facility near the point of discharge for each CSO outfall or group of nearby outfalls. According to the National CSO Control Policy, overflows that meet the minimum required treatment are no longer considered untreated overflows. Thus, by providing a treatment train capable of providing the minimum required treatment, which is the equivalent of primary treatment and disinfection, a CSO event is considered as a wet-weather event during which peak flow exceeds the design maximum for full treatment at the satellite facility.

The following proposed treatment train is considered for this control program evaluation:

1. Divert flows downstream of the regulator, and if possible downstream of the existing netting facility.
2. Provide fine screening (removal of solids greater than 0.5 inches) of the flows to remove additional floatables and coarse particles.
3. Provide interim pumping to offset the head losses associated with the treatment processes.
4. Provide high-rate primary treatment of the flows to remove solids in advance of disinfection. For evaluation purposes, the ActiFlo® clarification process by Veolia Water Technologies was used as a representative and applicable technology for such treatment.
5. Disinfection is then provided by peracetic acid, by providing a six-minute contact time.
6. The flow is then discharged through the existing outfall or possibly a modified outfall.

Figure 7-1 shows a representative site plan of the proposed satellite CSO treatment train as sized for Outfall 036A. The arrangement is typical for the proposed facilities, but the size of the treatment units would reflect the peak flow rates corresponding to the specific outfall. The treatment systems for this control program are considered for each CSO outfall, except that Outfalls 035A and 043A are combined due to the emergency relief function of Outfall 043A for that sewershed.

Grouping and consolidation of CSO outfalls for satellite treatment facilities can be evaluated as a variation to this control program. The treatment units would need to be sized for the combined peak flow rate from the consolidated outfalls and sites with sufficient space for the facilities would have to be acquired. Consolidation piping from the outfalls to the shared treatment locations would have to be routed and would be of significant size given the peak discharge flow rates. New outfall pipes typically would also be required as the consolidated flows would be greater than the capacity of any one existing outfall.

7.2.2 Analysis

The treatment facilities were sized based on the peak flow rates discharging from the outfalls as modeled from the Typical Year precipitation record using the future baseline network configuration. Table 7-5 indicates the maximum peak discharge rates for each outfall within the set of storm events to be controlled for the different control levels considered. As can be noted from this table, the treatment capacities required for a control level of 12 overflows per year are the same as that for 8 overflows per year. 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 event. As such, the sizing of satellite treatment facilities are often driven by a different set of storms than volume-based control programs and the storm generating the controlling peak discharge rate may be the same for a wide range of overflow events ranked according to CSO volume. To achieve a consistent level of control on the basis of peak flow typically requires a much higher volume to be treated than would have to be stored.

Figure 7-1: Satellite CSO Treatment Typical Site Plan

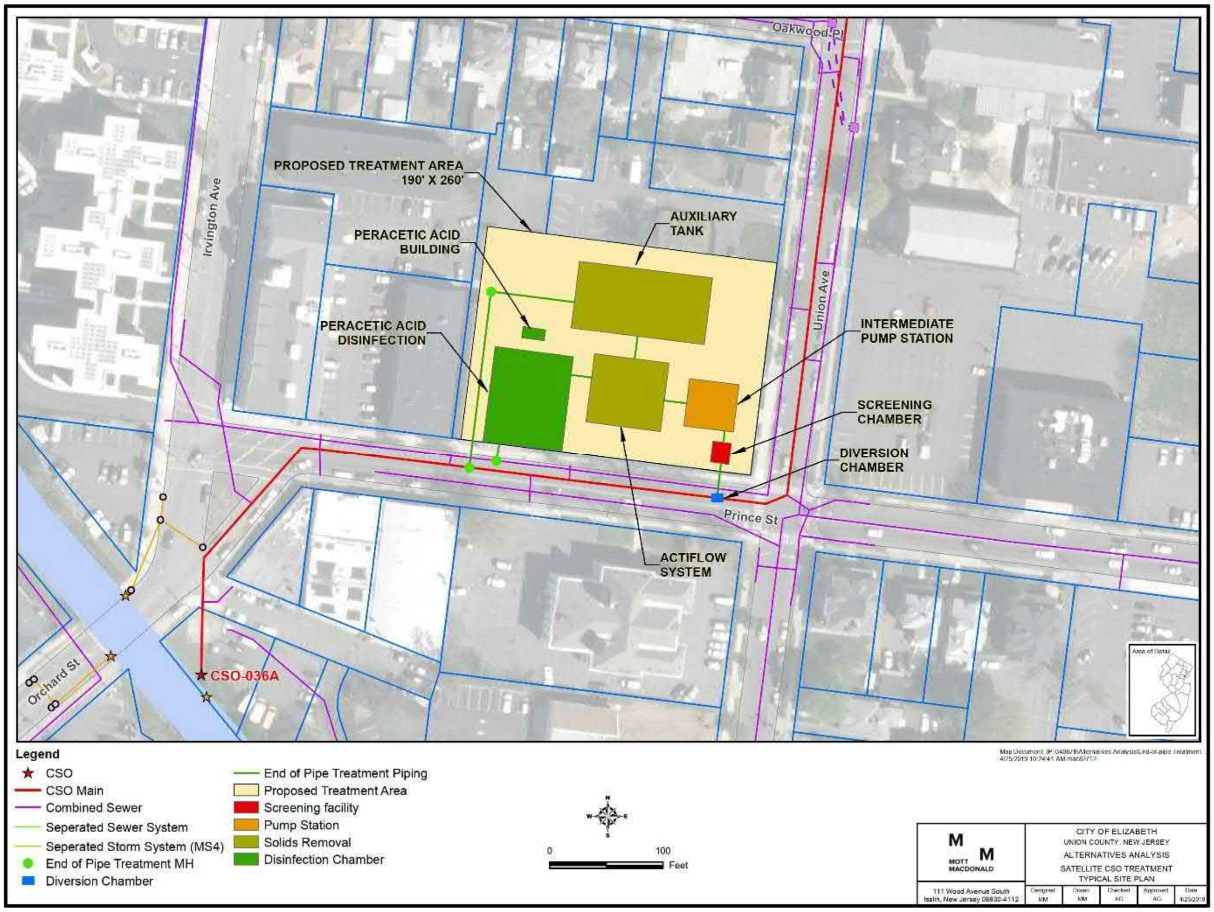

Table 7-5: Peak Discharge Rate by Control Level and Outfall

Outfall No.	Maximum Peak Discharge Rate (mgd)				
	Control Level (Overflows per Year)				
	0	4	8	12	20
001A	73.3	61.8	61.8	61.8	32.4
002A	62.0	61.0	53.1	53.1	24.1
003A	187.8	149.7	119.9	119.9	54.4
005A	61.2	54.4	43.6	43.6	27.1
008A	11.8	11.0	9.1	9.1	6.8
010A	31.8	25.2	23.8	23.8	14.2
012A	3.13	2.48	2.14	2.14	1.41
013A	20.8	19.9	19.5	19.5	13.9
014A	6.54	5.72	4.35	4.35	0.56
016A	28.0	23.8	22.3	22.3	12.5
021A	4.32	3.73	2.45	2.45	1.03
022A	51.9	48.4	40.2	40.2	19.4
026A	51.9	51.9	48.5	48.5	26.6
027A	42.9	30.9	29.5	29.5	14.8
028A	57.4	57.4	43.6	43.6	18.4
029A	60.5	60.5	49.9	49.9	23.9
030A	38.1	28.2	19.9	19.9	1.5

Outfall No.	Maximum Peak Discharge Rate (mgd)				
	Control Level (Overflows per Year)				
	0	4	8	12	20
031A	35.7	30.5	27.4	27.4	10.8
032A	40.7	31.5	29.0	29.0	11.0
034A	67.1	67.1	55.0	55.0	26.6
035A	47.4	43.9	33.5	33.5	17.5
036A	61.4	51.9	44.6	44.6	19.6
037A	46.5	39.2	31.0	31.0	17.2
038A	0.14	0.0	0.0	0.0	0.0
039A	17.0	15.9	14.7	14.7	8.3
040A	20.6	20.6	12.9	12.9	7.7
041A	146.5	135.0	108.0	108.0	58.4
042A	58.8	52.2	30.2	30.2	2.10
043A	2.25	2.02	0.0	0.0	0.0
Total	**1,338**	**1,186**	**980.0**	**980.0**	**472.5**

mgd = million gallons per day.

A sample hydrograph illustrating the overflow reduction analysis with satellite CSO treatment is shown in Figure 7-2. Based on the treatment sizing required for each site, the evaluation consists of diverting the flows from the CSO outfall to the treatment facility and once the outfall discharge has exceeded the treatment rate, the remaining flows are tracked as untreated overflow volume. Outfall flows were checked to make sure that overflows only occur for the number of events allowable for that level of control.

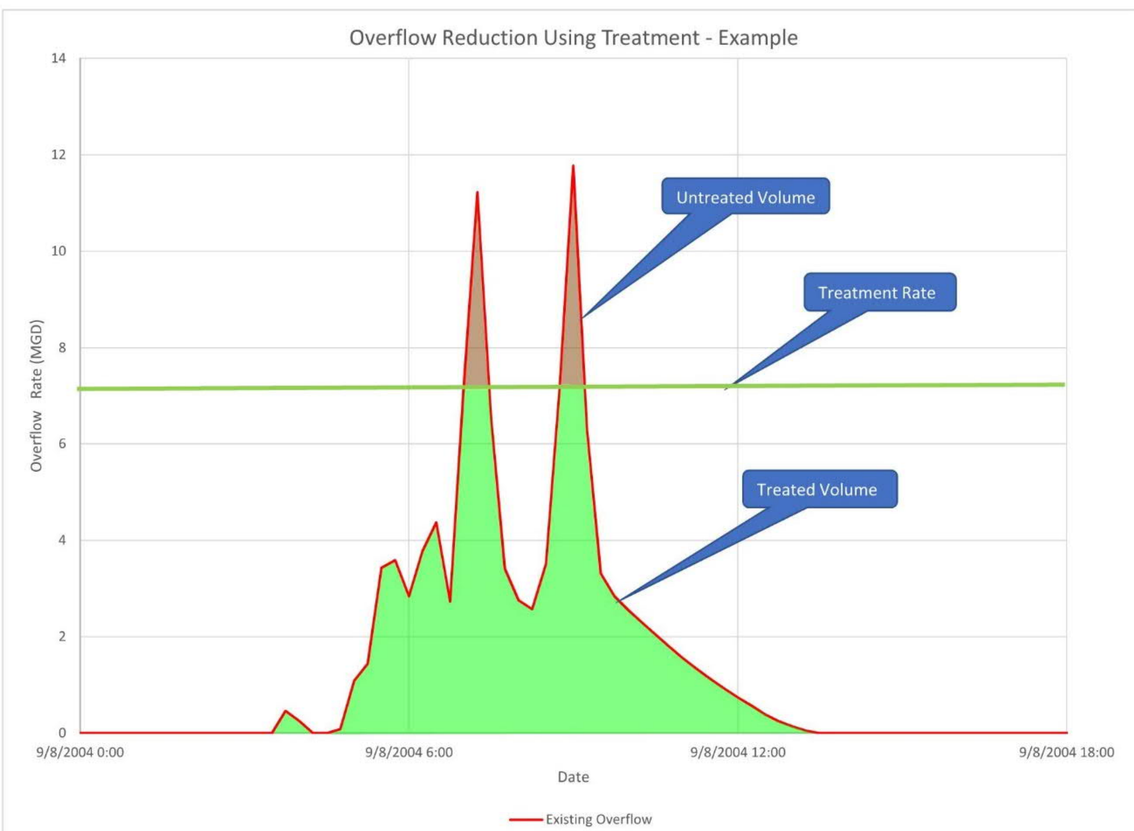

Figure 7-2: Sample Treatment Hydrograph

The preliminary siting analysis described in Section 6.1 demonstrated that given the dense existing development, ongoing and future redevelopment plans, and other land use constraints, there is a general lack of suitable available space for CSO control facilities along the outfall alignments. Accordingly, no specific sites are proposed for use at this time and this evaluation assumes that extensive land acquisition for the control program would be implemented, with the corresponding costs considered.

Table 7-6 tabulates the estimated area required for the satellite CSO treatment facilities for each control level treatment rate. The sizing is based on factors presented in the Updated Technical Guidance Manual prepared and distributed by the Passaic Valley Sewerage Commissioners (PVSC) for use by the NJ CSO Group members. The estimated footprints include allowances for screening, pumping, and disinfection support building structures, however additional land may be required for such siting considerations as vehicle circulation, access, parking and set-backs from abutting properties.

Table 7-6: Satellite Treatment Site Area Requirements by Control Level and Outfall

Outfall No.	Area Required (Acres)				
	Control Level (Overflows per Year)				
	0	4	8	12	20
001A	0.566	0.493	0.493	0.493	0.305
002A	0.494	0.487	0.437	0.437	0.251
003A	1.168	0.978	0.829	0.829	0.445
005A	0.489	0.445	0.376	0.376	0.271
008A	0.173	0.168	0.156	0.156	0.141
010A	0.301	0.259	0.250	0.250	0.188
012A	0.118	0.113	0.111	0.111	0.107
013A	0.231	0.225	0.222	0.222	0.187
014A	0.139	0.134	0.125	0.125	0.101
016A	0.277	0.249	0.240	0.240	0.178
021A	0.125	0.121	0.113	0.113	0.104
022A	0.429	0.407	0.354	0.354	0.222
026A	0.429	0.429	0.407	0.407	0.268
027A	0.371	0.295	0.286	0.286	0.192
028A	0.464	0.464	0.376	0.376	0.215
029A	0.484	0.484	0.417	0.417	0.250
030A	0.341	0.278	0.224	0.224	0.107
031A	0.326	0.292	0.273	0.273	0.167
032A	0.358	0.299	0.283	0.283	0.168
034A	0.526	0.526	0.449	0.449	0.268
035A	0.611	0.549	0.433	0.433	0.331
036A	0.490	0.429	0.382	0.382	0.223
037A	0.395	0.348	0.295	0.295	0.207
038A	0.000	0.000	0.000	0.000	0.000
039A	0.206	0.199	0.192	0.192	0.151
040A	0.229	0.229	0.180	0.180	0.147
041A	0.962	0.904	0.770	0.770	0.471
042A	0.473	0.431	0.291	0.291	0.111
043A	Included with 035A				
Total	**11.17**	**10.24**	**8.96**	**8.96**	**5.77**

7.2.3 Institutional Issues

The institutional issues surrounding satellite CSO treatment 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 required:

- Waterfront Development Permit
- Flood Hazard Area Permit
- USACE Nationwide 404 Permit
- Local Permits
- NJDEP Treatment Works Approval
- Soil Erosion and Sediment Control 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 the level of treatment proposed may need to be increased over time in response to more stringent water quality standards. This is because the effluent could contribute to pollution of the receiving waters. Future regulations could include increased treatment requirements that could greatly increase the costs and impacts of this alternative.

7.2.4 Implementability

Installation of satellite treatment facilities in urban areas can be challenging due to space and access limitations. Unlike satellite storage tanks, the top of satellite treatment facilities generally extend partially above grade level. As such, not as much excavation is required, reducing cost as well as the complexity of excavation in proximity to the foundation of nearby buildings. Further, groundwater is not as much of a concern as with subsurface storage tanks due to the shallower depths and corresponding reduced uplift forces. There is little available information on the soil conditions at the sites, however, given the depth to bedrock and proximity to the floodplain, soil conditions may be poor, and the facilities may need to be situated on piles.

7.2.5 Public Acceptance

Because the facilities proposed are generally above-grade, they have the potential to produce odors and noise, making them more difficult to site in residential and commercial areas. There may be concerns with odors, particularly for the outfalls in commercial and residential areas. Following construction, satellite treatment facilities may be less preferable to the public due to the permanent visibility of the above grade structures. It also uses land area that could otherwise be utilized by the community for other purposes. The construction required for satellite treatment is less than satellite storage tanks but is still large and invasive, making public acceptance of the project a concern. This is particularly true for outfalls located in heavily trafficked areas and on private property.

7.2.6 Performance Summary

To align with the system-wide levels of control of 0, 4, 8, 12 and 20 overflows per year, the satellite CSO treatment capacity for a given outfall was calculated as the maximum peak discharge rate within the set of storms allowed under the various levels of control. While the outfalls will continue to discharge many times a year, the flows will not be considered CSOs unless they exceed the treatment rate. The performance of Control Program 2 is summarized in Table 7-7 through Table 7-11, which present the results for the equivalent treatment for 0, 4, 8, 12, and 20 overflows per year.

Table 7-7: Control Program 2 - Satellite Treatment, Performance Summary for 0 Overflows

	Baseline 2015			Control Program			Change		
Outfall No.	**No. of Events**	**Volume (MG)**	**Duration (hours)**	**No. of Events**	**Volume (MG)**	**Duration (hours)**	**No. of Events**	**Volume (MG)**	**Duration (hours)**
001A	42	86.3	432	0	0.0	0	-42	-86.3	-432
002A	35	32.4	224	0	0.0	0	-35	-32.4	-224
003A	43	60.7	285	0	0.0	0	-43	-60.7	-285
005A	54	96.6	593	0	0.0	0	-54	-96.6	-593
008A	36	9.6	302	0	0.0	0	-36	-9.6	-302
010A	42	17.2	271	0	0.0	0	-42	-17.2	-271
012A	44	5.8	355	0	0.0	0	-44	-5.8	-355
013A	42	16.9	313	0	0.0	0	-42	-16.9	-313
014A	13	1.1	16	0	0.0	0	-13	-1.1	-16
016A	46	16.7	367	0	0.0	0	-46	-16.7	-367
021A	19	1.4	32	0	0.0	0	-19	-1.4	-32
022A	46	71.3	591	0	0.0	0	-46	-71.3	-591
026A	53	53.2	613	0	0.0	0	-53	-53.2	-613
027A	25	27.7	378	0	0.0	0	-25	-27.7	-378
028A	35	35.4	514	0	0.0	0	-35	-35.4	-514
029A	39	44.7	474	0	0.0	0	-39	-44.7	-474
030A	11	2.2	19	0	0.0	0	-11	-2.2	-19
031A	35	15.4	266	0	0.0	0	-35	-15.4	-266
032A	26	7.4	83	0	0.0	0	-26	-7.4	-83
034A	44	77.7	404	0	0.0	0	-44	-77.7	-404
035A	35	42.6	307	0	0.0	0	-35	-42.6	-307
036A	30	43.6	240	0	0.0	0	-30	-43.6	-240
037A	44	64.6	463	0	0.0	0	-44	-64.6	-463
038A	30	8.6	224	0	0.0	0	-30	-8.6	-224
039A	27	9.9	88	0	0.0	0	-27	-9.9	-88
040A	42	16.3	262	0	0.0	0	-42	-16.3	-262
041A	53	191.9	591	0	0.0	0	-53	-191.9	-591
042A	19	11.5	54	0	0.0	0	-19	-11.5	-54
043A	3	0.2	1	0	0.0	0	-3	-0.2	-1
Total	**-**	**1068.5**	**-**	**-**	**0.0**	**-**	**-**	**-1068.5**	**-**

Table 7-8: Control Program 2 - Satellite Treatment, Performance Summary for 4 Overflows

	Baseline 2015			Control Program			Change		
Outfall No.	**No. of Events**	**Volume (MG)**	**Duration (hours)**	**No. of Events**	**Volume (MG)**	**Duration (hours)**	**No. of Events**	**Volume (MG)**	**Duration (hours)**
001A	42	86.3	432	2	0.3	1	-40	-86.0	-431
002A	35	32.4	224	1	0.0	0	-34	-32.3	-224
003A	43	60.7	285	1	0.4	0	-42	-60.3	-284
005A	54	96.6	593	1	0.0	0	-53	-96.6	-592
008A	36	9.6	302	1	0.0	0	-35	-9.6	-301
010A	42	17.2	271	1	0.0	0	-41	-17.2	-271
012A	44	5.8	355	1	0.0	0	-43	-5.8	-355
013A	42	16.9	313	1	0.0	0	-41	-16.8	-313
014A	13	1.1	16	1	0.0	0	-12	-1.0	-16
016A	46	16.7	367	1	0.0	0	-45	-16.7	-367
021A	19	1.4	32	1	0.0	0	-18	-1.4	-32
022A	46	71.3	591	1	0.0	0	-45	-71.3	-591
026A	53	53.2	613	1	0.0	0	-52	-53.2	-613

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
027A	25	27.7	378	2	0.6	2	-23	-27.1	-376
028A	35	35.4	514	0	0.0	0	-35	-35.4	-514
029A	39	44.7	474	0	0.0	0	-39	-44.7	-474
030A	11	2.2	19	1	0.0	0	-10	-2.1	-19
031A	35	15.4	266	1	0.0	0	-34	-15.3	-266
032A	26	7.4	83	1	0.1	0	-25	-7.3	-83
034A	44	77.7	404	0	0.0	0	-44	-77.7	-404
035A	35	42.6	307	2	0.0	0	-33	-42.5	-306
036A	30	43.6	240	1	3.1	4	-29	-40.5	-236
037A	44	64.6	463	1	0.0	0	-43	-64.5	-462
038A	30	8.6	224	0	0.0	0	-30	-8.6	-224
039A	27	9.9	88	3	0.0	0	-24	-9.9	-88
040A	42	16.3	262	0	0.0	0	-42	-16.3	-262
041A	53	191.9	591	1	0.1	0	-52	-191.7	-591
042A	19	11.5	54	1	0.0	0	-18	-11.5	-54
043A	3	0.2	1	1	0.0	0	-2	-0.2	-1
Total	**-**	**1068.5**	**-**	**-**	**4.9**	**-**	**-**	**-1063.6**	**-**

Table 7-9: Control Program 2 - Satellite Treatment, Performance Summary for 8 Overflows

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
001A	42	86.3	432	2	0.3	1	-40	-86.0	-431
002A	35	32.4	224	4	0.3	1	-31	-32.1	-222
003A	43	60.7	285	3	1.4	1	-40	-59.4	-284
005A	54	96.6	593	4	0.4	2	-50	-96.2	-591
008A	36	9.6	302	4	0.0	1	-32	-9.6	-301
010A	42	17.2	271	2	0.1	1	-40	-17.1	-271
012A	44	5.8	355	3	0.0	0	-41	-5.8	-355
013A	42	16.9	313	2	0.0	0	-40	-16.8	-313
014A	13	1.1	16	5	0.0	1	-8	-1.0	-15
016A	46	16.7	367	2	0.0	1	-44	-16.7	-367
021A	19	1.4	32	4	0.0	1	-15	-1.4	-31
022A	46	71.3	591	3	0.2	1	-43	-71.1	-590
026A	53	53.2	613	2	0.1	1	-51	-53.1	-612
027A	25	27.7	378	3	0.7	3	-22	-27.0	-375
028A	35	35.4	514	4	0.5	1	-31	-34.9	-512
029A	39	44.7	474	4	0.3	1	-35	-44.4	-473
030A	11	2.2	19	3	0.2	1	-8	-2.0	-18
031A	35	15.4	266	3	0.1	1	-32	-15.3	-265
032A	26	7.4	83	2	0.1	0	-24	-7.3	-82
034A	44	77.7	404	4	0.4	2	-40	-77.3	-402
035A	35	42.6	307	3	0.6	2	-32	-42.0	-305
036A	30	43.6	240	2	4.4	5	-28	-39.1	-235
037A	44	64.6	463	4	0.3	1	-40	-64.2	-461
038A	30	8.6	224	3	0.0	0	-27	-8.6	-224
039A	27	9.9	88	5	0.0	0	-22	-9.9	-88
040A	42	16.3	262	5	0.4	3	-37	-15.9	-259
041A	53	191.9	591	3	1.2	2	-50	-190.7	-590

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
042A	19	11.5	54	4	0.8	2	-15	-10.7	-53
043A	3	0.2	1	3	0.0	1	0	-0.2	-1
Total	**-**	**1068.5**	**-**	**-**	**12.9**	**-**	**-**	**-1055.6**	**-**

Table 7-10: Control Program 2 - Satellite Treatment, Performance Summary for 12 Overflows

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
001A	42	86.3	432	2	0.3	1	-40	-86.0	-431
002A	35	32.4	224	4	0.3	1	-31	-32.1	-222
003A	43	60.7	285	3	1.4	1	-40	-59.4	-284
005A	54	96.6	593	4	0.4	2	-50	-96.2	-591
008A	36	9.6	302	4	0.0	1	-32	-9.6	-301
010A	42	17.2	271	2	0.1	1	-40	-17.1	-271
012A	44	5.8	355	3	0.0	0	-41	-5.8	-355
013A	42	16.9	313	2	0.0	0	-40	-16.8	-313
014A	13	1.1	16	5	0.0	1	-8	-1.0	-15
016A	46	16.7	367	2	0.0	1	-44	-16.7	-367
021A	19	1.4	32	4	0.0	1	-15	-1.4	-31
022A	46	71.3	591	3	0.2	1	-43	-71.1	-590
026A	53	53.2	613	2	0.1	1	-51	-53.1	-612
027A	25	27.7	378	3	0.7	3	-22	-27.0	-375
028A	35	35.4	514	4	0.5	1	-31	-34.9	-512
029A	39	44.7	474	4	0.3	1	-35	-44.4	-473
030A	11	2.2	19	3	0.2	1	-8	-2.0	-18
031A	35	15.4	266	3	0.1	1	-32	-15.3	-265
032A	26	7.4	83	2	0.1	0	-24	-7.3	-82
034A	44	77.7	404	4	0.4	2	-40	-77.3	-402
035A	35	42.6	307	3	0.6	2	-32	-42.0	-305
036A	30	43.6	240	2	4.4	5	-28	-39.1	-235
037A	44	64.6	463	4	0.3	1	-40	-64.2	-461
038A	30	8.6	224	3	0.0	0	-27	-8.6	-224
039A	27	9.9	88	5	0.0	0	-22	-9.9	-88
040A	42	16.3	262	5	0.4	3	-37	-15.9	-259
041A	53	191.9	591	3	1.2	2	-50	-190.7	-590
042A	19	11.5	54	4	0.8	2	-15	-10.7	-53
043A	3	0.2	1	3	0.0	1	0	-0.2	-1
Total	**-**	**1068.5**	**-**	**-**	**12.9**	**-**	**-**	**-1055.6**	**-**

Table 7-11: Control Program 2 - Satellite Treatment, Performance Summary – 20 Overflows

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
001A	42	86.3	432	11	9.5	15	-31	-76.8	-417
002A	35	32.4	224	12	4.6	7	-23	-27.7	-217
003A	43	60.7	285	11	11.4	8	-32	-49.3	-277
005A	54	96.6	593	10	2.9	8	-44	-93.7	-585
008A	36	9.6	302	6	0.3	4	-30	-9.3	-298

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
010A	42	17.2	271	9	0.7	4	-33	-16.5	-268
012A	44	5.8	355	7	0.0	1	-37	-5.8	-354
013A	42	16.9	313	9	0.7	5	-33	-16.1	-308
014A	13	1.1	16	10	0.6	9	-3	-0.4	-7
016A	46	16.7	367	9	0.8	4	-37	-15.9	-364
021A	19	1.4	32	9	0.2	7	-10	-1.3	-25
022A	46	71.3	591	13	4.8	14	-33	-66.5	-577
026A	53	53.2	613	11	2.9	6	-42	-50.3	-607
027A	25	27.7	378	11	5.3	16	-14	-22.4	-362
028A	35	35.4	514	12	5.0	10	-23	-30.5	-504
029A	39	44.7	474	14	4.6	9	-25	-40.0	-465
030A	11	2.2	19	9	1.7	5	-2	-0.5	-14
031A	35	15.4	266	12	2.5	8	-23	-12.8	-258
032A	26	7.4	83	9	1.5	4	-17	-5.9	-79
034A	44	77.7	404	12	8.3	16	-32	-69.4	-388
035A	35	42.6	307	12	3.5	10	-23	-39.1	-296
036A	30	43.6	240	11	14.9	18	-19	-28.7	-222
037A	44	64.6	463	10	2.1	7	-34	-62.5	-456
038A	30	8.6	224	3	0.0	0	-27	-8.6	-224
039A	27	9.9	88	11	1.0	10	-16	-8.8	-79
040A	42	16.3	262	13	1.5	10	-29	-14.7	-251
041A	53	191.9	591	10	12.0	12	-43	-179.8	-580
042A	19	11.5	54	11	8.8	17	-8	-2.8	-37
043A	3	0.2	1	3	0.0	1	0	-0.2	-1
Total	**-**	**1068.5**	**-**	**-**	**112.1**	**-**	**-**	**-956.4**	**-**

7.2.7 Cost Summary

The Class 5 (+100%, -50%) cost estimate for the satellite CSO treatment control program is summarized in Table 7-12.

Table 7-12: Control Program 2 – Satellite Treatment, Cost Summary

Control Level, Equivalent to Noted Overflows per Year	0	4	8	12	20
Total Construction Costs ($ Million)	$653.3	$606.3	$540.0	$540.0	$370.7
Land Acquisition Costs ($ Million)	$38.9	$35.7	$31.2	$31.2	$20.1
Non-Construction Costs ($ Million)	$173.0	$161.0	$143.0	$143.0	$98.0
Total Capital Cost ($ Million)	**$865.2**	**$803.0**	**$714.2**	**$714.2**	**$488.8**
Annual O&M Cost ($ Million)	$6.4	$6.1	$5.7	$5.7	$4.6
Total Present Worth ($ Million)	**$963.2**	**$896.0**	**$801.2**	**$801.2**	**$558.8**
CSO Volume Abated (MG)	1,069	1,064	1,056	1,056	956
Cost per Gallon Abated ($/gal)	$0.90	$0.84	$0.76	$0.76	$0.58

*Note: As described in Section 7.2.2, Table 7-5 indicates that the maximum peak discharge rates and thus the treatment capacities required for a control level of 12 overflows per year are the same as that for 8 overflows per year. As a result, the costs these two control levels are the same.

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7.3 Control Program 3: Pump Station and Treatment Plant Expansion

This section of the report describes and evaluates CSO control that can be achieved by expansion of the City of Elizabeth combined sewage pumping and conveyance capacity to deliver flow to the Joint Meeting of Essex and Union Counties (JMEUC) Wastewater Treatment Facility (WWTF) for treatment of additional wet weather combined sewage flow from the City of Elizabeth. There are two components of expanded treatment of combined sewer flows at the WWTF that have been evaluated:

Control Program 3A: Interim Plan for Increased CSO Treatment with Real Time Control
An interim plan that is based on changing the operation of the existing Trenton Avenue Pump Station (TAPS) to pump at the peak hydraulic capacity of the facility (55 mgd) has been developed and evaluated. This represents an increase of 19 mgd over the current peak pumping rate of 36 mgd as defined by the flow limit in the contractual agreement between the City of Elizabeth and JMEUC. In addition to a change in the contractual agreement, this change will also require upgrades to TAPS to improve the reliability of the facility to pump at the higher rate. In order to avoid stressing the plant during large wet weather events, the use of real time control (RTC) will enable higher flows to be pumped from TAPS without increasing peak flow rates for these large events above current levels. This will enable increased capture of combined sewer flows with no changes to the TAPS force main, JMEUC trunk sewers or WWTF required, as the existing force main, trunk sewers and WWTF can accept and treat flow at the increased TAPS pumping rate with RTC.

Control Program 3B: Expanded Wet-Weather Treatment for Combined Sewer Flows and CSO-Related Bypass
A long term plan to increase the capture and pumping of wet weather combined sewer flow at TAPS beyond the 55 mgd flow rate described above has also been developed and evaluated. At rates above roughly 55 mgd, additional pumping and force main capacity will need to be provided, along with additional treatment capacity at the WWTF. TAPS pumping rates up to 140 mgd have been evaluated, which will increase flow by as much as 104 mgd above the current pumping rate of 36 mgd. The potential use of a new CSO treatment process train has also been evaluated to treat the flow bypassed around the existing secondary treatment train and blend it with the normal plant effluent for discharge through the existing outfall to the Arthur Kill.

The basis for the evaluation of expanded treatment at the JMEUC WWTF is depicted graphically by Figure 7-3. This chart shows that a relatively large reduction in annual CSO volume (179 MG) can be achieved with the interim plan to increase flow to 55 mgd at TAPS. However, the incremental CSO capture drops off as TAPS pumping increases further, which also corresponds to an escalating level of system modifications required. This will be an important consideration in the final decision regarding the expanded peak flow capacity of TAPS in Control Program 3B.

The additional modifications to the combined sewer system associated with increasing the TAPS pump capacity and corresponding flow to treatment above 55 mgd are noted on Figure 7-3. For example, it is estimated that the size of the existing force main and pump station would be insufficient at approximately 65 to 70 mgd and a new expanded pump station and force main would be required. At about 85 mgd, substantial sections of the Westerly Interceptor, including the segments from Rahway Avenue to Clarkson Avenue, would need to be replaced. For flows above 90 mgd , further modifications to 10 regulators would be needed, while at about 120 mgd, undersized section of the Easterly Interceptor would need relief to match downstream conveyance capacity. New parallel interceptors along the eastern and western alignments throughout the system would be needed at approximately 140 mgd.

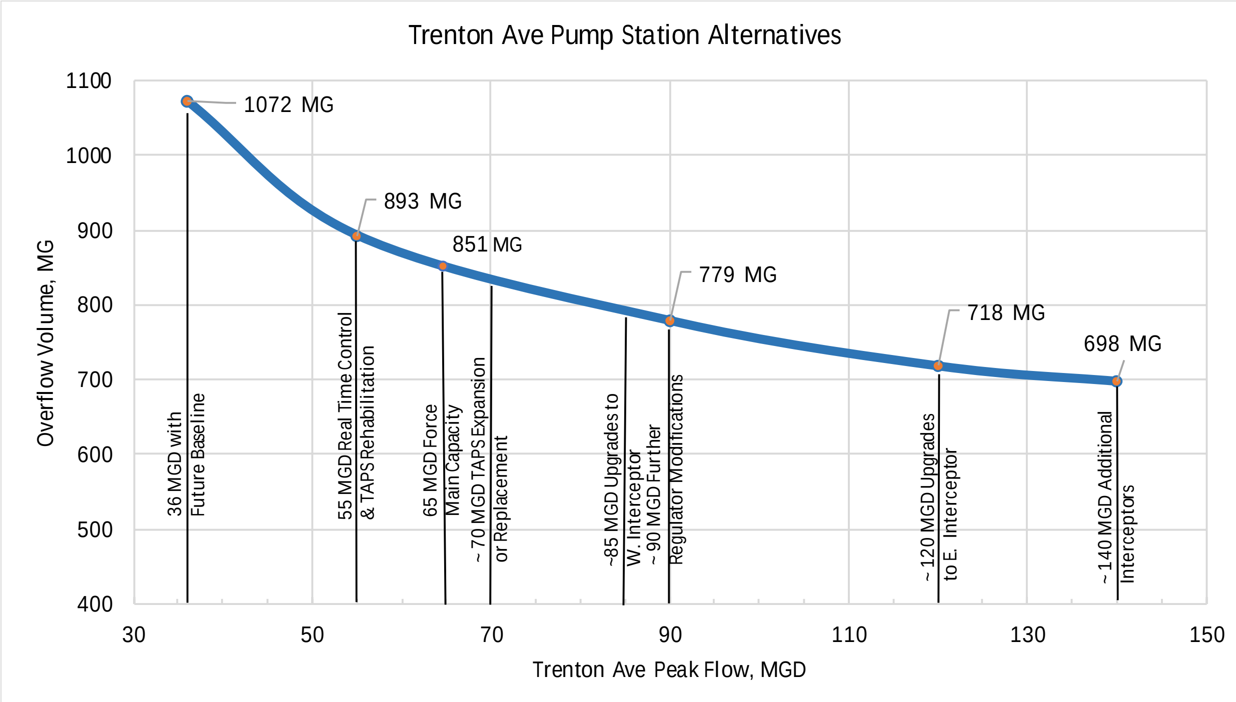

Figure 7-3: Impact of TAPS Pumping Rate on Future Baseline CSO Volume

7.3.1 Background

The JMEUC owns and operates the Edward P. Decher Secondary Wastewater Treatment Facility (WWTF) located in Elizabeth, New Jersey and the trunk sewers that capture flow from the sanitary sewer collection systems in its member communities. The NJPDES CSO Permit requires that the JMEUC meet its existing NJPDES plant effluent permit limits for all flows. These permit limits are listed on Table 7-13.

Table 7-13: NJPDES Permit Limits

Parameter	Effluent Limitation		
	Percent Removal	Concentration	Mass Loading[1]
5-day Carbonaceous Biochemical Oxygen Demand ($cBOD_5$), monthly average	85%	25 mg/L	7,100 kg/day
$cBOD_5$, weekly average	---	40 mg/L	11,355 kg/day
Total Suspended Solids (TSS), monthly average	85%	30 mg/L	8,519 kg/day
TSS, weekly average	---	45 mg/L	12,779 kg/day
Oil & Grease, monthly average	---	10 mg/L	---
Oil & Grease, weekly average	---	15 mg/L	---
pH	---	6.0 to 9.0 SU	---
Fecal Coliform, Monthly Geometric Average	---	200/100 ml	---
Fecal Coliform, Weekly Geometric Average	---	400/100 ml	---
Chlorine Produced Oxidants, Monthly Average	---	0.062 mg/L	17.6 kg/day
Chlorine Produced Oxidants, Daily Maximum	---	0.088 mg/L	25.0 kg/day
Dissolved Oxygen, Minimum Weekly Average	---	4.0 mg/L	---

Parameter	Effluent Limitation		
	Percent Removal	Concentration	Mass Loading[1]
Nickel, Total Recoverable, Daily Maximum	---	0.02 mg/L	5.8 kg/day
Silver, Total Recoverable, Daily Maximum	---	0.01 mg/L	2.8 kg/day
Zinc, Total Recoverable, Monthly average	---	0.39 mg/L	128 kg/day
Zinc, Total Recoverable, Daily Maximum	---	0.78 mg/L	236 kg/day
Lead, Total Recoverable, Monthly Average	---	0.13 mg/L	36.9 kg/day
Lead, Total Recoverable, Daily Maximum	---	0.24 mg/L	68.1 kg/day
Copper, Total Recoverable, Daily Maximum	---	45.8 ug/l	13 kg/day
Mercury, Total (as Hg), Monthly average	---	0.40 ug/l	114 g/day

Notes: 1 – Effluent mass loading values based on a flow rate of 75 mgd.

Existing System Configuration

The existing configuration of the JMEUC trunk sewers and WWTF that captures and treats combined sewage from the City of Elizabeth combined sewer system is illustrated below in Figure 7-4.

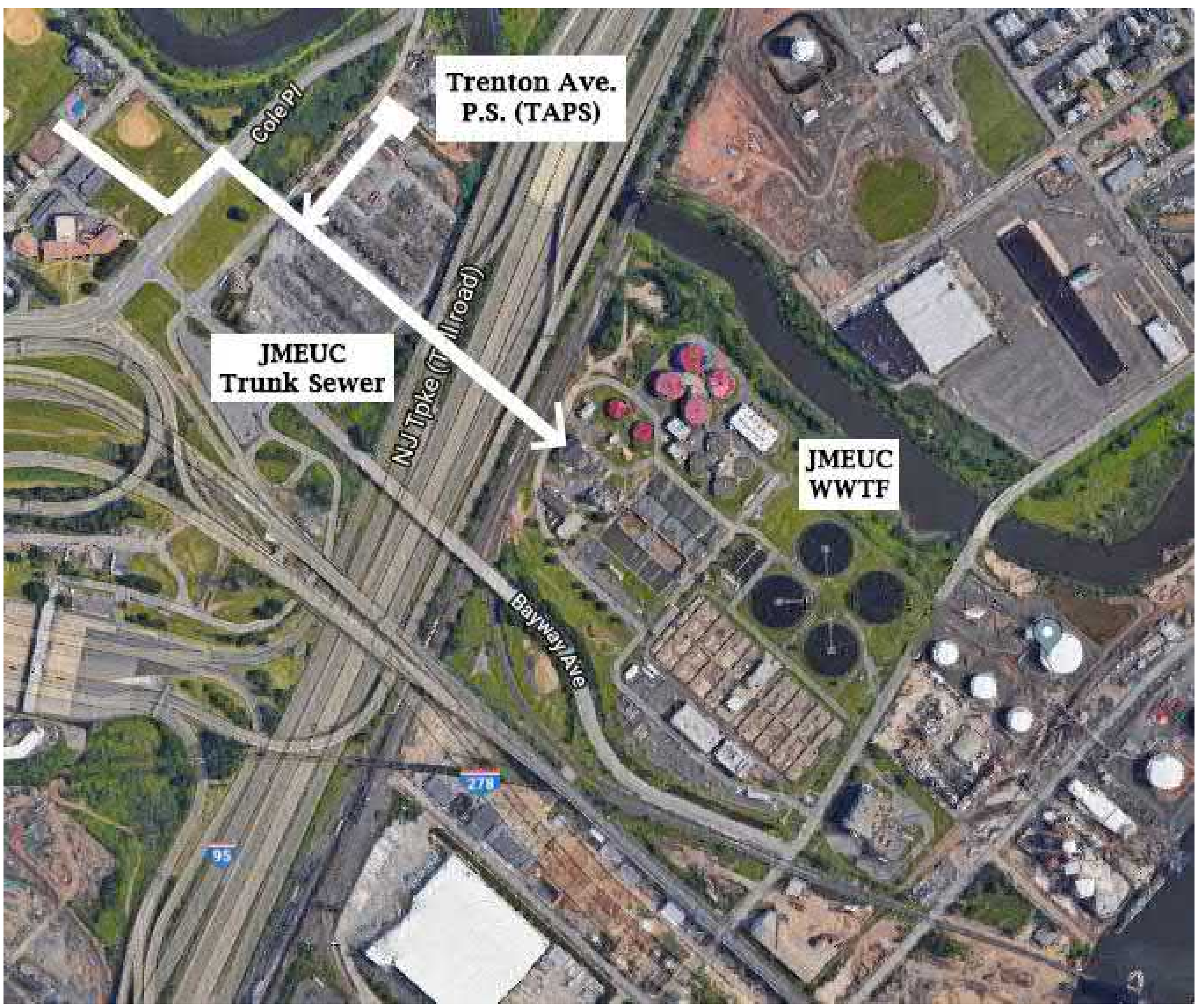

Figure 7-4: Existing Trunk Sewer Alignment (NTS; approximate location)

The trunk sewer collection system from the City of Elizabeth force main connection pumped from the TAPS to the JMEUC facility includes two twin 5'-8" x 5'-7" concrete box sewers. The combined non-surcharged capacity of the twin-barrel trunk sewers has been estimated as 141 mgd based on hydraulic evaluations prepared by Keyspan/Paulus, Sokolowski and Sartor, LLC in March 2002 (this value is consistent with more recent hydraulic modeling of these sewers presented in the System Characterization Report). The twin-barrel trunk sewers are located approximately 5-feet below grade and slope at a 0.067-percent grade toward the JMEUC WWTF Screenings Chamber. The force main from the TAPS discharges into the north barrel of the twin trunk sewers. Flows are equalized between the north and south barrels at a junction chamber located upstream on Bayway Avenue.

7.3.2 Wastewater Treatment Facility Description and Capacity Evaluation

The JMEUC WWTF is rated for an average flow of 85 mgd, but has been shown to receive and properly process significantly higher flow rates during wet weather events. The following subsections briefly describe each unit process at the facility.

Table 7-14 presents the treatment and hydraulic capacities of the existing liquid unit processes at the JMEUC WWTF. The treatment capacities are based on design guidelines for normal-strength (cBOD and TSS) municipal wastewater streams; various guidelines, including 10 State Standards, New Jersey administrative codes, and process-specific evaluations, are referenced. The hydraulic capacities are based on the peak flow that can pass through each unit without overflowing.

Preliminary Treatment
Preliminary treatment consists of coarse mechanical bar screens, fine mechanical bar screens, and long rectangular gravity grit tanks. The screens are installed in two 12-foot-wide parallel influent channels which split into four 9.5-foot-wide channels for grit removal.

Table 7-14: Design Capacities

	Treatment Capacity	Hydraulic Capacity
	mgd	mgd
Screening	220	220
Grit Removal	220	220
Primary Settling Tanks	180	180
Aeration Tanks	180	180
Final Settling Tanks	122	180
Chlorine Contact Tanks	73	180

The coarse screens have 3.5-inch clear openings and the fine screens have 0.75-inch clear openings. The hydraulic capacity of clean screens is 220 mgd with 0.2 feet of combined head loss. The grit channels are 9.5 feet wide by 7 feet deep by 57 feet long and have a hydraulic capacity of over 220 mgd. The channel geometry is unconventional, however based on fall velocities of coarse sand (medium grit) approximately 70 percent removal of medium grit can be expected at 220 mgd.

Primary Settling Tanks (PST)
There are four rectangular PSTs; each 200 feet long, by 75 feet wide by 13.8 feet deep. The 10 State Standards provides a peak hour hydraulic loading rate for primary clarifiers of 1,500 to 3,000 gallon per day per square foot (gpd/ft^2). A hydraulic loading rate of 3,000 gpd/ft^2 was used to determine the capacity of the PSTs. Based on this hydraulic loading rate, with all four units in service, the facility has primary treatment capacity of 180 mgd. However, operating experience has shown that the permit requirements

for effluent quality can be achieved with the PSTs operating at significantly higher flow rates. In fact, only two tanks are normally used, with the use of a third tank reserved for high storm flow conditions.

Aeration Capacity
There are four aeration tanks, each with a volume of approximately 3.97 million gallons (total volume of 15.89 million gallons). Each aeration tank has eight, 100-hp, and two-speed surface aerators capable of providing a maximum of 2,360 lb/hr of oxygen per tank. Based on NJAC regulations of 38 lbs of cBOD per 1,000 cubic feet of volume; the four aeration tanks can treat a maximum of 96,939 lb/day of cBOD. The facility re-rating report (Hazen and Sawyer, P.C., June 1990) indicates that the existing aeration tanks are capable of treating 180 mgd.

Final Settling Tanks (FST)
There are four FSTs, each with a diameter of 180 feet and depth of 15 feet. Based on the recommended peak hour hydraulic loading rate of secondary clarifiers from 10 State Standards of 1,200 gpd/ft^2, the FSTs can adequately treat 122 mgd. However, operating experience has shown that the permit requirements for effluent quality can be achieved at significantly higher flow rates.

Chlorine Contact Tanks
There are two CCTs, each with dimensions of 50 ft by 126 ft by 10 ft water depth. Based on the contact time at peak hour flow rate of 20 minutes as defined in N.J.A.C. 7:14A-23.23(c), the existing chlorine contact tank volume is sufficient to treat a peak hour flow of 73 mgd. However, operating experience has shown that the permit requirements for effluent pathogen concentrations can be achieved at significantly higher flow rates.

Hydraulic Capacity
In addition to treatment process limitations, the plant was evaluated for hydraulic limitations. Though the plant is hydraulically rated for 180 to 220 mgd based on the tide elevation at the outfall, some known hydraulic problems exist at high wet weather flows.

Final Settling Tanks (FST)
FST Nos. 2 and 2A are significantly farther from the post chlorination chamber than the other two final settling tanks. During high flow events, this can cause the secondary effluent to backup into FST Nos. 2 and 2A effluent troughs, flooding the weirs. This problem has been observed to occur at flows greater than 180 mgd, when tides exceed six feet above mean sea level (msl).

Outfall – High/Low Tide
The JMEUC facility discharges to the Arthur Kill, a tidal water body. The flow rate of the facility is limited at high tide due to the weir elevation of the chlorine contact tanks and FSTs. At tide elevations greater than eight feet above mean sea level, the facility can only discharge 120 mgd. A value of eight feet above sea level corresponds to a 13-year storm. At tides less than six feet above mean sea level (corresponding to a one-year recurrence interval), the hydraulic capacity of the facility is 180 mgd. The mean high tide for the Arthur Kill is approximately 2.5 feet above mean sea level.

Therefore, the hydraulic capacity of the facility decreases during large wet weather events due to the tidal elevation of the Arthur Kill. The JMEUC is in the process of designing a new Effluent Pump Station that will alleviate the hydraulic limitations due to tidal events. The Effluent Pump Station design capacity is 360 mgd, which is greater than the maximum predicted flows determined by the modeling work previously discussed.

Current Solids Handling
The facility has 1.2 mgd of primary sludge pumping capacity, 24 mgd of waste sludge pumping capacity, and 33 mgd of return sludge pumping capacity. Primary and waste activated sludge (WAS) is thickened

using four gravity thickeners. From the gravity thickeners the sludge is pumped to two of three existing gravity belt thickeners. The thickened sludge is then fed to four primary anaerobic digesters. From the four primary digesters the sludge is transferred to two sludge storage tanks before being pumped to the dewatering facility. At the dewatering facility, digested sludge is dewatered by centrifuge, lime is added for stabilization, and Class A stabilized product is hauled offsite for land application.

Gravity Thickeners

There are four gravity thickeners, each with a diameter of 65 feet and side water depth of 10 feet. Based on the recommended (M&E, Wastewater Engineering, 2003) maximum hydraulic loading rate of gravity thickeners processing combined waste and primary sludge of 300 gpd/ft^2 and a maximum solids loading rate of 16 lb/ft^2/d, the gravity thickeners can adequately treat 3.98 mgd and 212,400 lb/d of sludge. Based on the 2013 through 2018 facility data, on average, 141,600 lb/d of primary sludge and WAS is sent to the gravity thickeners.

Gravity Belt Thickeners

There are three two-meter gravity belt thickeners used for thickening WAS and primary sludge prior to digestion. Based on the recommended (M&E, Wastewater Engineering, 2003) maximum hydraulic loading rate of gravity thickeners of 200 gpm/meter, each GBT can process 0.576 mgd of sludge and if all three GBTs are operating, 1.73 mgd of sludge can be processed. Based on a conservative maximum solids loading rate of 900 lb/hr/meter the GBTs can process 129,600 lb/d of sludge.

Primary Digesters

There are four primary digesters, each with a diameter of 95 ft and side water depth of 33.5 ft. Based on a solids retention time of 17 days and a loading rate of 200 lb volatile suspended solids (VSS)/1000 ft^3/d, the primary digesters can adequately treat a maximum month sludge flow of 0.42 mgd of sludge and 190,000 lb/d of VSS.

Electric Service Capacity

Additional treatment capacity generally requires additional power. The following information is summarized from the Long-Term Control Plan, Cost and Performance Analysis Report, NJPDES Individual Permit No. NJ0024741, CDM, March 2007. This information is provided as general background on the facility. If expanded treatment of CSO flows at the JMEUC WWTF is carried forward in the LTCP, updated electrical service and loads should be evaluated during preliminary design of any facility upgrades.

- The current peak demand load for the JMEUC facility: approximately 3,900 kilowatts (KW).
- The assumed power factor is 88 percent and the peak kilovolt-ampere (KVA) demand of 4,400 KVA.
- These 5KV transformers that serve the main switchgear have a maximum rating of 5,250 KVA using forced air cooling.
- Under normal operating conditions the facility is powered by both utility services and both main transformers.
- There is a power take-off at the 26 KV bus for the Dewatering and Drying Facilities. These take-offs do not burden the main transformers. Switching provisions are provided that allow one utility service to power both main transformers.
- The only time the facility relies on a single main transformer is if a main transformer fails or during periods of maintenance.
- Power system diversity is assumed to be 65 percent and it was assumed the main transformers carry approximately 2,800 KVA. For a safety margin, a 25 percent factor was applied such that the peak load on a single transformer can be as high as 3,500 KVA.
- It is not recommended that the facility be operated at more than 90 percent of the forced air-cooled rating and the total connected load should be limited to 4,700 KVA.

- It is assumed that there is 1,200 KVA of spare capacity within a transformer.

Land Availability
Limited space is available for expansion at the JMEUC facility site. Though some unused land exists on the JMEUC property southwest of the primary clarifiers, northwest of the sludge storage tanks and northwest of the final settling takes, much of the remaining property is either in the flood plain, classified as Wetlands or earmarked for other construction projects (the Effluent Pump Station). The new floodwall currently being designed under a FEMA contract effectively eliminates the open space between plant facilities and the Elizabeth River. Therefore, it may be necessary to purchase more land near the existing facility in order to implement some of the treatment options.

7.3.3 Existing Plant Data Summary

Table 7-15 presents the existing average influent and effluent data for the JMEUC facility based on facility data from 2013 through 2018.

Table 7-15: Existing Average Influent and Effluent Data

Parameter	Average Influent	Average Effluent
Flow, mgd	54.2	54.2
Temperature, degrees Celsius	18.7	NA
Total Suspended Solids (TSS), mg/L	178.4	92.6
Volatile Suspended Solids (VSS), mg/L	85.8	NA
Biochemical Oxygen Demand (BOD), mg/L	208.3	21.7
Carbonaceous Biochemical Oxygen Demand (cBOD), mg/L	167.8	11.6
Chemical Oxygen Demand (COD), mg/L	18.9	19.8
Ammonia (NH_3), mg/L	434.6	65.2
Nitrate (NO_3), mg/L	1.3	1.7
Total Phosphorus (TP), mg/L	4.0	2.2

Wet Weather Duration, Frequency, and Existing Wet Weather Plant Data Summary
The JMEUC NJPDES permit limits are based on monthly and weekly averages. To determine whether the treatment process modifications would allow JMEUC to meet its permit limits, several assumptions were made. It was assumed that a large storm event resulting in a flow rates as high as the new maximum flow from the TAPS would occur once per month. Furthermore, it was assumed that these large wet weather events would occur over a period of three consecutive days.

Therefore, for determining the monthly and weekly averages needed to comply with the facility's NJPDES permit, it was assumed that the large wet weather flow would occur on three consecutive days. For the calculation, the other days of the month were assumed to have the average facility flow and effluent concentrations.

Using the 2013 through 2018 flow data, the non-dry weather days and dry weather days were selected according to the methodology presented Table 7-16.

Table 7-16: Definition of Dry and Non-Dry Days

Day	Definition
Dry Day	Day on which 0.00 to 0.09 inch of precipitation occurs
Non-Dry Day(s)	Day on which 0.10 to 0.29 inch of precipitation occurs, or

	Day on which 0.30 to 0.99 inch of precipitation occurs, and the next day, or Day on which 1.00 to 1.99 inches of precipitation occurs, and the next two days, or Day on which 2.00 or more inches of precipitation occurs, and the next three days.

The maximum flow from the Trenton Avenue Pump Station (TAPS) is currently limited to 36 mgd, which is currently discharged into the existing north barrel of the twin-barrel trunk sewers that feed the JMEUC WWTF. It is JMEUC's understanding that through operational changes, the current TAPS maximum flow rate can be increased to 55 mgd, the hydraulic capacity of the facility (with some facility upgrades to improve reliability at this higher peak flow rate).

Based on modeling of system performance and discussions with the City of Elizabeth, the maximum future flow rate that can be delivered to the JMEUC WWTF by the combined sewer system (with additional pumping and conveyance facilities) is approximately 140 mgd. If the TAPS is upgraded on an interim basis to deliver 55 mgd, the additional future maximum CSO flow that needs to be handled is 85 mgd. These flow rates have been used in the evaluations presented in this report, recognizing that they are subject to adjustment going forward if expanded treatment of CSO flows at the JMEUC WWTF is carried forward in the LTCP process.

Storm flow modeling for the entire JMEUC WWTF service area was completed by CDM Smith using InfoWorks® ICM modeling software (Innovyze®), as presented in the JMEUC System Characterization Report (Revised Report dated December 5, 2018). For the Typical Year the expected instantaneous peak flow is 251.3 mgd, which includes 140 mgd from the TAPS. It is standard practice for wastewater treatment facility processes to be designed to treat the peak hour flow and the modeled peak hour flow is 238 mgd, which also includes 140 mgd from the TAPS. Therefore, this analysis will consider the 251.3 mgd as the total flow that needs to be passed hydraulically through processes and 238 mgd as the total flow that needs to receive treatment.

Table 7-17 presents the existing average wet weather influent and effluent data for the JMEUC facility based on facility data from 2013 through 2018.

Table 7-17: Existing Average Wet Weather Influent and Effluent Data

Parameter	Wet Weather Average Influent	Wet Weather Average Effluent
Flow, mgd	116	116
Temperature, degrees Celsius	13.7	14.1
Total Suspended Solids (TSS), mg/L	121	24.4
Volatile Suspended Solids (VSS), mg/L	82.1	NA
Biochemical Oxygen Demand (BOD), mg/L	109	27.9
Carbonaceous Biochemical Oxygen Demand (cBOD), mg/L	84.1	17.2
Chemical Oxygen Demand (COD), mg/L	268	70.5
Ammonia (NH_3), mg/L	8.5	10.5
Nitrate (NO_3), mg/L	5.2	2.6
Total Phosphorus (TP), mg/L	2.0	1.9

The JMEUC facility continues to meet their existing effluent permit limits, even with the large wet weather events.

The JMEUC WWTF data was further analyzed and the total suspended solids (TSS) and carbonaceous biochemical oxygen demand (cBOD) results are summarized in Table 7-18 for the average and large wet weather influent, primary effluent and plant effluent conditions. Large Wet Weather flows are defined as 100 mgd and greater.

Table 7-18: TSS and cBOD Summary

	Flow, mgd	TSS, mg/L	TSS, lb/d	cBOD, mg/L	cBOD, lb/d
Average JMEUC WWTF Influent	54.2	178.4	80,685	167.8	73,125
Large Wet Weather JMEUC WWTF Influent	115.8	120.7	115,503	84.1	79,545
Average JMEUC WWTF Primary Effluent	54.2	92.6	42,115	122.1	53,660
Large Wet Weather JMEUC WWTF Primary Effluent	115.8	95.0	92,527	76.1	72,845
Average JMEUC WWTF Effluent	54.2	13.0	6,078	11.6	5,350
Large Wet Weather JMEUC WWTF Effluent	115.8	24.4	24,363	17.2	16,817

7.3.4 Control Program 3A: Interim Plan for Increased CSO Treatment with Real Time Control

The Nine Minimum Controls require that wet weather flow to the wastewater treatment facility (WWTF) be maximized. It is understood that this requirement refers to maximizing flow within the existing legal agreements, in coordination with plant capacity and without adversely affecting other users of the wastewater collection system. In accordance with this understanding, the City of Elizabeth has been complying with this requirement, by operating the Trenton Avenue Pumping Station (TAPS) per the City's agreement with the JMEUC. This agreement sets a maximum flow rate at TAPS, which helps to ensure that flow from TAPS does not cause or contribute to flow rates that could exceed the capacity of the JMEUC trunk sewers or WWTF.

The existing contract between the City and JMEUC limits the peak flow rate (maximum of 36 mgd) and the total daily volume (18 MG). A stress test conducted in coordination with JMEUC determined that the existing pumps at TAPS could achieve a peak discharge up to 55 mgd with all pumps running. By revising certain institutional, operational and mechanical conditions, the potential exists to increase pumping to convey additional wet weather flows for treatment within existing conveyance capacities. Furthermore, the existing JMEUC facilities can accept and treat a 55 mgd discharge from the Trenton Avenue Pump Station without an expansion of the JMEUC trunk sewer or WWTF, qualified by being limited to the time interval between the peak wet weather inflows at TAPS and at the JMEUC WWTF headworks.

The JMEUC System Characterization Report (revised report dated December 5, 2018; see Section 7) described the potential to increase pumping to the WWTF by operating the TAPS pumps at the physical (hydraulic) limit of the pumping capacity (55 mgd), rather than at the contractual limit (36 mgd). It was noted that significantly greater capture of combined sewer flow from Elizabeth could be achieved during wet weather by operating TAPS at this higher rate (see also Figure 7-3 above).

The analysis presented in this report assumes that TAPS pumping would be increased to a maximum rate of 55 mgd, however, this rate may be modified during subsequent refinement of the plan. It has been noted that the TAPS facility may require refurbishment to operate reliably at the 55 mgd pumping rate. It has also been determined that the capacity of the force main discharging TAPS flow to the JMEUC North Barrel Trunk Sewer may be greater than 55 mgd, perhaps on the order of 65 mgd. It may therefore be prudent to consider during implementation of the interim plan the potential for the TAPS refurbishment to enable pumping slightly above the current hydraulic capacity of 55 mgd (up to 65 mgd). Factors to consider would include the hydraulic capacity of the JMEUC Trunk Sewers and the process capacities at the WWTF to accommodate flows above 55 mgd.

7.3.4.1 Increased Pumping at Trenton Avenue Pump Station

As part of Section 7.3.2 of the JMEUC System Characterization Report, the JMEUC analyzed the impact of increasing the TAPS pumping rate from the contractual peak flow limit of 36 mgd to the estimated total existing capacity of 55 mgd. It was determined that the additional 19 mgd in peak flow from the TAPS would increase the hydraulic grade line (HGL) in both the North and South Barrels of the JMEUC trunks. As seen in Figure 7-5 and Figure 7-6, the maximum increase in the HGL under this scenario is estimated to be roughly 1 ft near the TAPS force main connection, with impacts extending roughly one mile upstream. Downstream at the WWTF headworks the increase in the HGL is less than 0.5 ft., and additional flow from the TAPS is not predicted to impact the HGL through the primary treatment train, as the primary settling tank effluent weirs have ample length to pass additional TAPS flow.

In the review comments NJDEP provided for the System Characterization Report (letter from NJDEP dated November 8, 2018), NJDEP expressed concern regarding the potential impact that delivering additional flow to the WWTF could have on the disinfection system. To address this concern, the JMEUC evaluated the use of real time control (RTC) of TAPS pumping to maintain peak flow rates no higher than existing for the largest events in the Typical Year. The RTC would take advantage of the peak timing difference in wet weather flows from the separate sewer systems in JMEUC's upstream municipalities and the flows from Elizabeth's combined system, which peak much more quickly.

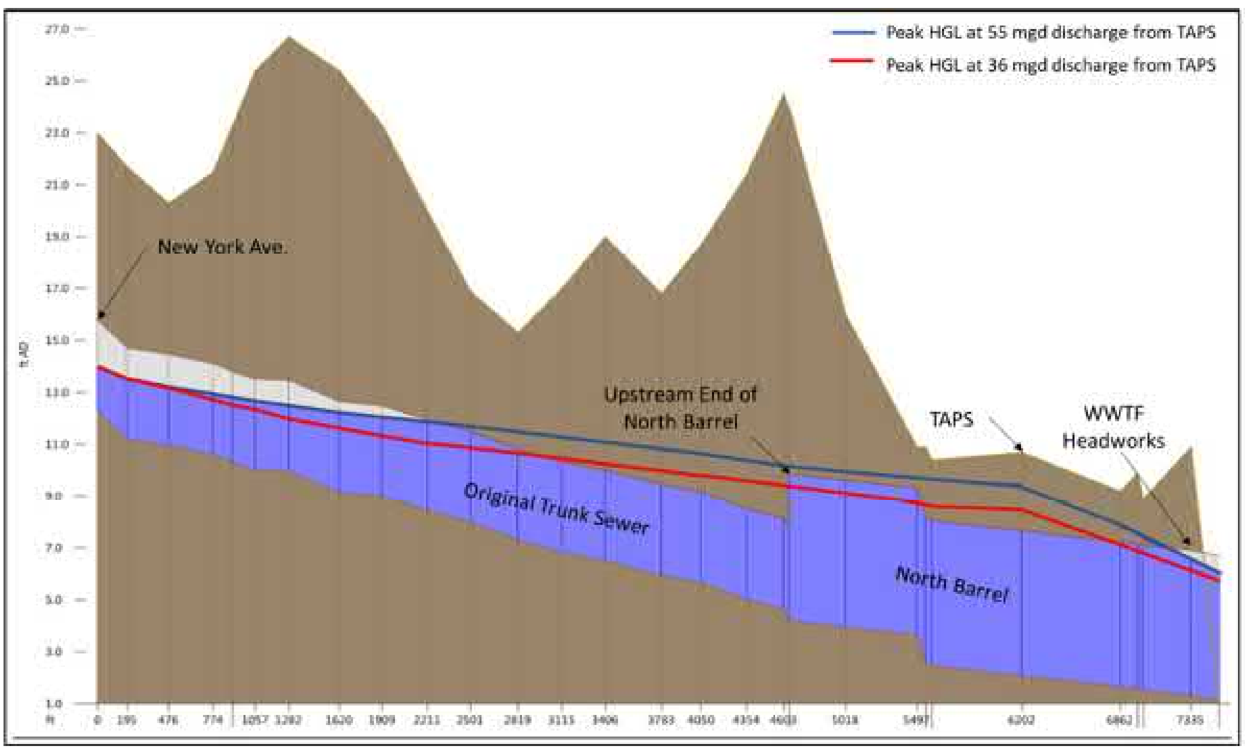

Figure 7-5: Comparison of Peak HGL's through JMEUC's North Barrel when increasing TAPS capacity to 55 mgd

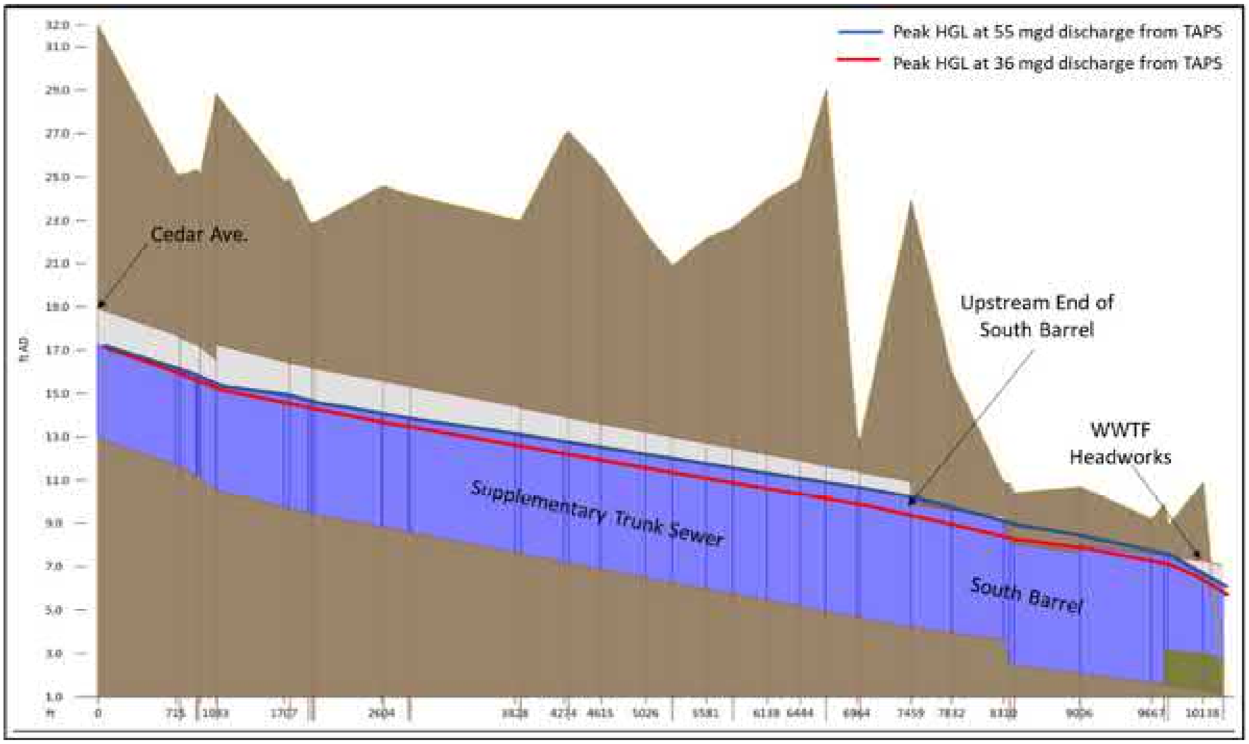

Figure 7-6: Comparison of Peak HGL's through JMEUC's South Barrel when increasing TAPS capacity to 55 mgd

7.3.4.2 Peak Timing Difference

Review of output from the calibrated Baseline Merged Model using Typical Year rainfall indicates a peak timing difference of roughly 2-3 hours between peak inflow to the TAPS wet well (from Elizabeth), and peak inflow to the WWTF from JMEUC's upstream municipalities. Figure 7-7 shows an example of this timing difference for the 9/18/2004 rainfall event.

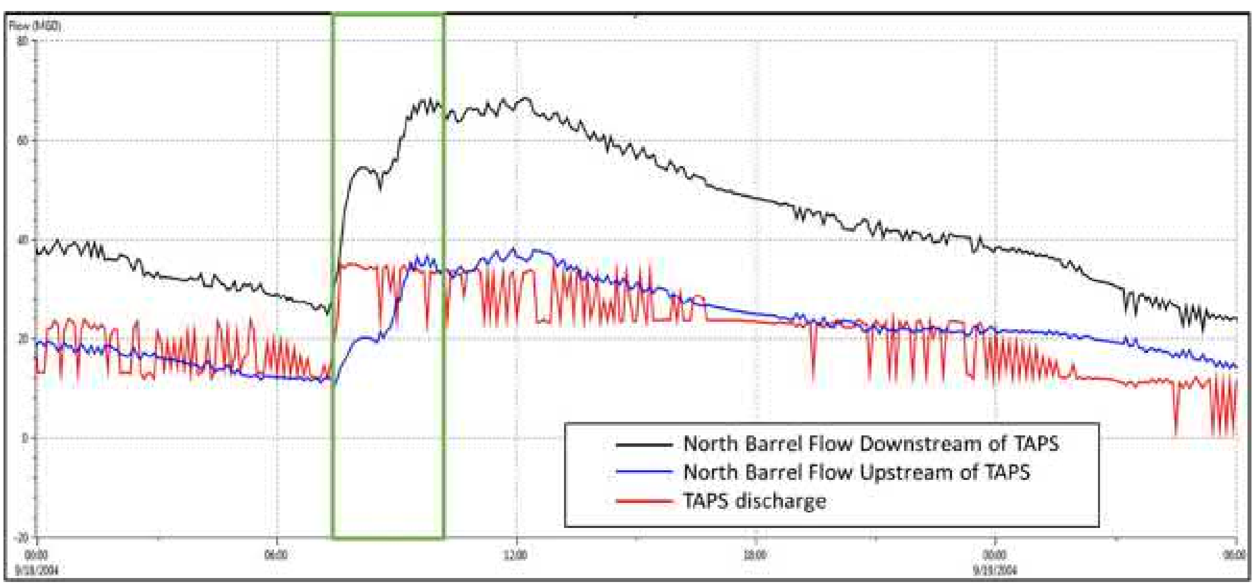

Figure 7-7: Peak Timing Difference in Flows through TAPS and From JMEUC's Upstream Municipalities for 9/18/2004 Event

Because peak WWF from Elizabeth's combined sewer system reaches the TAPS wet well more quickly than flows from JMEUC's upstream municipalities, the TAPS could discharge flow to the WWTF at rates higher than the contractual peak flow limit of 36 mgd as storm flows arrive at TAPS and ramp back down as the separate sewer flows increase to maintain peak rates no higher than existing. To determine the impact that this operating strategy at the TAPS would have on Elizabeth's Typical Year CSO volume, a theoretical control rule to simulate RTC was added to the calibrated Baseline Merged Model, described below.

7.3.4.3 Real Time Control Model Implementation

The existing TAPS has five pumps, only three of which currently operate for a 36 mgd pump rate. It has been estimated that by bringing the remaining two pumps on-line, the capacity of the TAPS could be increased to 55 mgd. Additionally, there are two sluice gates with fixed opening heights upstream of the TAPS wet well which currently limit flow into the wet well to 36 mgd. By bringing the two inactive pumps on-line and adding an RTC to regulate the two gate opening heights as a function of flow in JMEUC's North Barrel Trunk Sewer, pumped flow volume through the TAPS can be increased without increasing peak flows at the WWTF by taking advantage of the peak flow timing difference shown in Figure 7-7.

To model the RTC described above, the two off-line pumps in the calibrated Baseline Merged Model were brought on-line. The corresponding gate opening heights which allow 36 mgd and 55 mgd into the TAPS wet well were known from previous modeling work undertaken as part of the SCR. With the two gate heights known, a linear relationship between flow through the gates and the gate opening heights was developed. A linear relationship was used because InfoWorks ICM models sluice gates submerged on the upstream side using a standard orifice equation where flow varies linearly with opening height.

Next, a review of Typical Year flows in JMEUC's North Barrel Trunk Sewer directly upstream of the TAPS forcemain was completed. This review indicated peak DWF during the Typical Year through the North Barrel does not exceed 30 mgd. Additionally, there are five events during the Typical Year when flow through the North Barrel is predicted to exceed 50 mgd. Using this range of flows in the North Barrel (30 mgd to 50 mgd), a control rule was developed which linearly relates gate opening heights (controlling flow into the TAPS wet well and through the TAPS forcemain) to flow in JMEUC's North Barrel directly upstream of the TAPS forcemain. The control rule used to model this RTC strategy is described schematically in Figure 7-8.

Qualitatively, this control rule limits flow through the TAPS to 36 mgd when flow through JMEUC's North Barrel is high (i.e. greater than 50 mgd) and allows 55 mgd to flow through the TAPS when flow through JMEUC's North Barrel is low (i.e. less than 30 mgd). When flow through JMEUC's North Barrel is between 30 mgd and 50 mgd, peak flow through the TAPS is between 36 mgd and 55 mgd. This allows unused capacity at the WWTF during rainfall events to be used for additional combined flow from Elizabeth which reaches the WWTF more quickly than flow from JMEUC's upstream municipalities.

Typical Year simulation results indicate that RTC of the gate opening heights directly upstream of the TAPS wet well has the potential to increase flow through the TAPS by roughly 140 MG during the Typical Year. This additional flow through the TAPS represents the additional CSO capture volume from Elizabeth that would result with implementation of the RTC. Figure 7-9 shows the gate opening heights of the two modeled sluice gates during the Typical Year. Model results indicate that flow into the TAPS is only predicted to be limited to 36 mgd three times during the Typical Year, as highlighted in Figure 7-9. There are an additional 13 events during the Typical Year which cause the sluice gates to close to a limited extent, restricting flow into the TAPS wet well to less than 55 mgd, but still greater than 36 mgd.

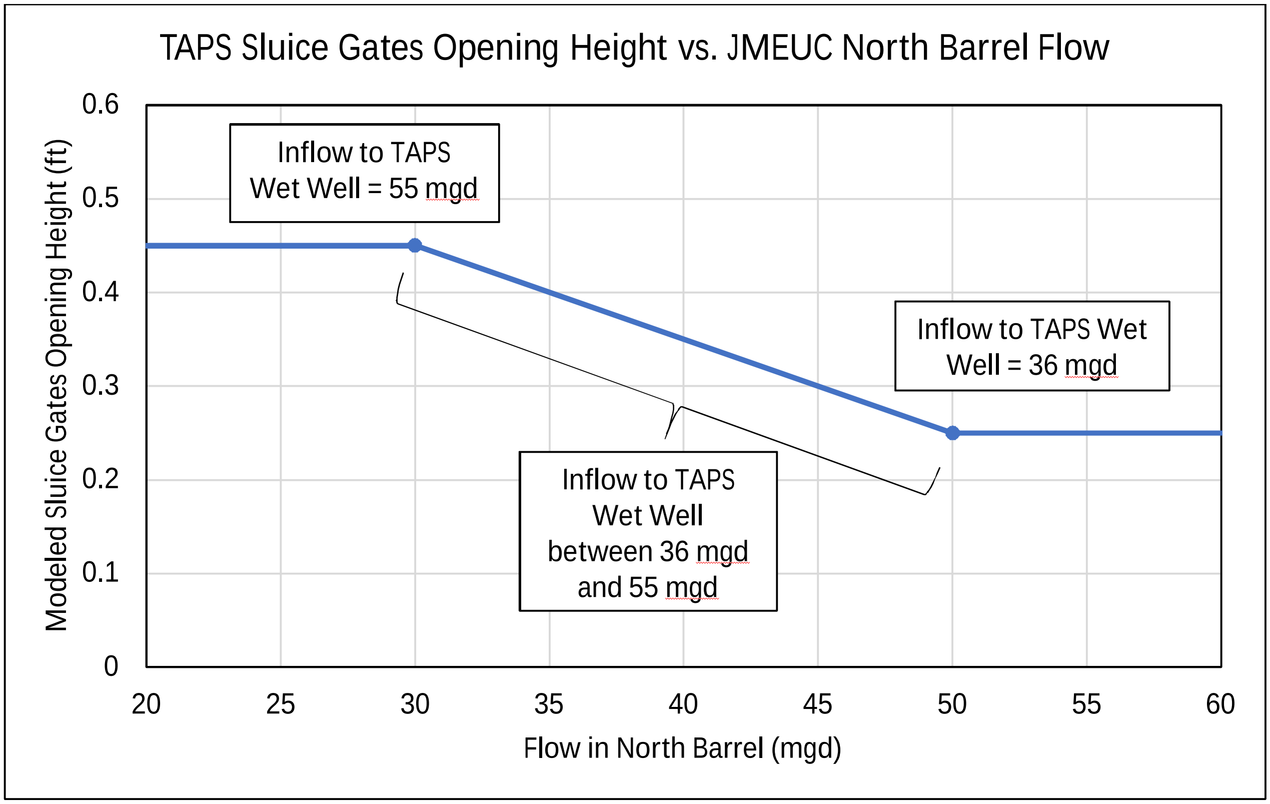

Figure 7-8: Modeled Control Rule used to Simulate TAPS RTC

Figure 7-10 shows simulated flow through both the TAPS forcemain (into the JMEUC system) and flow in the JMEUC's North Barrel Trunk Sewer, roughly 600 ft upstream of the TAPS forcemain connection into the North Barrel Trunk Sewer, for the 9/28/2004 event. This figure shows how the control rule operates in the model to allow TAPS to discharge up to 55 mgd to the JMEUC's system when flow in the North Barrel is less than 30 mgd, significantly reducing Elizabeth's CSO volumes during wet weather events due to the additional pumping capacity at the TAPS. When flow through the JMEUC's North Barrel exceeds 30 mgd, the TAPS sluice gates begin to close, limiting the TAPS discharge to between 36 mgd and 55 mgd, as can be seen in Figure 7-10. Throttling of the gates to restrict flow to the TAPS wet well effectively limits peak flows at the WWTF to peak flow rates currently experienced under existing conditions.

This throttling of the gates to restrict flow to the TAPS wet well effectively limits peak flows at the WWTF to peak flows currently experienced under existing conditions. Figure 7-11 shows time series plots of WWTF inflows under both existing and control rule conditions for the Typical Year. The wet weather events causing the sluice gates to close enough to limit the TAPS discharge to 36 mgd are boxed in red. This figure indicates that while the smaller wet weather events, which the existing WWTF treatment processes are all fully able to handle, are predicted to cause an increase in both peak wet weather inflows and volumes at the WWTF, larger wet weather peak inflows are essentially identical to those that would be experienced under existing conditions due to the modeled control rule.

Figure 7-12 shows the simulated flow through primary treatment at the WWTF for the 9/28/2004 event under both existing and control rule conditions. This event is one of three events in where the control rule limits the TAPS discharge to the existing contractual limit of 36 mgd. As can be seen in this figure, volumetric flow through the WWTF is increased (with this increase representing an increase in Elizabeth's CSO capture volume) while the existing peak flow rate is maintained.

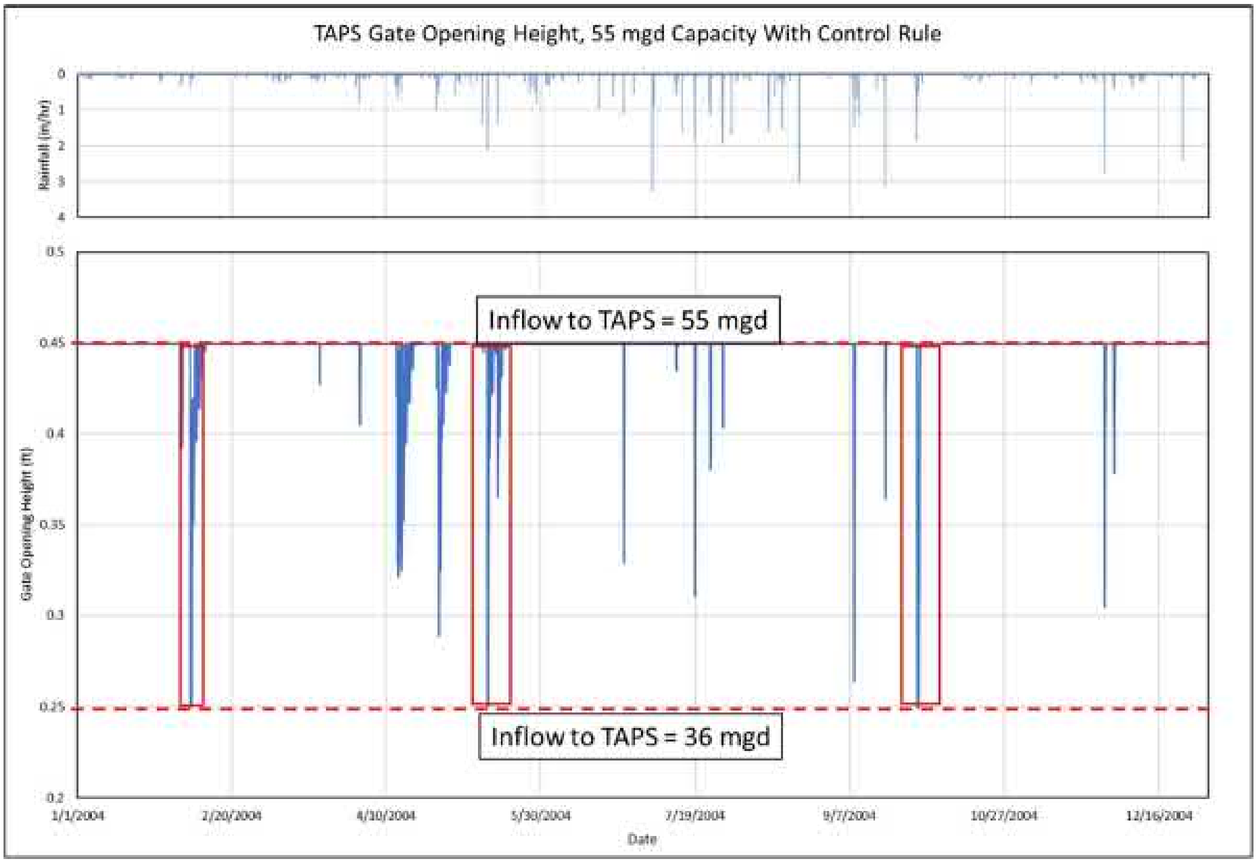

Figure 7-9: Simulated TAPS Sluice Gate Opening Heights during the Typical Year with Modeled Control Rule

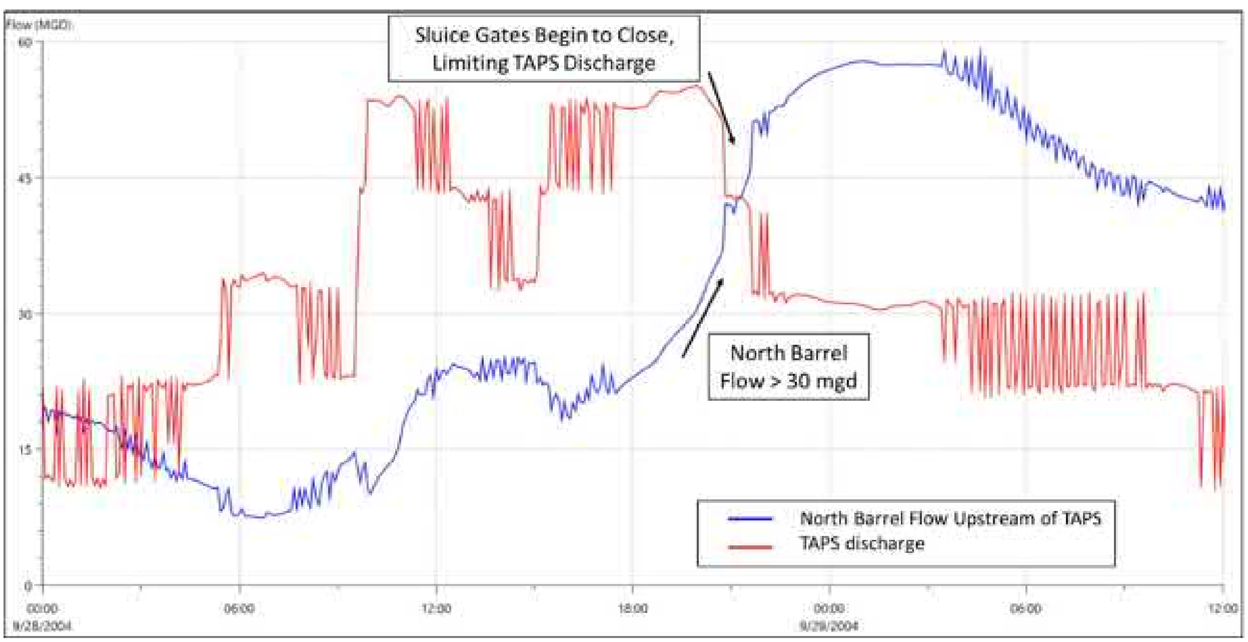

Figure 7-10: Flow through the TAPS Forcemain and in North Barrel with Modeled Control Rule

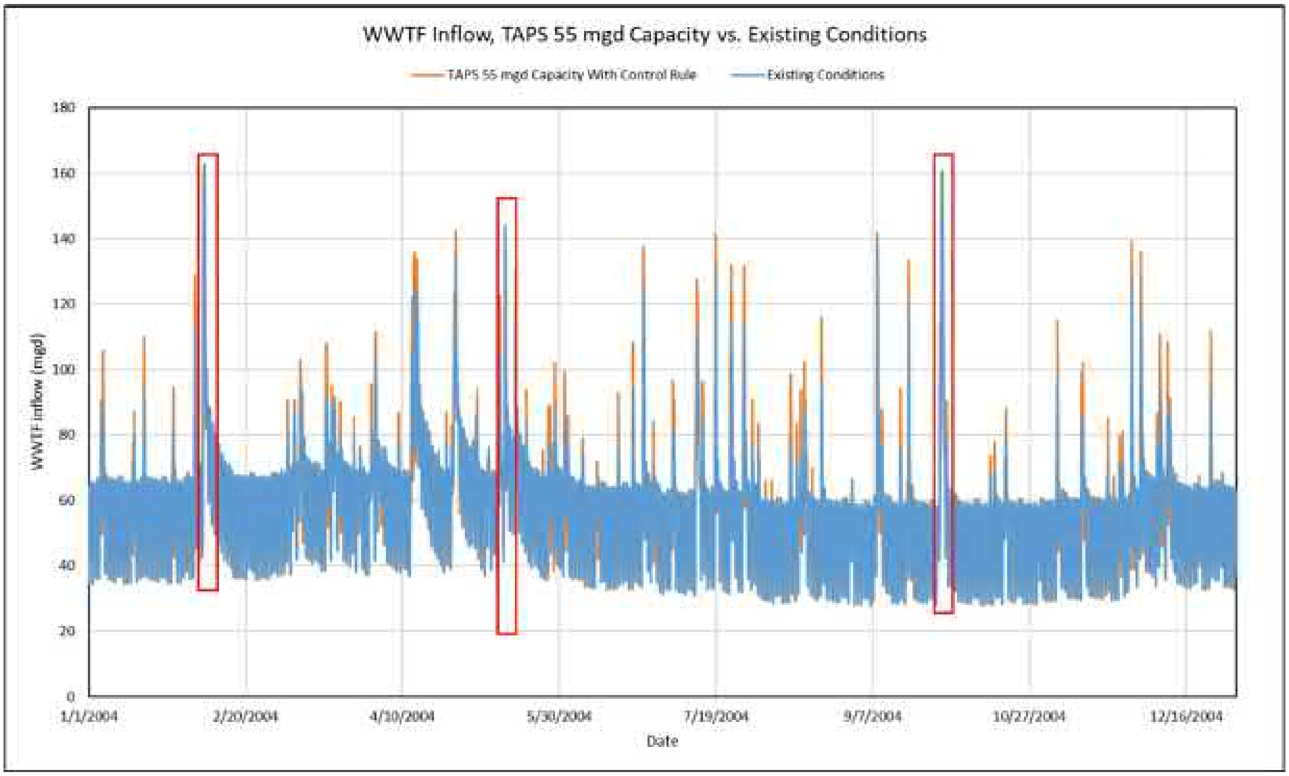

Figure 7-11: WWTF Inflows Under Existing and Control Rule Conditions

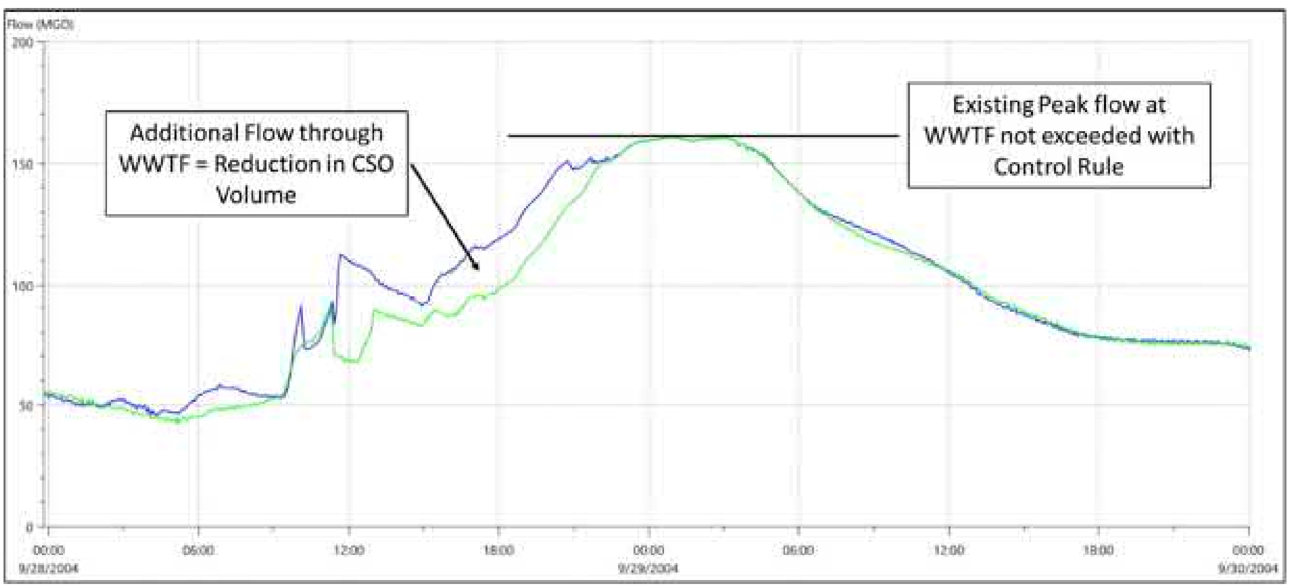

Figure 7-12: Flow through Primary Treatment at WWTF under Existing and RTC Conditions for 9/28/2004 Event

Typical Year volumetric flow through the TAPS with the control rule was also compared to volumetric flow through the TAPS with a constant maximum pumping capacity of 55 mgd (i.e. all pumps on-line and the sluice gates open to a constant height allowing up to 55 mgd into the TAPS wet well). When compared to the TAPS operating with a control rule, simulation results indicate that only 13 MG of additional flow during the Typical Year is passed through the TAPS when its capacity is fixed at 55 mgd, less than 10% more than with the control rule. This result indicates that the proposed RTC strategy would be very efficient at passing additional wet weather flows through the TAPS while maintaining existing peak wet weather flow rates at the WWTF.

7.3.4.4 Required Trenton Avenue Pump Station Upgrade

The Trenton Avenue Pump Station was constructed in 1955 and certain pieces of equipment are original. Given the stress placed on the equipment if operated at 55 mgd consistently during wet weather, a number of upgrades are required to reliably provide the desired performance. The following list summarizes the major components that would require upgrades:

- Mechanical bar screens – During dry weather TAPS receives debris consisting of rags, “flushable” wipes and other materials. During wet weather the debris load increases sharply as the first flush of litter, leaves, etc. is washed off the streets and into the combined sewer system. In response, during wet weather events, the TAPS influent gates are throttled to reduce the amount of debris reaching the screens. Throttling the gates holds the debris in the system to be released after the storm when the flow rate is lower, thus reducing the amount of debris entering the pumping station. To operate the pumping station at 55 mgd, the gates would need to remain open during wet weather, which would result in the debris reaching the screens at a rate higher than they can handle. Accordingly, the screens would need to be upgraded to prevent blinding of the screens and allow proper operation of the pumping station.
- Screenings handling system – Currently, the screenings are raked from the screen and passed through a grinder and discharged downstream of the screens. From time to time, the ground screenings reconstitute and cause pump clogging, which is addressed through regular maintenance. With the increased rate of the flow and upgraded screens, the amount of screenings will increase, creating the potential for more frequent pump clogging. To prevent this, the existing grinder would be replaced with a screenings washer-compactor system, which would discharge screenings to a dumpster. This would also reduce the solids and organic loads delivered to the WWTF.
- Pumps – The pump casings are original from construction in 1955. To improve operational reliability, the pumps including casings, impellers and motors, would be replaced. It is noted that even with the pump replacement, the firm capacity would not be considered 55 mgd, since all pumps must be operating to reach the desired flow rate. This is in contrast to the firm capacity which assumes the largest pump is out of operation.
- Structural repairs – Given the age and condition of TAPS, it is likely that to accommodate the required improvements, structural repairs and modifications will be required. This includes modification to allow installation of the new screens, repairs that may be needed to protect new equipment from exposure to harsh conditions within the pumping station and improvements to accommodate additional loads from new pumps and pumping rates.
- Real time controls – A carefully planned control program will be required to obtain the desired performance while providing protection to the WWTF, equipment, force main and upstream users of the interceptor. To accomplish this, several sensors are anticipated:
    - Level and flow sensor in the North Barrel of the JMEUC Trunk Sewer – The flow component of this sensor will be used to set the allowable flow rate from TAPS. The level sensor will serve as a cut-off switch should the level in the interceptor exceed a predetermined level, to protect upstream users.
    - Flow sensor on the TAPS force main – The City recently installed a new flow meter at the TAPS, which will be tied into the RTC system to control the number of pumps running and pump speed so that TAPS is providing the maximum flow as is allowed under the control rules.

- Flow signal from JMEUC WWTF – While under normal operations it is expected that the WWTF can accommodate the additional 19 mgd in flow from TAPS, under abnormal conditions the additional flow may be problematic. To protect the WWTF the conditions at critical unit processes will be monitored and this information will be used to determine if flows at TAPS should be reduced to ensure the plant is not overwhelmed. It is also envisioned that certain alarms at the plant will trigger a reduction in pumping rate at TAPS to the 36 mgd contract limit. Likely this will involve integrating data from JMEUC's SCADA system, but may also require additional level and flow sensors on unit processes.

7.3.5 Control Program 3B: Expanded Wet-Weather Treatment for Combined Sewer Flows and CSO-Related Bypass

Several CSO flow path scenarios (through/around the existing JMEUC WWTF) were evaluated, all of which include various degrees of bypassing of flow around the existing WWTF treatment train (including the secondary treatment portion; in accordance with an approved NJPDES permit authorizing bypass). All flows presented represent the flows that require treatment and can be accommodated by currently estimated existing process capacities or by future treatment processes. During design, hydraulic analyses will be performed to ensure all unit processes can pass the expected peak hydraulic flows.

There are three flow path scenarios that were evaluated, and are summarized as:

- Option A: All additional flow is handled by a new treatment train
- Option B: Minimize capacity of a new treatment train (maximize use of existing capacity)
- Option C: Maximize use of existing secondary capacity (minimize additional pumping).

Each scenario is described in detail below.

Option A: Allow up to 55 mgd (consistent with trunk capacity) from TAPS to enter the JMEUC WWTF via the North Barrel and be treated through all existing processes. Deliver the additional 85 mgd CSO flow to the WWTF via separate force main, bypassing all existing treatment units. Therefore, the predicted flow through the existing JMEUC WWTF would be 153 mgd (238 mgd peak hour minus 85 mgd from TAPS). The bypassed flow (85 mgd) would be treated for inorganic solids removal and disinfection, discussed later in this section, and blended with the WWTF effluent before discharging at the plant outfall.

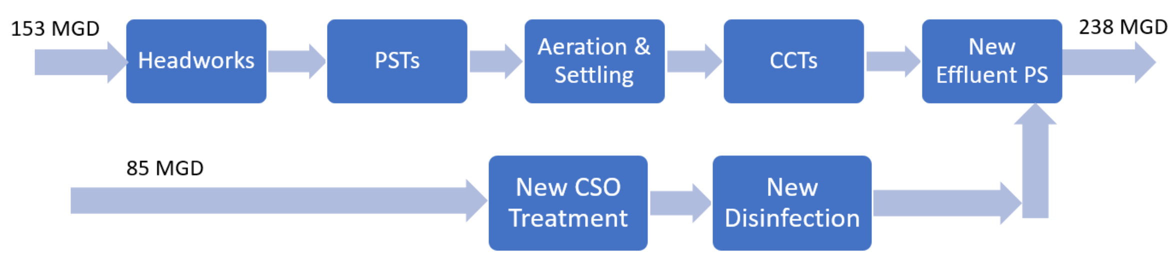

Figure 7-13: CSO Flow Path Option A (flows shown are peak hour treatment flows)

Option B: Allow up to 220 mgd to enter the headworks of the JMEUC WWTF, divert flows greater than 220 mgd around the existing JMEUC WWTF and blend with the WWTF effluent before discharging to the plant outfall. Allow up to 180 mgd to enter the Primary Settling Tanks (PSTs) and all processes downstream; divert flows greater than 180 mgd around the existing JMEUC WWTF secondary processes and blend with the WWTF effluent before discharging to the plant outfall. This option would likely require two intermediate pump stations: 1. Before the existing headworks to handle up to 18 mgd (31.3 mgd hydraulic peak) and 2. After the existing headworks to handle up to 40 mgd (71.3 mgd hydraulic peak).

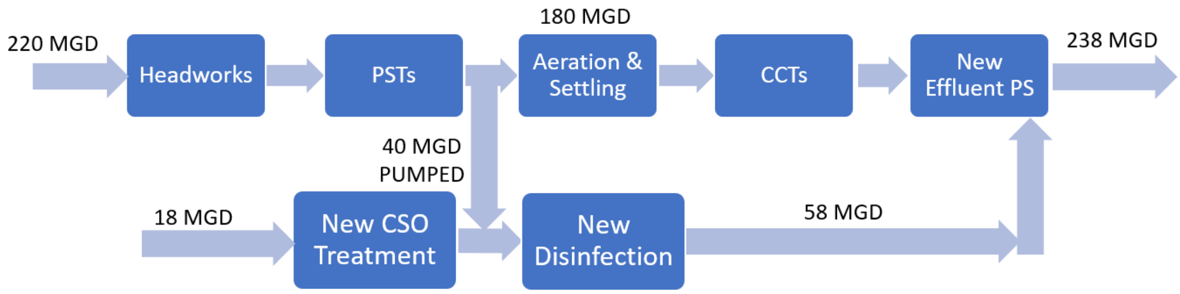

Figure 7-14: CSO Flow Path Option B (flows shown are peak hour treatment flows)

Option C: Allow up to 180 mgd to enter the headworks of the JMEUC WWTF, divert flows greater than 180 mgd around the existing JMEUC WWTF and blend with the WWTF effluent before discharging to the plant outfall. This option would require a new pump station before the existing headworks to handle up to 58 mgd (71.3 mgd hydraulic peak).

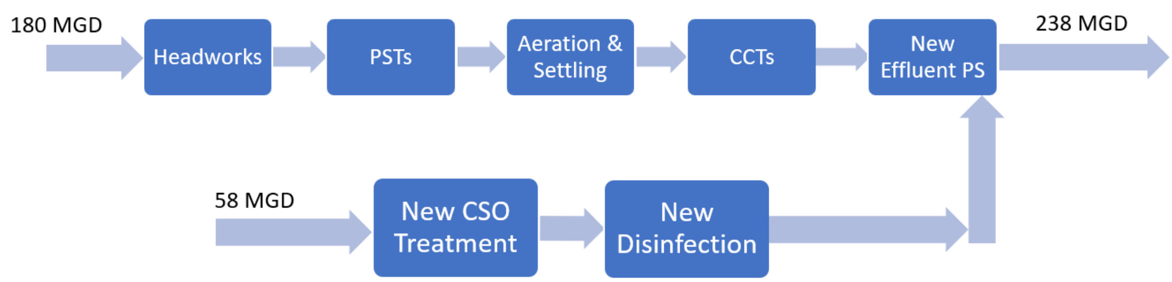

Figure 7-15: CSO Flow Path Option C (flows shown are peak hour treatment flows)

Options A, B and C require that additional disinfection capacity be provided at the WWTF. Figure 7-16 presents a summary of the three flow path options with the treatment flows identified for each unit process.

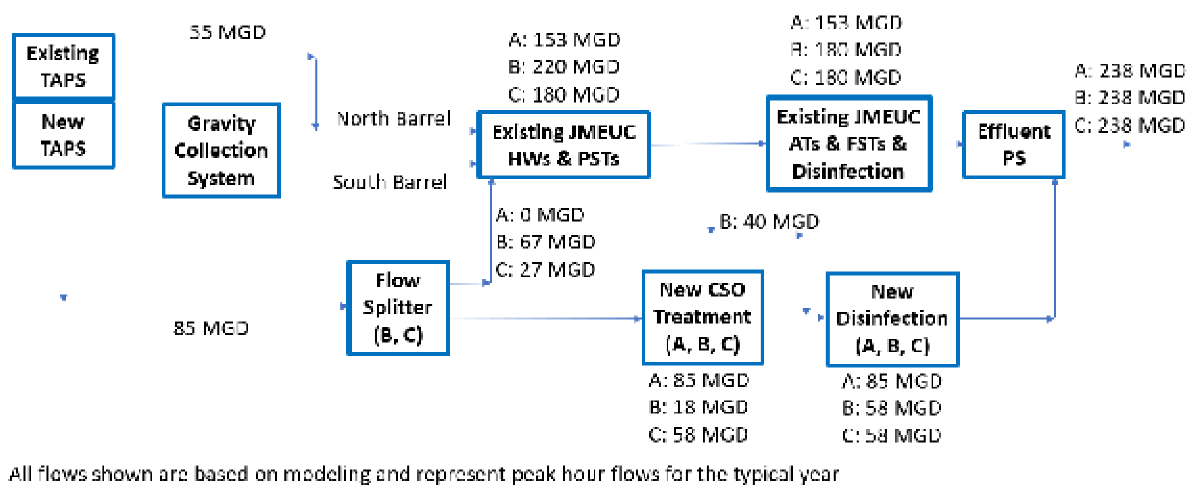

Figure 7-16: CSO Flow Path Options (flows shown are peak hour treatment flows)

JMEUC has selected Option A for further consideration in the LTCP because it simplifies the TAPS expansion and new force main design/construction, and reduces stress on existing JMEUC WWTF unit processes.

7.3.5.1 CSO Treatment Options

The PVSC CSO LTCP Technical Guidance Manual (TGM) (Greeley & Hansen and CDM Smith, January 2018) presented many options for CSO treatment. This section does not intend to repeat the work completed for that manual and instead focuses on the treatment options that are most applicable to the specific characteristics and logistics of the CSO flows that may come to the JMEUC WWTF.

Band screens, belt screens and drum screens were not considered because they received low ratings in the PVSC CSO LTCP Technical Guidance Manual and the site constraints at the JMEUC WWTF further validate those low ratings. While modified vortex and polishing filter options in the PVSC CSO LTCP Technical Guidance Manual received favorable ratings, their land requirements are relatively large compared to standard vortex separators and, as will be presented later in this section, the increased removal efficiencies for these systems are not needed to meet the effluent permit limits.

Therefore, only three treatment options were evaluated for the JMEUC WWTF CSO flows: fine screens following mechanical bar screens, standard vortex/swirl units, and ballasted flocculation.

Table 7-19 presents an overview of the treatment options considering the needs and land available at the JMEUC WWTF.

Table 7-19: CSO Flow Treatment Summary

Treatment Option		Benefits	Limitations	TSS Removal, %	cBOD Removal, %
1	Fine Screens following Mechanical Bar Screens	Small footprint (approx. 30 ft x 18 ft)	Need regulators (weirs) and screenings container	15	0
2	Vortex/Swirl Units	Easy to operate, TSS removal	Larger footprint (approx. 42 ft x 51 ft), Need ancillary tank to hold screenings (and odor control)	35	15
3	Ballasted Flocculation	Good TSS and BOD removal	Larger footprint than others (approx. 78 ft x 64 ft), Need ancillary tank, Start-up time	80	50

7.3.5.2 Analysis of the Impact of CSO-Related Bypass of the Secondary Treatment Process on Blended Effluent Quality

Blending is defined for the purposes of this evaluation as taking combined sewer influent flow from the TAPS during wet weather events to the JMEUC WWTF, providing parallel treatment, and reintroducing the treated flow to the normal effluent stream prior to discharge via the existing outfall conduit. Because this flow path would bypass the normal secondary treatment processes, the proposed CSO treatment train would be implemented in accordance with an approved NJPDES permit authorizing bypass.

The data and past information used for this evaluation included JMEUC facility data from 2013 through 2018; NJPDES Permit Limits for the JMEUC WWTF, specifically weekly average TSS and BOD concentrations of 45 mg/L and 40 mg/L, respectively; Facility Re-Rating Report, Hazen & Sawyer, June

1990; and the Long Term Control Plan, Cost and Performance Analysis Report, NJPDES Individual Permit No. NJ0024741, CDM, March 2007.

The following summarizes the basis criteria for this evaluation:

- Peak flow with the addition of the CSO flows that requires treatment is 238 mgd.
- No cBOD or TSS reduction achieved via preliminary treatment.
- Influent cBOD and TSS concentrations from separate sanitary sewer collection systems and combined sewer TAPS flow may have different concentrations which can be further evaluated as the project progresses.
- Wet weather days defined as per Table 7-16.
- Large Wet Weather flows defined as 100 mgd and greater.
- Storm event defined as lasting 3 days.
- Hydraulic capacity of existing JMEUC WWTF components are presented in Table 7-14.

For the permit evaluation it was assumed that large wet weather events will correspond with typical wet weather events. CDM Smith calculated the large wet weather event TSS and cBOD loads for the 3 storm days and calculated the typical wet weather event TSS and cBOD loads for the other 4 days in a week.

The typical average wet weather TSS and cBOD effluent concentrations from 2013-2018 data are 13.9 and 22.4 mg/L, respectively.

The effluent concentrations were calculated by taking a weighted average using expected effluent quality as follows: 3 days at the "large wet weather event expected effluent quality" and 4 days at the "average wet weather effluent quality". Values are from the 2013-2018 facility data set.

The blended flow results were compared to the permitted maximum average week concentrations for TSS and cBOD (45 and 40 mg/L, respectively).

Scenarios Evaluated

The flow path scenarios evaluated are the same as those presented in the previous section that evaluated the CSO flow paths (i.e. A, B and C). In addition, the three different CSO treatment technologies described on Table 7-19 (i.e. 1, 2 and 3) are considered, which creates a matrix of nine total scenarios for evaluation. To summarize, those scenarios are:

A1 – A3: Up to 153 mgd through HWs/PSTs/Aeration/FSTs; bypassed flows (up to 85 mgd) are treated with a CSO-treatment process and disinfected.

B1 – B3: Up to 220 mgd through HWs & PSTs, up to 180 mgd through Aeration/FSTs; bypassed flows (18 mgd before HWs plus 40 mgd before aeration) are treated through a CSO-treatment process and disinfected.

C1 – C3: Up to 180 mgd through HWs/PSTs/Aeration/FSTs; bypassed flows (up to 58 mgd) are treated through a CSO-treatment process and disinfected.

The analysis in this sub-section considered all nine options and calculated the expected effluent TSS and cBOD concentrations for each.

Predicted Blended Effluent Concentrations

For all blending scenarios, various CSO-treatment alternatives were evaluated and all blending scenarios resulted in the facility being expected to meet the weekly average TSS and cBOD permit limits (45 and 40 mg/L, respectively). Results of the analysis are presented in Table 7-20.

Table 7-20: Blending Analysis Summary

<table>
<tr><th colspan="2">Blending Scenario</th><th colspan="2">A</th><th colspan="2">B</th><th colspan="2">C</th></tr>
<tr><td colspan="2">Flow Thru Existing HWs, mgd</td><td colspan="2">153</td><td colspan="2">220</td><td colspan="2">180</td></tr>
<tr><td colspan="2">Flow Thru Existing PSTs, mgd</td><td colspan="2">153</td><td colspan="2">220</td><td colspan="2">180</td></tr>
<tr><td colspan="2">Flow Thru Existing Aeration & FSTs, mgd</td><td colspan="2">153</td><td colspan="2">180</td><td colspan="2">180</td></tr>
<tr><td colspan="2">Flow to New CSO Treatment, mgd</td><td colspan="2">85</td><td colspan="2">18 + 40</td><td colspan="2">58</td></tr>
<tr><td colspan="2">Flow to New Disinfection, mgd</td><td colspan="2">85</td><td colspan="2">58</td><td colspan="2">58</td></tr>
<tr><td colspan="2" rowspan="2">Option</td><td colspan="2">Blended Effluent [1]</td><td colspan="2">Blended Effluent [1]</td><td colspan="2">Blended Effluent [1]</td></tr>
<tr><td>TSS, mg/L</td><td>cBOD, mg/L</td><td>TSS, mg/L</td><td>cBOD, mg/L</td><td>TSS, mg/L</td><td>cBOD, mg/L</td></tr>
<tr><td>1</td><td>Fine Screens following Mechanical Bar Screens</td><td>30.4</td><td>24.4</td><td>17.1</td><td>17.3</td><td>18.6</td><td>17.8</td></tr>
<tr><td>2</td><td>Vortex Units</td><td>26.7</td><td>22.4</td><td>14.9</td><td>16.0</td><td>16.1</td><td>16.5</td></tr>
<tr><td>3</td><td>Ballasted Flocculation</td><td>18.4</td><td>17.9</td><td>10.1</td><td>13.1</td><td>10.4</td><td>13.4</td></tr>
</table>

[1] For comparison, weekly average permit effluent limits is 45 mg/L for TSS and 40 mg/L for cBOD.

7.3.5.3 Disinfection Options

There are several chemical and physical disinfection technologies typically used in wastewater treatment:

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

The PVSC CSO LTCP Technical Guidance Manual (TGM) (Greeley & Hanson and CDM Smith, January 2018) does not suggest using Ozonation for the disinfection of CSO flows as it is costly to operate and maintain, produces off-gas that can be toxic, is a complex system, and is not typically utilized for disinfection at wastewater treatment plants where flow is more controlled and less variable. Their analysis of the three remaining technologies did not result in one technology being significantly more advantageous than the others.

Therefore, recognizing that the JMEUC WWTF already has a chlorination facility on site, JMEUC has selected chlorination (and dechlorination) as the disinfection technology to include for further consideration of CSO treatment and disinfection in the LTCP.

The PVSC CSO LTCP Technical Guidance Manual suggests the following design guidelines for the use of chlorination/dechlorination for CSO flows:

- High-rate disinfection: using high-intensity mixing to accomplish disinfection within a short contact time, generally 5 minutes.
- Chlorination Chemical: Sodium Hypochlorite
- Dechlorination Chemical: Sodium Bisulfite
- Use of chemical induction mixers (one for sodium hypochlorite and one for sodium bisulfite) and mixing zone to induce a “G” value of at least 1,000/sec.

The NJDEP requires a minimum of 20 minutes detention time be provided at the design peak hour flow (PHF). However common wastewater practice is to provide 15 minutes detention time at the design PHF. For example, Florida has more stringent requirements for high level disinfection for unrestricted reuse and the minimum detention time requirement is 15 minutes. Effectiveness of disinfection can be increased maintaining higher residual chlorine at lower detention times. The product of “chlorine residual times contact time (CT)” is used for this purpose. The CT approach allows the plants to maintain lower residual chlorine and higher detention time during dry weather flows. On the other hand, the plant can increase residual chlorine in the CCTs during peak wet weather flows. For example, while targeting a CT value of 45 mg-min/L would require 3 mg/L residual chlorine at 15 min detention time, the effectiveness of disinfection can be increased by targeting a CT value of 60 mg-min/L by increasing residual chlorine to 4 mg/L at 15 min detention time.

The JMEUC WWTF has very limited space for construction of new contact tank volume and it is important to maximize the effectiveness of the existing disinfection facilities. JMEUC believes that effective disinfection can be achieved at the JMEUC WWTF with 15 minutes minimum detention time by maintaining higher residual chlorine in the existing CCTs. The analysis herein assumes that this approach will be employed for disinfection of the flow in the existing treatment train.

For disinfection of flow in the potential new CSO treatment train, per the PVSC CSO LTCP TGM, experience has shown that the long contact time required for conventional wastewater treatment is not appropriate for the treatment of CSOs. Chemical disinfection of CSOs can be accomplished using high-rate disinfection, which is defined as employing high-intensity mixing to accomplish disinfection within a short contact time, generally five minutes. The evaluation of CSO treatment herein assumes that this approach will be employed for disinfection of the flow in the potential new CSO treatment train. Two alternatives for the provision of high-rate disinfection have been developed:

Alternative 1: New CSO Chlorine Contact Tank with High-Rate Mixing Section
The typical length to width ratio for chlorine contact tanks (CCTs) is 40:1 and the maximum recommended depth is 8 to 10 feet. At 85 mgd and 5 minutes of detention time, the required volume is 0.3 million gallons. With a depth of 10 feet, length-to-width ratio of 40:1, and three passes; the resulting overall estimated dimensions of the CSO CCT are 36 ft wide by 136 ft long. There is room on the south side of the WWTP site near Bayway Ave. to construct a new CCT pending further site analysis and constructability review. Additional equipment required includes: hypochlorite pump(s), hypochlorite storage tank(s), piping, ancillary equipment. This equipment could be installed in the structure that would house the CSO flow treatment equipment. The treatment structure would be located upstream of the CSO CCT.

Alternative 2: Chlorine Contact Conduit with High-Rate Mixing Section
An alternative chlorination option studied was to construct a conduit on the WWTP site to deliver the treated CSO flow to the new Effluent Pump Station along the new proposed floodwall of appropriate dimensions to provide 5 mins of high-rate chlorination time for 85 mgd. This route is approximately 1800 ft long and would require a conduit diameter of 6 feet. This option has been determined unfeasible due to the multiple obstacles along the potential route.

7.3.5.4 Conceptual-Level Costs for CSO Treatment and Disinfection Options

Costs are based on the information provided in the PVSC CSO LTCP Technical Guidance Manual (Greeley & Hansen and CDM Smith, January 2018), as well as estimates of equipment costs and installation effort, and do not included engineering or permit fees. All costs are presented in 2019 dollars and all capital cost estimates are based on the Engineering News Record (ENR) Index of 11,205 for January 2019.

All estimated costs include the following markups:

- installation (50% of equipment cost)
- general contractor conditions (10% of total direct cost)
- general contractor overhead and profit (10% of the total direct cost)
- planning-level design/cost estimation contingency (50% of the equipment cost).

Table 7-21 presents the planning-level cost estimate summary for the evaluated CSO treatment options plus disinfection along with estimated annual operations and maintenance costs. There will be additional cost to expand the TAPS, and to construct a new force main from TAPS to the WWTF. These costs are not included in Table 7-21 as they are applicable to all options.

Table 7-21: WWTF CSO Treatment Planning-Level Cost Estimate

	CSO Treatment Capital Cost	CSO Disinfection Capital Cost	Site Work and Yard Piping	Total Capital Cost	CSO Treatment & Disinfection Annual Cost
Mechanical Bar Screens Followed by Fine Screens	$2.6M	$9.5M	$4.2M	$16.3M	$490,000
Vortex: StormKing	$23M	$9.5M	$4.2M	$36.7M	$450,000
Vortex: Hydrovex	$14M	$9.5M	$4.2M	$27.7M	$450,000
Vortex: Sansep	$10M	$9.5M	$4.2M	$23.7M	$440,000
Ballasted Flocculation: DensaDeg	$17M	$9.5M	$4.2M	$30.7M	$660,000
Ballasted Flocculation: Actiflo	$33M	$9.5M	$4.2M	$46.7M	$675,000

7.3.5.5 Selected CSO Treatment and Disinfection Approach

JMEUC has selected Option A for further consideration in the LTCP because it simplifies the TAPS expansion and new force main design/construction, reduces stress on existing JMEUC WWTF unit processes and eliminates the need to increase unit capacities. Option A allows up to 55 mgd (consistent with trunk capacity) from the TAPS to enter the JMEUC WWTF via the North Barrel and be treated through all existing processes and directs the additional 85 mgd CSO flow to the WWTF via a separate force main, bypassing all existing treatment units. Therefore, the predicted flow through the existing JMEUC WWTF would be 153 mgd (238 mgd peak hour minus 85 mgd from TAPS). The bypassed flow (85 mgd) would be treated and blended with the normal WWTF effluent before discharging to the river, in accordance with an approved NJPDES permit authorizing bypass.

To meet the treatment objectives (effluent permit limits), JMEUC has selected Option 1 for further consideration in the LTCP. Option 1 includes: mechanical bar screens followed by fine screens along with a new high-rate disinfection pipeline that will also serve to convey the CSO flow to the new effluent pump station. Figure 7-17 presents the recommended flow path along with the expected blended effluent TSS and cBOD concentrations. This treatment option is not anticipated to have any impact on the JMEUC WWTF's solids handling unit processes.

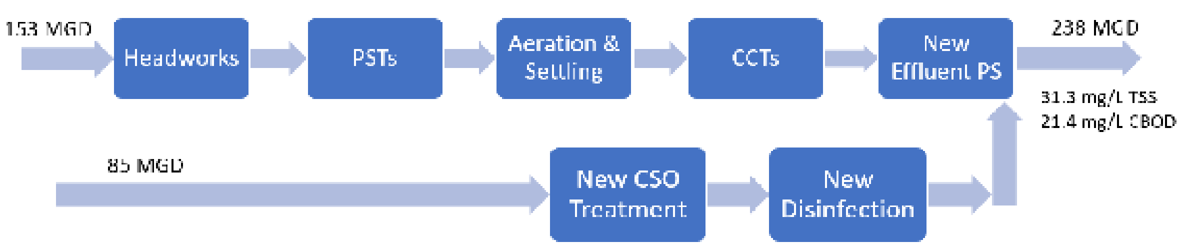

Figure 7-17: CSO Flow Path A with expected blended effluent TSS and cBOD concentrations

Figure 7-18 presents a planning-level site layout that includes the recommended unit processes: mechanical bar screens followed by fine screens, and the CSO treatment train chlorine contact pipeline.

Background plan and guidance on proposed facility location furnished courtesy of CME Associates Inc.

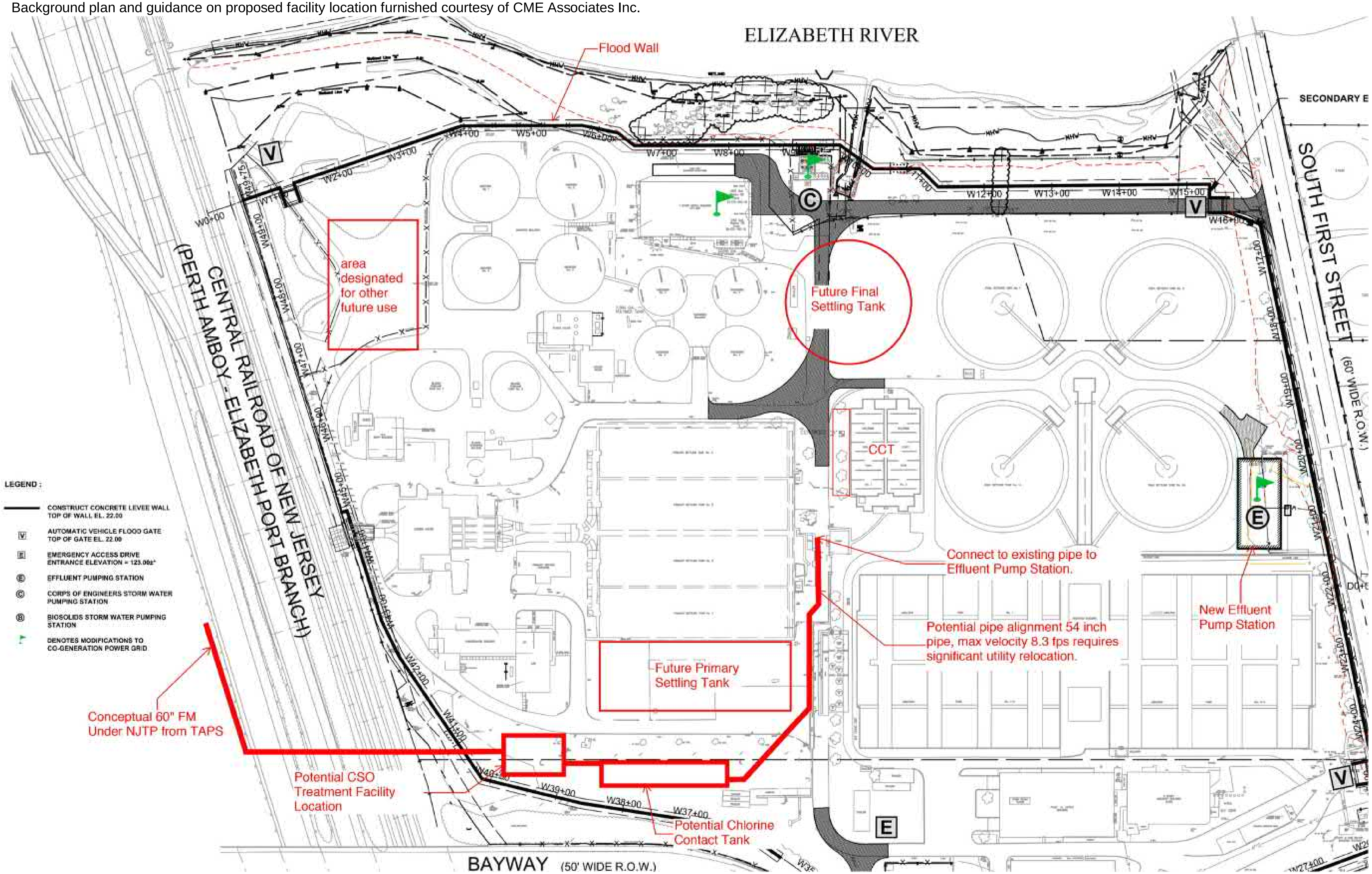

Figure 7-18: Potential Site Layout for CSO Treatment Train at JMEUC WWTF

7.3.6 Performance Summary

7.3.6.1 Control Program 3A

The Typical Year overflow performance for Control Program 3A, Increased CSO Treatment with Real Time Control, is summarized in Table 7-22. Compared to the 2015 baseline condition, the total annual CSO overflow volume is reduced from 1,068.5 MG to 892.7 MG, for a decrease of 175.8 MG. This CSO volume reduction improves the system-wide percent capture from 66.5% to 72.0% using Trenton Avenue Pump Station as the point of analysis, or from 83.1% to 85.9% using the JMEUC WWTF as the point of analysis. However, the overall number of system-wide CSO events per year does not decrease under Control Program 3A because the number of overflows at the outfalls with the most activations does not change.

Table 7-22: Control Program 3A – Increased CSO Treatment with 55 MGD Conveyance Real Time Control, Performance Summary

Outfall No.	Baseline 2015			Control Program 3A			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
001A	42	86.3	432	40	71.7	401	-2	-14.6	-32
002A	35	32.4	224	34	31.2	222	-1	-1.1	-1
003A	43	60.7	285	43	61.1	286	0	0.4	1
005A	54	96.6	593	52	101.5	580	-2	4.9	-13
008A	36	9.6	302	33	8.6	260	-3	-1.0	-42
010A	42	17.2	271	44	17.3	271	2	0.1	0
012A	44	5.8	355	33	4.0	242	-11	-1.8	-113
013A	42	16.9	313	40	16.5	282	-2	-0.4	-31
014A	13	1.1	16	11	0.9	14	-2	-0.2	-3
016A	46	16.7	367	45	15.1	337	-1	-1.6	-30
021A	19	1.4	32	14	1.1	28	-5	-0.4	-4
022A	46	71.3	591	39	47.9	527	-7	-23.4	-64
026A	53	53.2	613	53	44.1	602	0	-9.1	-11
027A	25	27.7	378	21	21.9	345	-4	-5.8	-33
028A	35	35.4	514	30	28.1	448	-5	-7.3	-65
029A	39	44.7	474	29	25.4	330	-10	-19.2	-144
030A	11	2.2	19	11	1.9	19	0	-0.2	0
031A	35	15.4	266	30	11.2	197	-5	-4.2	-69
032A	26	7.4	83	25	7.1	82	-1	-0.3	-1
034A	44	77.7	404	32	57.8	223	-12	-19.8	-180
035A	35	42.6	307	26	21.4	195	-9	-21.2	-112
036A	30	43.6	240	30	44.1	243	0	0.5	3
037A	44	64.6	463	30	26.7	182	-14	-37.8	-281
038A	30	8.6	224	4	0.0	14	-26	-8.6	-210
039A	27	9.9	88	27	9.0	88	0	-0.9	-1
040A	42	16.3	262	28	6.7	117	-14	-9.6	-145
041A	53	191.9	591	51	199.5	587	-2	7.6	-5
042A	19	11.5	54	19	11.0	53	0	-0.5	-2
043A	3	0.2	1	1	0.0	0	-2	-0.1	-1
Total	**-**	**1068.5**	**-**	**-**	**892.7**	**-**	**-**	**-175.8**	**-**

The performance of the control program for the September 28, 2004 event is depicted in Figure 7-19 and Figure 7-20, which show that with the control rules implemented, the total volume of flow conveyed to the JMEUC WWTF is increased without impacting the peak flow.

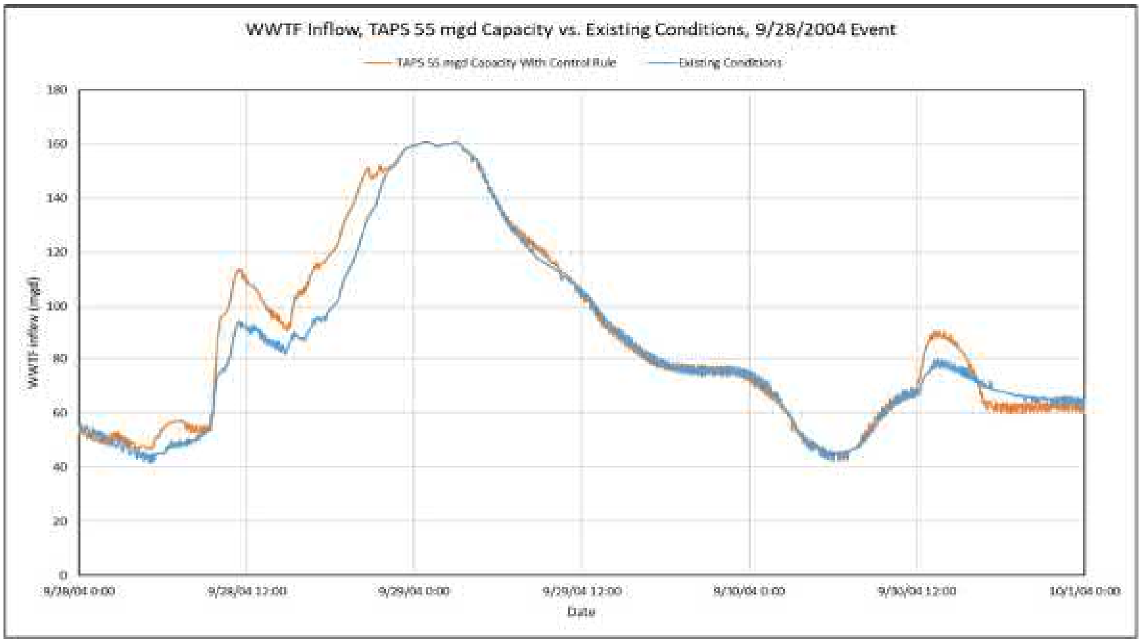

Figure 7-19: JMEUC influent flow for September 28th hydrograph with and without TAPS pumping rate increased to 55 mgd

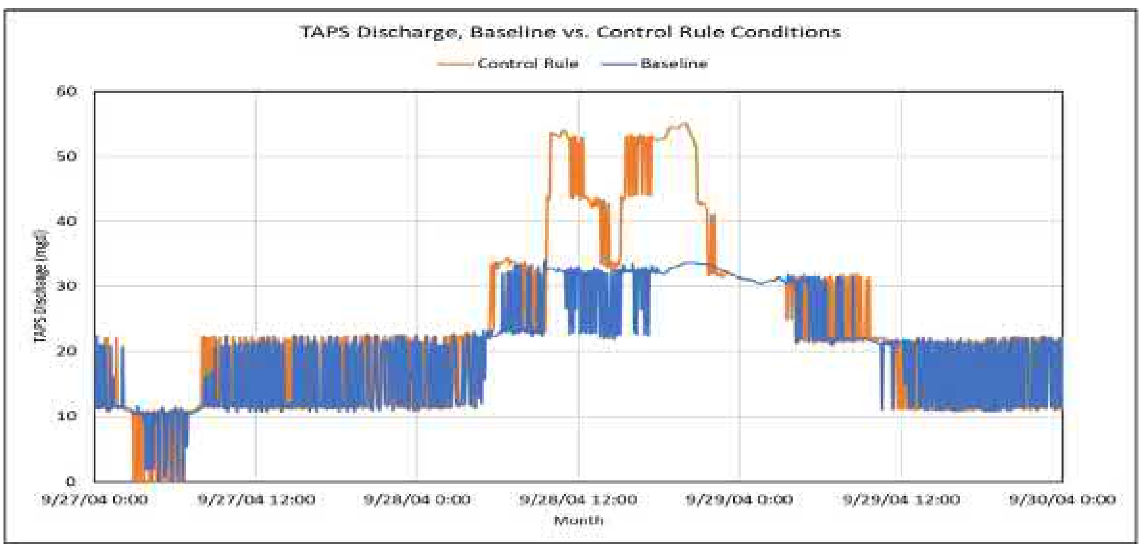

Figure 7-20: TAPS flow for September 28th hydrograph with and without TAPS pumping rate increased to 55 mgd.

7.3.6.2 Control Program 3B

In Table 7-23 through Table 7-26, the Typical Year CSO performance for each outfall is presented with a TAPS discharge capacity increasing to 65, 90, 120, and 140 mgd, respectively. A comparison against the 2015 baseline notes that the predicted decrease in the system-wide CSO volume is approximately 217, 290, 350, and 370 million gallons, respectively for these conveyance capacities.

Relative to a TAPS inflow analysis, the system-wide percent capture corresponding to the 65, 90, 120, and 140 mgd maximum pump rates is 73.3%, 75.6%, 77.5%, and 78.1%. These values, along with the percent capture relative to a JMEUC WWTF inflow analysis, are summarized in Table 7-27.

Table 7-23: Control Program 3B – Expanded WWTF for CSO Treatment with 65 MGD Conveyance, Performance Summary

Outfall No.	Baseline 2015			Control Program 3B, 65 mgd Conveyance			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
001A	42	86.3	432	40	66.4	374	-2	-19.9	-58
002A	35	32.4	224	34	30.8	222	-1	-1.5	-1
003A	43	60.7	285	43	61.1	286	0	0.3	1
005A	54	96.6	593	51	105.3	558	-3	8.7	-35
008A	36	9.6	302	32	8.1	227	-4	-1.5	-74
010A	42	17.2	271	44	17.2	271	2	0.0	-1
012A	44	5.8	355	32	3.4	193	-12	-2.5	-163
013A	42	16.9	313	39	16.3	279	-3	-0.6	-34
014A	13	1.1	16	9	0.8	13	-4	-0.2	-3
016A	46	16.7	367	45	14.6	321	-1	-2.2	-46
021A	19	1.4	32	13	1.1	27	-6	-0.4	-6
022A	46	71.3	591	39	42.7	517	-7	-28.6	-74
026A	53	53.2	613	53	42.0	599	0	-11.1	-13
027A	25	27.7	378	19	19.9	296	-6	-7.8	-82
028A	35	35.4	514	28	26.0	399	-7	-9.4	-115
029A	39	44.7	474	29	20.5	305	-10	-24.2	-169
030A	11	2.2	19	11	1.9	18	0	-0.3	-1
031A	35	15.4	266	30	10.1	185	-5	-5.2	-81
032A	26	7.4	83	25	7.0	82	-1	-0.4	-1
034A	44	77.7	404	31	51.9	171	-13	-25.8	-233
035A	35	42.6	307	23	16.7	129	-12	-25.9	-178
036A	30	43.6	240	30	44.1	243	0	0.5	3
037A	44	64.6	463	26	18.2	117	-18	-46.3	-345
038A	30	8.6	224	3	0.0	7	-27	-8.6	-217
039A	27	9.9	88	27	9.0	88	0	-0.9	-1
040A	42	16.3	262	27	4.3	98	-15	-11.9	-164
041A	53	191.9	591	50	201.3	566	-3	9.4	-25
042A	19	11.5	54	18	10.7	47	-1	-0.8	-7
043A	3	0.2	1	1	0.0	0	-2	-0.2	-1
Total	**-**	**1068.5**	**-**	**-**	**851.3**	**-**	**-**	**-217.2**	**-**

Table 7-24: Control Program 3B – Expanded WWTF for CSO Treatment with 90 MGD Conveyance, Performance Summary

Outfall No.	Baseline 2015			Control Program 3B, 90 mgd Conveyance			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
001A	42	86.3	432	40	58.1	373	-2	-28.3	-59
002A	35	32.4	224	34	30.0	222	-1	-2.4	-2
003A	43	60.7	285	43	61.1	286	0	0.3	1
005A	54	96.6	593	51	106.0	613	-3	9.4	20
008A	36	9.6	302	29	4.4	145	-7	-5.3	-157

Outfall No.	Baseline 2015			Control Program 3B, 90 mgd Conveyance			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
010A	42	17.2	271	43	17.1	270	1	-0.1	-1
012A	44	5.8	355	7	0.3	12	-37	-5.5	-343
013A	42	16.9	313	38	14.1	265	-4	-2.7	-48
014A	13	1.1	16	2	0.0	2	-11	-1.0	-14
016A	46	16.7	367	44	12.1	293	-2	-4.6	-74
021A	19	1.4	32	11	0.9	24	-8	-0.5	-8
022A	46	71.3	591	36	29.5	452	-10	-41.8	-139
026A	53	53.2	613	53	38.6	598	0	-14.6	-15
027A	25	27.7	378	19	15.8	295	-6	-11.9	-83
028A	35	35.4	514	28	23.1	399	-7	-12.3	-115
029A	39	44.7	474	29	15.1	326	-10	-29.6	-147
030A	11	2.2	19	11	1.7	18	0	-0.4	-1
031A	35	15.4	266	30	8.9	183	-5	-6.4	-83
032A	26	7.4	83	25	6.9	82	-1	-0.5	-1
034A	44	77.7	404	29	42.1	151	-15	-35.6	-253
035A	35	42.6	307	23	11.3	120	-12	-31.2	-187
036A	30	43.6	240	30	44.1	243	0	0.5	3
037A	44	64.6	463	23	11.5	62	-21	-53.0	-400
038A	30	8.6	224	3	0.0	7	-27	-8.6	-218
039A	27	9.9	88	27	8.9	88	0	-1.0	-1
040A	42	16.3	262	26	3.1	79	-16	-13.1	-182
041A	53	191.9	591	50	206.2	609	-3	14.3	17
042A	19	11.5	54	14	7.8	38	-5	-3.7	-16
043A	3	0.2	1	1	0.0	0	-2	-0.2	-1
Total	**-**	**1068.5**	**-**	**-**	**778.5**	**-**	**-**	**-290.0**	**-**

Table 7-25: Control Program 3B – Expanded WWTF for CSO Treatment with 120 MGD Conveyance, Performance Summary

Outfall No.	Baseline 2015			Control Program 3B, 120 mgd Conveyance			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
001A	42	86.3	432	40	54.7	372	-2	-31.7	-60
002A	35	32.4	224	34	30.0	222	-1	-2.4	-2
003A	43	60.7	285	43	61.1	286	0	0.3	1
005A	54	96.6	593	51	88.2	607	-3	-8.4	15
008A	36	9.6	302	29	4.3	145	-7	-5.3	-156
010A	42	17.2	271	43	17.0	270	1	-0.2	-1
012A	44	5.8	355	8	0.3	12	-36	-5.5	-343
013A	42	16.9	313	37	14.2	265	-5	-2.7	-48
014A	13	1.1	16	3	0.0	3	-10	-1.0	-14
016A	46	16.7	367	45	12.4	294	-1	-4.3	-73
021A	19	1.4	32	11	0.9	24	-8	-0.5	-8
022A	46	71.3	591	36	29.1	465	-10	-42.2	-126
026A	53	53.2	613	53	38.3	598	0	-14.8	-15
027A	25	27.7	378	16	7.6	243	-9	-20.1	-135
028A	35	35.4	514	23	12.7	304	-12	-22.7	-209
029A	39	44.7	474	29	14.3	326	-10	-30.3	-148

Outfall No.	Baseline 2015			Control Program 3B, 120 mgd Conveyance			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
030A	11	2.2	19	11	1.7	18	0	-0.4	-1
031A	35	15.4	266	30	8.8	183	-5	-6.6	-83
032A	26	7.4	83	25	6.9	82	-1	-0.5	-1
034A	44	77.7	404	29	39.1	149	-15	-38.6	-254
035A	35	42.6	307	13	2.4	32	-22	-40.2	-275
036A	30	43.6	240	30	44.1	243	0	0.5	3
037A	44	64.6	463	23	10.0	57	-21	-54.5	-406
038A	30	8.6	224	0	0.0	0	-30	-8.6	-224
039A	27	9.9	88	27	8.9	88	0	-1.0	-1
040A	42	16.3	262	13	0.8	18	-29	-15.5	-243
041A	53	191.9	591	52	202.7	608	-1	10.8	17
042A	19	11.5	54	14	7.9	38	-5	-3.6	-16
043A	3	0.2	1	0	0.0	0	-3	-0.2	-1
Total	**-**	**1068.5**	**-**	**-**	**718.4**	**-**	**-**	**-350.1**	**-**

Table 7-26: Control Program 3B – Expanded WWTF for CSO Treatment with 140 MGD Conveyance, Performance Summary

Outfall No.	Baseline 2015			Control Program 3B, 140 mgd Conveyance			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
001A	42	86.3	432	40	53.1	372	-2	-33.2	-60
002A	35	32.4	224	34	29.1	221	-1	-3.2	-2
003A	43	60.7	285	40	59.4	270	-3	-1.4	-15
005A	54	96.6	593	51	85.7	532	-3	-10.9	-60
008A	36	9.6	302	29	4.2	145	-7	-5.4	-156
010A	42	17.2	271	42	16.6	270	0	-0.6	-2
012A	44	5.8	355	8	0.3	12	-36	-5.5	-343
013A	42	16.9	313	37	13.8	265	-5	-3.1	-48
014A	13	1.1	16	3	0.0	3	-10	-1.0	-14
016A	46	16.7	367	45	12.1	293	-1	-4.7	-74
021A	19	1.4	32	11	0.9	24	-8	-0.6	-8
022A	46	71.3	591	36	28.3	465	-10	-43.0	-126
026A	53	53.2	613	53	37.3	591	0	-15.9	-22
027A	25	27.7	378	16	7.4	243	-9	-20.3	-135
028A	35	35.4	514	23	12.3	304	-12	-23.1	-209
029A	39	44.7	474	29	13.9	287	-10	-30.7	-187
030A	11	2.2	19	11	1.7	17	0	-0.5	-2
031A	35	15.4	266	30	8.6	183	-5	-6.8	-83
032A	26	7.4	83	25	6.7	82	-1	-0.7	-1
034A	44	77.7	404	29	38.0	149	-15	-39.7	-255
035A	35	42.6	307	13	2.3	32	-22	-40.2	-275
036A	30	43.6	240	30	42.8	233	0	-0.7	-7
037A	44	64.6	463	21	9.7	54	-23	-54.8	-408
038A	30	8.6	224	0	0.0	0	-30	-8.6	-224
039A	27	9.9	88	27	8.6	88	0	-1.3	-1
040A	42	16.3	262	13	0.7	18	-29	-15.5	-243
041A	53	191.9	591	52	197.0	524	-1	5.1	-67

Outfall No.	Baseline 2015			Control Program 3B, 140 mgd Conveyance			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
042A	19	11.5	54	14	7.7	38	-5	-3.9	-16
043A	3	0.2	1	0	0.0	0	-3	-0.2	-1
Total	**-**	**1068.5**	**-**	**-**	**698.2**	**-**	**-**	**-370.3**	**-**

Table 7-27: Control Program 3B – Percent Capture by Conveyance Capacity Rate

TAPS Conveyance Capacity (mgd)	Additional Capture Volume (MG)	% Capture, TAPS Inflow	% Capture, JMEUC Inflow
65	217	73.3	86.6
90	290	75.6	87.7
120	350	77.5	88.7
140	370	78.1	89.0

7.3.7 Cost Summary

Table 7-28 indicates the estimated capital costs, annual O&M costs, and life cycle cost for the Trenton Avenue Pump Station improvements identified under Control Program 3A. The costs for the mechanical bar screen replacement were developed based on the 2018 PVSC TGM document, while costs for the screenings handling system, pumping systems, real time controls, and structural modifications were estimated from manufacturer budgetary proposals for equipment costs and cost data from similar projects. Operating and maintenance costs is 1% of the construction cost.

Table 7-28: Control Program 3A - Increased CSO Treatment with Real Time Control, Cost Summary

Cost Item	Estimated Cost ($ Million)
Pumping Systems	$3.71
Mechanical Bar Screens	$1.65
Screenings Handling	$0.65
Structural Modifications	$0.70
Real Time Controls	$0.54
Total Construction Costs	*$7.25*
Land Acquisition Costs	$0.00
Non-Construction Costs	$1.81
Total Capital Cost	**$9.06**
Annual O&M Cost	$0.07
Total Present Worth	**$10.2**
Item	**Value**
CSO Volume Abated (MG)	176
Cost per Gallon Abated ($/gal)	$0.06

As noted in Section 7.3.5, the estimated total construction cost for the new CSO treatment and disinfection train at the JMEUC WWTF under Control Program 3B is $16.3 million, based on mechanical bar screen followed by fine screening treatment prior to chlorination/dechlorination approach. In addition to JMEUC WWTF CSO treatment, this control program would include a range of improvements across the combined sewer system that depend on the increased conveyance capacity to be achieved. The construction of a new pump station and force main to supplement or replace the existing TAPS would be necessary for capacities greater than about 65 mgd. Further modifications would be required to replace or relieve sections of the Westerly Interceptor and then the Easterly Interceptor, along with modifications to regulators for additional discharge capacities.

Based on data presented in the Elizabeth 2007 Cost and Performance Analysis Report, the construction costs in 2019 dollars for the CSS improvements with increased conveyance above 65 mgd varied from approximately $20 million to $45 million. Table 7-29 incorporates the higher value from this range in a cost summary for Control Program 3B associated with a 140 mgd conveyance rate, along with costs for the WWTF CSO treatment facility and the upgrade to the existing TAPS. As the selection of the LTCP approach progresses, the costs for the CSS conveyance improvements would be refined and coordinated with the overall targeted capacity.

Table 7-29: Control Program 3B – Expanded WWTF for CSO Treatment, Cost Summary

Cost Item	Estimated Cost ($ Million)
Existing TAPS Upgrade	$7.25
WWTF CSO Treatment Train	$16.30
CSS Conveyance Improvements	$45.0
Total Construction Costs ($ Million)	*$68.6*
Land Acquisition Costs ($ Million)	$0.00
Non-Construction Costs ($ Million)	$17.1
Total Capital Cost ($ Million)	**$85.7**
Annual O&M Cost ($ Million)	$1.01
Total Present Worth ($ Million)	**$101.1**
Item	**Value**
CSO Volume Abated (MG)	370
Cost per Gallon Abated ($/gal)	$0.27

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7.4 Control Program 4: Satellite Storage Facilities

7.4.1 Description

This control program involves the siting of storage tanks near CSO outfalls. Each facility consists of:

- A diversion structure;
- 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 towards the JMEUC treatment plant.
- Increased pumping capacity at the Trenton Avenue Pump Station (TAPS)

The required sizing of storage tanks for various control levels was determined, and the storage tanks were input into the model to identify any impacts to CSO reduction. 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 sizing of these satellite storage facilities is based on increased CSO conveyance and treatment, with the pumping capacity at the TAPS taken to be upgraded to 65 mgd. It is noted that conveyance of 65 MGD to TAPS produces a conservative estimate for storage sizing because more conveyance to TAPS would reduce satellite storage required. CSO treatment costs outlined in Control Program 3 are based on higher flows to the WWTF (140 MGD) from TAPS. Given this increased conveyance of 65 MGD to TAPS, the cost analysis for this control program also includes capital and operating costs for a CSO wet weather treatment facility at the JMEUC. As such, elements of Control Program 3 are incorporated in this control program. It should also be noted that the stored flow will be dewatered to the JMEUC WWTF as capacity in the interceptor sewers and WWTF is restored post-event. This represents a significant volume of additional flow to be treated annually at the WWTF and the associated O&M costs have not yet been estimated.

7.4.2 Analysis

Each outfall location has been evaluated below to determine the area that would be required for a satellite storage facility. It was assumed that the configuration and function of each satellite storage facility would be similar. Flow to the outfall would be redirected to an off-line underground storage tank and would be stored up to the tank volume, which is selected based on the targeted level of control. Flow in excess of the tank capacity would be discharged as overflow. Following the wet weather event, the tank would be dewatered and flow sent to the interceptor. It is noted that if end-of-pipe storage is selected, analysis of available interceptor capacity would be required.

For each basin, tank volume was determined based on the control levels of 0, 4, 8, 12, and 20 overflows per year. It was assumed that a satellite storage tank would have a depth of 15 feet, and the facility would also include dewatering pumps, screens, and connecting pipes. Volume requirements for each of the outfalls at the various control levels are summarized in the Table 7-30. Converting overflow volumes into a required tank footprint based on assumed water storage depth of 15 feet produces the areas indicated.

Table 7-30: Storage Volume and Tank Areas Requirements by Outfall and Control Level

	Tank Volume (MG)					Tank Area (Acres)				
	Overflows per Year					Overflows per Year				
Outfall No.	0	4	8	12	20	0	4	8	12	20
001A	12.5	4.9	2.4	2.2	1.0	2.55	1.01	0.48	0.44	0.21
002A	4.7	2.0	1.2	1.2	0.5	0.96	0.40	0.25	0.25	0.10

	Tank Volume (MG)					Tank Area (Acres)				
	Overflows per Year					Overflows per Year				
Outfall No.	**0**	**4**	**8**	**12**	**20**	**0**	**4**	**8**	**12**	**20**
003A	8.8	4.4	2.8	2.8	0.8	1.80	0.91	0.58	0.58	0.16
005A	10.5	6.8	6.8	6.8	6.8	2.15	1.39	1.39	1.39	1.39
008A	1.4	0.5	0.4	0.2	0.2	0.29	0.10	0.08	0.05	0.03
010A	2.5	1.1	0.7	0.6	0.3	0.51	0.22	0.14	0.13	0.06
012A	0.5	0.2	0.2	0.1	0.1	0.11	0.04	0.04	0.03	0.02
013A	2.5	1.0	0.7	0.6	0.3	0.51	0.21	0.14	0.12	0.06
014A	0.2	0.1	0.0	0.0	0.0	0.03	0.02	0.01	0.01	0.00
016A	2.1	0.8	0.6	0.5	0.3	0.44	0.17	0.12	0.10	0.05
021A	0.2	0.1	0.0	0.0	0.0	0.05	0.03	0.01	0.01	0.00
022A	6.6	2.6	1.8	1.3	0.8	1.36	0.54	0.37	0.27	0.16
026A	5.6	2.2	1.7	1.2	0.7	1.15	0.44	0.34	0.25	0.15
027A	4.2	1.4	1.4	0.9	0.3	0.85	0.29	0.28	0.18	0.05
028A	4.5	1.9	1.1	1.1	0.3	0.92	0.39	0.23	0.23	0.06
029A	3.2	1.8	0.9	0.9	0.4	0.65	0.37	0.18	0.18	0.09
030A	0.6	0.3	0.1	0.1	0.0	0.13	0.06	0.03	0.03	0.00
031A	1.6	0.8	0.5	0.5	0.2	0.33	0.16	0.10	0.10	0.03
032A	0.9	0.5	0.4	0.4	0.1	0.19	0.11	0.08	0.08	0.02
034A	9.6	3.8	2.3	1.8	1.1	1.95	0.77	0.46	0.36	0.22
035A	3.4	1.8	1.0	0.7	0.5	0.69	0.37	0.21	0.14	0.10
036A	7.2	3.3	1.9	1.9	0.7	1.48	0.67	0.38	0.38	0.15
037A	3.7	1.3	0.6	0.6	0.3	0.76	0.27	0.13	0.13	0.06
038A	0.0	0.0	0.0	0.0	0.0	0.00	0.00	0.00	0.00	0.00
039A	1.5	0.9	0.4	0.4	0.1	0.31	0.17	0.09	0.09	0.03
040A	0.8	0.5	0.2	0.2	0.1	0.17	0.10	0.05	0.05	0.02
041A	23.3	10.3	9.3	6.7	5.1	4.77	2.11	1.89	1.38	1.05
042A	1.8	1.0	0.6	0.6	0.0	0.37	0.20	0.11	0.11	0.00
043A	Included with 035A					Included with 035A				
Total	**124.5**	**56.3**	**39.7**	**34.4**	**21.0**	**25.5**	**11.5**	**8.1**	**7.0**	**4.3**

The sizing of storage control facilities in the InfoWorks model for multiple outfalls is time consuming to model the facilities, computationally expensive to run test simulation, 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.

Time series data at a 15 min timestep for each overflow is available from the InfoWorks 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. Once the storage facility is calculated to be full, remaining flows are tracked as overflows. The tank volume is tracked and once the volume is exceeded the tank is considered full and no additional volume is accepted.

Dewatering was only applied to the storage if there was no overflow from the regulator for a minimum of 12 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 a period of 12 hours with no overflow. A sample output can be seen below in Figure 7-21. Overflows beyond the tank

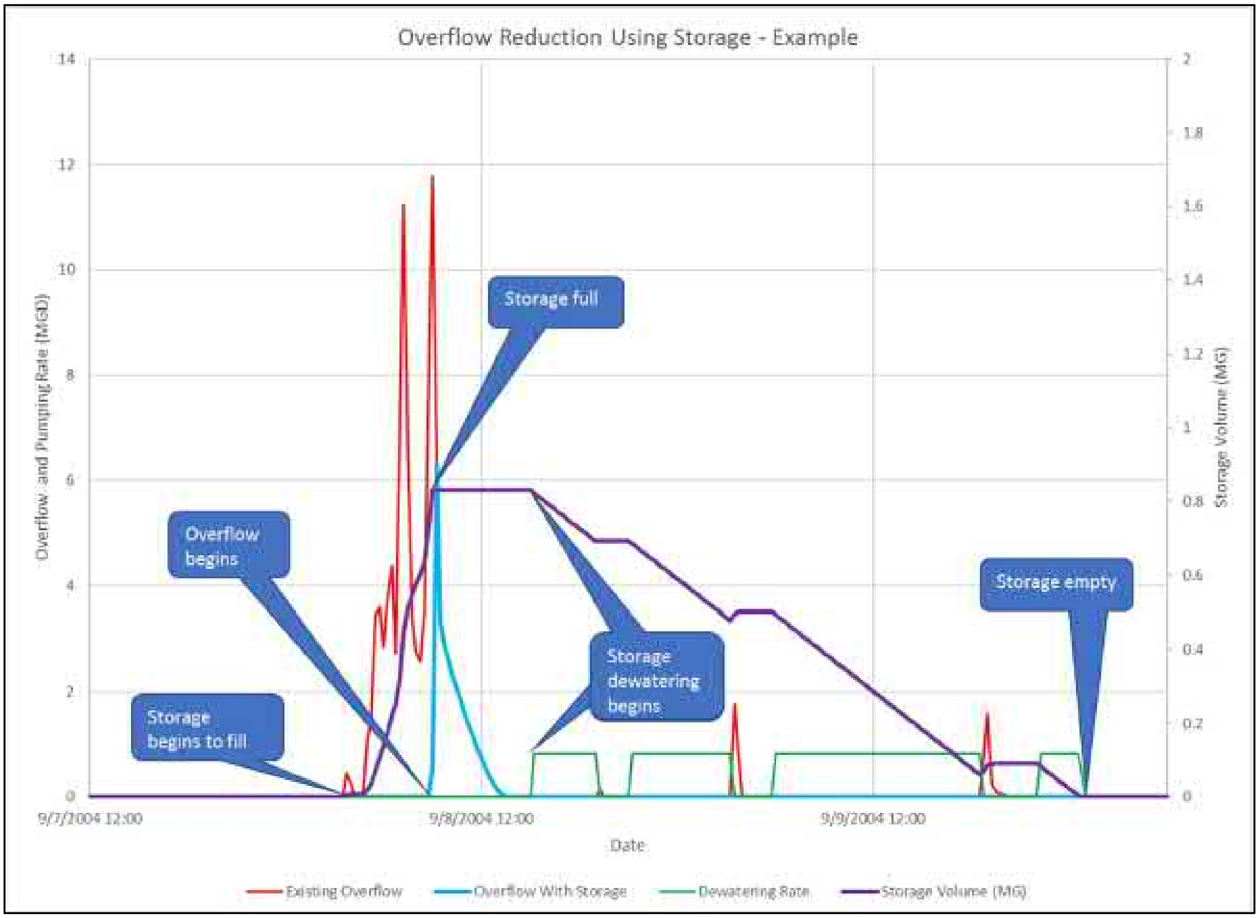

Figure 7-21: Example Hydrograph for Tank Storage Analysis

volume were tracked and a list of storms identifying remaining overflows generated. The list of remaining overflows generated was compared to the allowable list, and the storage volume increased until the remaining storms consisted only of those allowable on the systemwide basis. The resulting tank volumes were then modeled in the InfoWorks model. A typical model configuration is shown below in Figure 7-22.

7.4.3 Institutional Issues

The institutional issues surrounding Control Program 4 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 required:

- Waterfront Development Permit
- Flood Hazard Area Permit
- USACE Nationwide 404 Permit
- Local Permits
- NJDEP Treatment Works Approval
- Soil Erosion and Sediment Control 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.

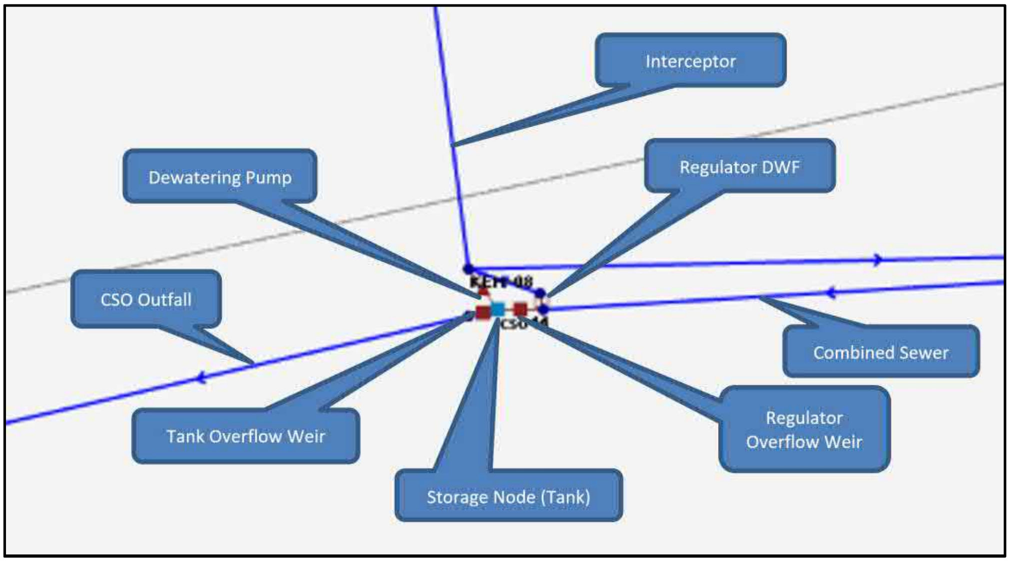

Figure 7-22: Point Storage - Sample Model Configuration

Permitting requirements to construct a satellite storage facility will impact the feasibility and favorability of a site. Depending on site ownership, proximity to regulated waters, natural heritage considerations, or other regulatory considerations, each site will be evaluated for the type of permits needed as well as the length of time and level of difficulty it would take to receive those permits.

An example of a site with permitting configures for a potential storage facility site for Outfalls 030A and 031A, shown in Figure 7-23 below. The site is known to be classified as Green Acres land, which limits the type of development that would be allowed on the site. Due to its proximity to the Arthur Kill, it is also known to be within the 100-year flood hazard area limit, which further limits the type of allowable development and will require a permit for construction from NJDEP. The site would be further evaluated to determine if, based on the regulatory requirements, it would be a good candidate for a satellite storage facility.

7.4.4 Implementability

Installation of satellite storage tanks in urban areas can be challenging due to space and access limitations. The size of a satellite storage facility is limited by the available space in the vicinity of the outfall. In urbanized areas such as the City of Elizabeth, there is limited open space, as a result this could prevent a satellite storage facility from being viable.

The tanks have been sized assuming a 15-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 25 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 in the area of construction must be relocated, protected, or supported.

Control of groundwater may be a significant challenge, as noted previously, groundwater is thought to be shallow throughout the City.

There is little available information on the soil characteristics at the tank sites, however, given the depth to bedrock and proximity 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

Figure 7-23: Example of Site with Permitting Considerations

levels could produce inundation with little rainfall meaning the tank would be empty and prone to floatation.

An example of site implementability related to available land is presented for Outfall 034A. As can be seen from Table 7-30 above, Outfall 034A requires 2.29 acres of open area to house a storage facility to control CSOs to zero events in the Typical Year. Figure 7-24 below shows an available area of 193' x 193' (0.86 ac) near the outfall. While this area is not sufficient to control to zero overflows, it is large enough for a storage facility that can control to four overflows in the Typical Year. An example layout of this site for a control level of 4 overflows is shown below.

Site ownership may also impact the viability of a satellite storage facility. Ownership by a public entity would generally be preferred to a private owner, due to access required for construction and ongoing maintenance, as well as granting of easements or property acquisition.

Figure 7-25 below depicts a potential satellite storage site for Outfall 001A. The site is 215' x 215' (1.06 ac) which is sufficient to control to four (4) overflows per year. The site is owned by NJDOT and is located near Newark Airport between Spring Street and U.S. Highway 1, 550 feet west of Outfall 001A. Working with a public agency to obtain access to the site is preferable to a private owner. It is noted that at this site, diversion and return pipes for this site would need to cross several major highways to reach the outfall on the other side of US 1-9 and Route 81. Construction on the site and across the highways would require extensive NJDOT approvals and easement grants. There would also likely be some traffic disruption for site access during construction as well as for tank maintenance.

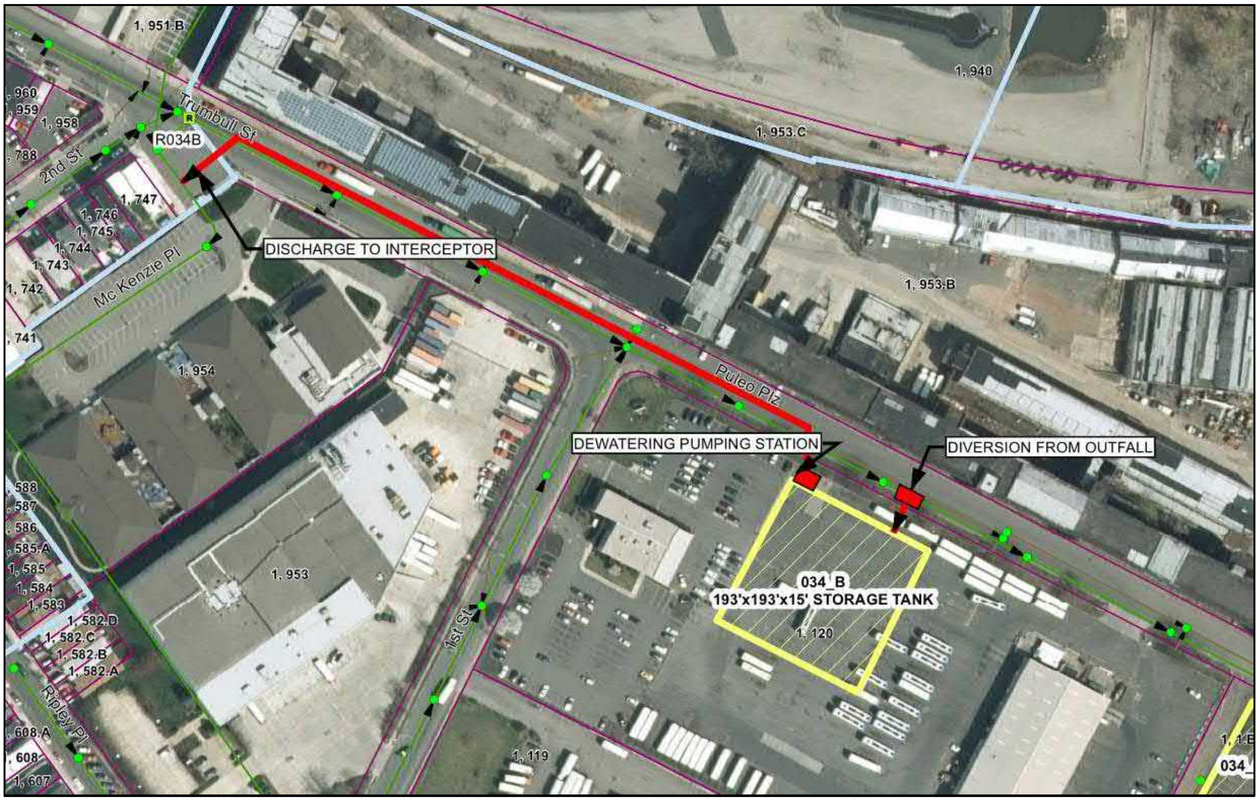

Figure 7-24: Example Storage Tank Layout for Outfall 034

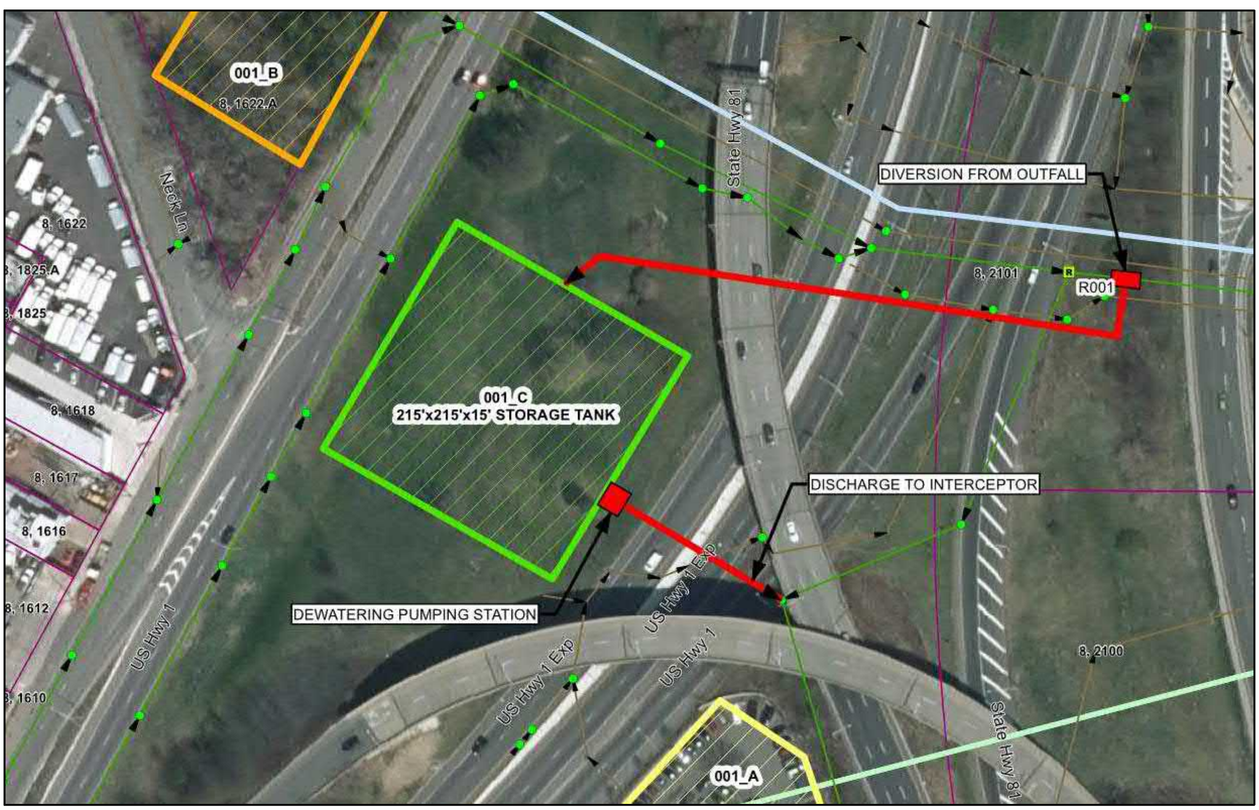

Figure 7-25: Example Storage Tank Layout for Outfall 001

7.4.5 Public Acceptance

The construction required for storage tanks is large and invasive making public acceptance of the project a concern. Once construction is completed tanks are generally preferable from the stand point of public acceptance since the majority of the facility is underground. Aboveground features will still be required such as electrical facilities, odor control, access points to pumps, flushing systems, and access ways to the tanks for periodic maintenance.

Another significant consideration in evaluating the feasibility of a site is determining the length and degree of disruption that this facility would have on the surrounding community. During construction, this would include traffic impacts, road closures, noise or odor pollution, and access to the site. Following construction, this may include continued disruption to surrounding businesses and residents caused by the use of the site, access to the site for maintenance of the facility, alternate potential uses of that land, and odor considerations.

An example of a site with public acceptance considerations is the potential end-of-pipe storage site for Outfall 002A, as shown in Figure 7-26 below. The site is 105' x 280' (0.67 ac) which is sufficient to control to four (4) overflows per year. The site is located in the parking area of a warehouse distribution center, adjacent to Outfall 002A and has a private owner. There is potential additional space available in the triangular grass area to the north of the site.

Selection of this site would involve potential interferences with existing infrastructure and would likely result in disruption to business operations both during construction due to the narrow access laneway as well as post-construction when access to the site is needed for maintenance. This site would also result in a loss of parking spaces, and cooperation with the site owner would be required to obtain easements for site access and permanent facilities.

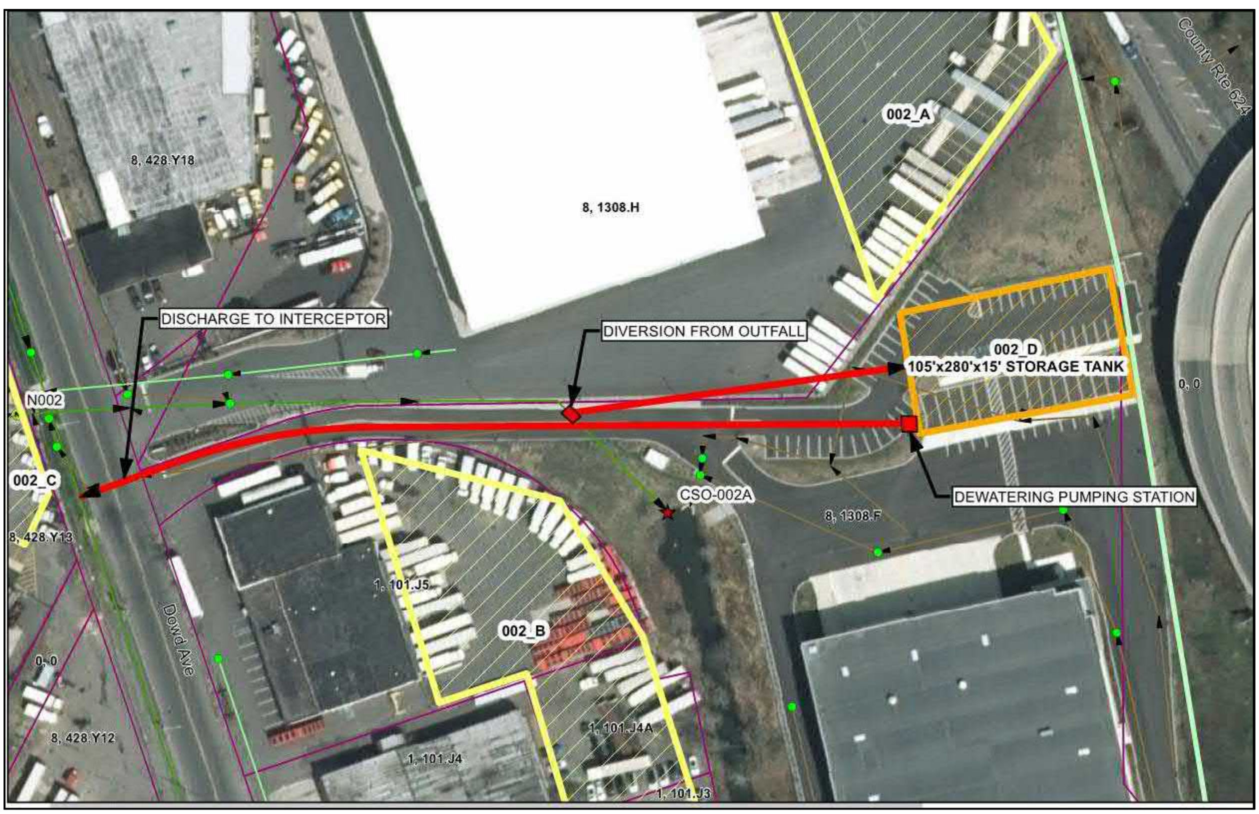

Figure 7-26: Example Storage Tank Layout for Outfall 002A

7.4.6 Performance Summary

The performance of Control Program 4 is summarized in Table 7-31 through Table 7-35, which show the Typical Year annual overflow details associated a control level of 0, 4, 8, 12, and 20 overflows per year, respectively.

Table 7-31: Control Program 4 – Satellite Storage, Performance Summary – 0 Overflows

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
001A	42	86.3	432	0	0.0	0	-42	-86.3	-432
002A	35	32.4	224	0	0.0	0	-35	-32.4	-224
003A	43	60.7	285	0	0.0	0	-43	-60.7	-285
005A	54	96.6	593	0	0.0	0	-54	-96.6	-593
008A	36	9.6	302	0	0.0	0	-36	-9.6	-302
010A	42	17.2	271	0	0.0	0	-42	-17.2	-271
012A	44	5.8	355	0	0.0	0	-44	-5.8	-355
013A	42	16.9	313	0	0.0	0	-42	-16.9	-313
014A	13	1.1	16	0	0.0	0	-13	-1.1	-16
016A	46	16.7	367	0	0.0	0	-46	-16.7	-367
021A	19	1.4	32	0	0.0	0	-19	-1.4	-32
022A	46	71.3	591	0	0.0	0	-46	-71.3	-591
026A	53	53.2	613	0	0.0	0	-53	-53.2	-613
027A	25	27.7	378	0	0.0	0	-25	-27.7	-378
028A	35	35.4	514	0	0.0	0	-35	-35.4	-514
029A	39	44.7	474	0	0.0	0	-39	-44.7	-474
030A	11	2.2	19	0	0.0	0	-11	-2.2	-19
031A	35	15.4	266	0	0.0	0	-35	-15.4	-266
032A	26	7.4	83	0	0.0	0	-26	-7.4	-83
034A	44	77.7	404	0	0.0	0	-44	-77.7	-404
035A	35	42.6	307	0	0.0	0	-35	-42.6	-307
036A	30	43.6	240	0	0.0	0	-30	-43.6	-240
037A	44	64.6	463	0	0.0	0	-44	-64.6	-463
038A	30	8.6	224	0	0.0	0	-30	-8.6	-224
039A	27	9.9	88	0	0.0	0	-27	-9.9	-88
040A	42	16.3	262	0	0.0	0	-42	-16.3	-262
041A	53	191.9	591	0	0.0	0	-53	-191.9	-591
042A	19	11.5	54	0	0.0	0	-19	-11.5	-54
043A	3	0.2	1	0	0.0	0	-3	-0.2	-1
Total	**-**	**1068.5**	**-**	**-**	**0.0**	**-**	**-**	**-1068.5**	**-**

Table 7-32: Control Program 4 - Satellite Storage, Performance Summary – 4 Overflows

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
001A	42	86.3	432	2	9.7	10	-40	-76.6	-422
002A	35	32.4	224	4	5.2	12	-31	-27.2	-211
003A	43	60.7	285	4	11.1	12	-39	-49.6	-273
005A	54	96.6	593	2	6.0	14	-52	-90.6	-578
008A	36	9.6	302	4	1.2	12	-32	-8.4	-290
010A	42	17.2	271	4	2.4	12	-38	-14.8	-259
012A	44	5.8	355	2	0.4	10	-42	-5.5	-345

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
013A	42	16.9	313	4	2.3	11	-38	-14.6	-302
014A	13	1.1	16	3	0.1	1	-10	-1.0	-15
016A	46	16.7	367	4	2.1	13	-42	-14.6	-354
021A	19	1.4	32	3	0.2	4	-16	-1.2	-28
022A	46	71.3	591	2	5.3	10	-44	-66.0	-581
026A	53	53.2	613	4	5.1	14	-49	-48.1	-599
027A	25	27.7	378	4	5.6	25	-21	-22.1	-353
028A	35	35.4	514	4	4.2	24	-31	-31.2	-490
029A	39	44.7	474	2	2.5	7	-37	-42.1	-467
030A	11	2.2	19	1	0.3	1	-10	-1.8	-18
031A	35	15.4	266	4	1.8	9	-31	-13.6	-257
032A	26	7.4	83	4	1.0	4	-22	-6.3	-79
034A	44	77.7	404	2	7.5	10	-42	-70.2	-394
035A	35	42.6	307	2	2.5	7	-33	-40.1	-300
036A	30	43.6	240	4	6.9	11	-26	-36.7	-229
037A	44	64.6	463	4	3.8	12	-40	-60.7	-451
038A	30	8.6	224	0	0.0	0	-30	-8.6	-224
039A	27	9.9	88	3	0.9	5	-24	-9.0	-84
040A	42	16.3	262	1	0.3	1	-41	-15.9	-261
041A	53	191.9	591	2	17.4	13	-51	-174.5	-579
042A	19	11.5	54	4	2.5	7	-15	-9.1	-47
043A	3	0.2	1	0	0.0	0	-3	-0.2	-1
Total	**-**	**1068.5**	**-**	**-**	**108.2**	**-**	**-**	**-960.3**	**-**

Table 7-33: Control Program 4 - Satellite Storage, Performance Summary – 8 Overflows

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
001A	42	86.3	432	8	24.4	29	-34	-61.9	-403
002A	35	32.4	224	7	10.2	20	-28	-22.1	-203
003A	43	60.7	285	7	21.9	20	-36	-38.8	-265
005A	54	96.6	593	2	6.0	14	-52	-90.6	-578
008A	36	9.6	302	7	2.0	20	-29	-7.7	-282
010A	42	17.2	271	7	5.0	22	-35	-12.2	-249
012A	44	5.8	355	3	0.4	10	-41	-5.5	-345
013A	42	16.9	313	7	4.3	20	-35	-12.6	-293
014A	13	1.1	16	6	0.5	6	-7	-0.5	-10
016A	46	16.7	367	8	3.7	22	-38	-13.0	-345
021A	19	1.4	32	6	0.7	9	-13	-0.8	-23
022A	46	71.3	591	8	9.8	35	-38	-61.5	-556
026A	53	53.2	613	8	8.0	23	-45	-45.2	-590
027A	25	27.7	378	5	5.9	29	-20	-21.8	-349
028A	35	35.4	514	7	9.2	37	-28	-26.2	-476
029A	39	44.7	474	7	7.6	26	-32	-37.1	-448
030A	11	2.2	19	6	0.9	3	-5	-1.3	-16
031A	35	15.4	266	7	3.7	16	-28	-11.7	-250
032A	26	7.4	83	7	2.0	8	-19	-5.4	-75
034A	44	77.7	404	7	15.1	20	-37	-62.6	-384
035A	35	42.6	307	4	5.0	12	-31	-37.6	-295

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
036A	30	43.6	240	7	15.9	20	-23	-27.7	-220
037A	44	64.6	463	8	8.4	23	-36	-56.2	-440
038A	30	8.6	224	0	0.0	0	-30	-8.6	-224
039A	27	9.9	88	7	3.4	14	-20	-6.5	-75
040A	42	16.3	262	7	1.5	8	-35	-14.8	-253
041A	53	191.9	591	3	20.5	13	-50	-171.4	-578
042A	19	11.5	54	7	5.3	14	-12	-6.2	-41
043A	3	0.2	1	0	0.0	0	-3	-0.2	-1
Total	**-**	**1068.5**	**-**	**-**	**201.0**	**-**	**-**	**-867.5**	**-**

Table 7-34: Control Program 4 - Satellite Storage, Performance Summary – 12 Overflows

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
001A	42	86.3	432	9	26.2	31	-33	-60.1	-401
002A	35	32.4	224	7	10.2	20	-28	-22.1	-203
003A	43	60.7	285	7	21.9	20	-36	-38.8	-265
005A	54	96.6	593	2	6.0	14	-52	-90.6	-578
008A	36	9.6	302	12	3.2	35	-24	-6.4	-267
010A	42	17.2	271	8	5.3	24	-34	-11.9	-248
012A	44	5.8	355	8	0.6	21	-36	-5.3	-335
013A	42	16.9	313	8	5.2	23	-34	-11.7	-290
014A	13	1.1	16	6	0.5	6	-7	-0.5	-10
016A	46	16.7	367	12	4.7	32	-34	-12.0	-335
021A	19	1.4	32	6	0.7	9	-13	-0.8	-23
022A	46	71.3	591	11	15.0	61	-35	-56.3	-531
026A	53	53.2	613	12	12.9	45	-41	-40.3	-568
027A	25	27.7	378	8	9.2	45	-17	-18.5	-333
028A	35	35.4	514	7	9.2	37	-28	-26.2	-476
029A	39	44.7	474	7	7.6	26	-32	-37.1	-448
030A	11	2.2	19	6	0.9	3	-5	-1.3	-16
031A	35	15.4	266	7	3.7	16	-28	-11.7	-250
032A	26	7.4	83	7	2.0	8	-19	-5.4	-75
034A	44	77.7	404	11	20.0	28	-33	-57.7	-376
035A	35	42.6	307	8	7.6	19	-27	-35.0	-288
036A	30	43.6	240	7	15.9	20	-23	-27.7	-220
037A	44	64.6	463	8	8.4	23	-36	-56.2	-440
038A	30	8.6	224	0	0.0	0	-30	-8.6	-224
039A	27	9.9	88	7	3.4	14	-20	-6.5	-75
040A	42	16.3	262	7	1.5	8	-35	-14.8	-253
041A	53	191.9	591	10	38.6	48	-43	-153.2	-543
042A	19	11.5	54	7	5.3	14	-12	-6.2	-41
043A	3	0.2	1	0	0.0	0	-3	-0.2	-1
Total	**-**	**1068.5**	**-**	**-**	**245.6**	**-**	**-**	**-822.9**	**-**

Table 7-35: Control Program 4 - Satellite Storage, Performance Summary – 20 Overflows

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
001A	42	86.3	432	18	42.4	56	-24	-43.9	-376
002A	35	32.4	224	16	18.9	41	-19	-13.4	-183
003A	43	60.7	285	16	43.7	41	-27	-17.1	-244
005A	54	96.6	593	2	6.0	14	-52	-90.6	-578
008A	36	9.6	302	17	4.5	47	-19	-5.1	-254
010A	42	17.2	271	17	9.5	43	-25	-7.7	-229
012A	44	5.8	355	14	1.2	42	-30	-4.6	-313
013A	42	16.9	313	16	8.6	42	-26	-8.2	-271
014A	13	1.1	16	9	0.8	10	-4	-0.2	-6
016A	46	16.7	367	18	7.9	54	-28	-8.8	-313
021A	19	1.4	32	10	1.0	14	-9	-0.4	-18
022A	46	71.3	591	18	23.1	92	-28	-48.2	-500
026A	53	53.2	613	19	21.1	75	-34	-32.1	-538
027A	25	27.7	378	13	15.4	92	-12	-12.3	-286
028A	35	35.4	514	18	18.7	82	-17	-16.7	-432
029A	39	44.7	474	14	12.5	43	-25	-32.1	-431
030A	11	2.2	19	10	1.9	6	-1	-0.3	-13
031A	35	15.4	266	14	7.0	27	-21	-8.3	-239
032A	26	7.4	83	13	4.8	18	-13	-2.6	-65
034A	44	77.7	404	16	29.6	44	-28	-48.1	-360
035A	35	42.6	307	11	9.4	24	-24	-33.2	-283
036A	30	43.6	240	16	28.7	36	-14	-14.8	-204
037A	44	64.6	463	16	12.2	36	-28	-52.4	-427
038A	30	8.6	224	0	0.0	0	-30	-8.6	-224
039A	27	9.9	88	13	6.6	26	-14	-3.3	-62
040A	42	16.3	262	12	3.0	17	-30	-13.3	-245
041A	53	191.9	591	14	58.5	75	-39	-133.4	-516
042A	19	11.5	54	12	10.5	26	-7	-1.1	-29
043A	3	0.2	1	0	0.0	0	-3	-0.2	-1
Total	**-**	**1068.5**	**-**	**-**	**407.4**	**-**	**-**	**-661.1**	**-**

7.4.7 Cost Summary

The Class 5 (+100%, -50%) cost estimate for Control Program 4 is summarized in Table 7-36.

Table 7-36: Control Program 4 – Satellite Storage, Cost Summary

Control Level, Equivalent to Noted Overflows per Year	0	4	8	12	20
Satellite Storage Tanks ($ Million)	$832.0	$450.4	$342.2	$309.0	$203.7
Treatment Plant Facility($ Million)	$16.3	$16.3	$16.3	$16.3	$16.3
TAPS Upgrade (65 mgd) ($ Million)	$7.2	$7.2	$7.2	$7.2	$7.2
Total Construction Costs ($ Million)	*$855.6*	*$473.9*	*$365.7*	*$332.6*	*$227.2*
Land Acquisition Costs ($ Million)	$88.7	$40.2	$28.3	$24.3	$15.0
Non-Construction Costs ($ Million)	$231.0	$124.0	$91.0	$83.0	$55.0
Total Capital Cost ($ Million)	**$1,175.3**	**$638.1**	**$485.0**	**$439.9**	**$297.2**
Annual O&M Cost ($ Million)	$8.6	$4.7	$3.7	$3.3	$2.3

Control Level, Equivalent to Noted Overflows per Year	0	4	8	12	20
Total Present Worth ($ Million)	**$1,306.0**	**$709.5**	**$541.3**	**$490.0**	**$332.2**
CSO Volume Abated (MG)	1069	960	867	823	661
Cost per Gallon Abated ($/gal)	$1.22	$0.74	$0.62	$0.60	$0.50

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7.5 Control Program 5: Tunnel Storage and Secondary Controls

7.5.1 Description

This Control Program includes a tunnel of approximately 19,800 feet in length, with one segment extending along the southern waterfront of the City and the second segment along the west side of the Elizabeth River. This deep tunnel storage will service 26 CSO outfalls. The tunnel would be constructed in rock at a dept of the approximately 120 feet, with 8 vertical shafts (7 consolidation drop shafts and 1 work shaft/dewatering pump station shaft). The tunnel would be dewatered and discharge to the JMEUC WWTF and would include an overflow to the river. Overall, the control program would consist of:

- Consolidation piping and drop shafts for 7 outfall groups
- Satellite storage for Outfalls 001A and 002A
- Sewer separation for Outfall 037A
- Tunnel dewatering pump station
- Expanded wet weather treatment
- Increased pumping from Trenton Avenue PS

A conceptual layout is shown in Figure 7-27. It is noted that the layout and feasibility of tunnels is highly dependent on geotechnical conditions. For the purpose of this analysis, it has been assumed that the tunnel will be constructed in rock, which is the most favorable condition. Variations in soil type may increase both risk and expense.

7.5.2 Analysis

Initially, three tunnel segments were investigated – the southern segment along the waterfront, the western segment, and the eastern segment. It was determined that because the eastern segment would only be servicing outfalls 001A and 002A, at a significant length and associated high cost, that the alternative would proceed with just the western and southern routing. These two segments service all of the outfalls except 001A, 002A and 037A. Outfalls 001A and 002A would be serviced with satellite storage tankage, and the drainage area to Outfall 037A would undergo sewer separation.

The total length of the proposed sewer segments is 19,800 feet. The tunnel was input into the InfoWorksICM model. Initial tunnel diameters were developed for the range of control levels based on the required volume from the model. Since InfoWorksICM dynamically tracks storage, the tunnel volumes could be modeled explicitly using conduits in the model.

In this alternative, the majority of tunnel infrastructure would be located below grade, however land acquisition would be required for siting of launch and drop shafts during construction. Land would also be required for siting the dewatering pump station. This alternative requires much less land acquisition than some of the alternatives such as satellite storage and satellite treatment.

The area required for the range of control alternatives is presented in Table 7-37 below.

Table 7-37: Land Required for Range of Control Levels

Control Level Overflows per year	0	4	8	12	20
Deep Tunnel Storage (acres)	4.50	4.50	4.50	4.50	4.50
Outfall 001A Tank (acres)	2.55	1.01	0.48	0.44	0.21
Outfall 002A Tank (acres)	0.96	0.40	0.25	0.25	0.10
Total Land Required (acres)	**8.01**	**5.91**	**5.23**	**5.19**	**4.81**

Figure 7-27: Tunnel Storage Conceptual Layout

7.5.3 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
- USACE Nationwide 404 Permit
- Local Permits
- NJDEP Treatment Works Approval
- Permits and coordination with railroads and State DOT
- Soil Erosion and Sediment Control 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.

7.5.4 Implementability

Implementing a tunnel within the confines of a dense urban area is challenging. Mining and recovery shaft areas are required for this alternative to be feasible, and available area in Elizabeth for this purpose is minimal. This alternative also requires area to site a dewatering pumping station and a tunnel overflow, and available area in this highly urbanized town is limited. While it is possible to control the flow into the tunnel through the use of automated gates and level sensors, the tunnel must still be provided with a relief point.

Based on available geotechnical information, it has been assumed that the tunnel would need to be constructed about 120 feet below grade in order to be constructed in rock, which is the preferred TBM tunnel excavation material. The soil type would need to be confirmed prior to construction, and any variations in ground material could increase the costs and could carry a greater risk of subsidence due to soil loss, potentially damaging nearby buildings and other surface infrastructure. The construction of de-aeration chambers at tunnel level may be further complicated by soft ground conditions.

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.

The routing of the tunnels would be maintained within existing public right-of-way, but deviations may be necessary given typical turning radius capabilities for boring machines. Where the tunnel alignment passes underneath private properties, negotiations with the property owners would be required to obtain easement rights.

7.5.5 Public Acceptance

The construction required for tunnels is large and invasive making public acceptance of the project a concern. The proposed tunnel shaft sites are located on underdeveloped parcels of land, and there may be concerns related to heavy mechanical facilities in areas that are in close proximity to residential development. Shaft sites located in industrial areas may raise fewer concerns from the public.

Following construction, tunnels are generally preferable from the stand point of public acceptance since the majority of the facility is underground. Aboveground features will still be required such air release, electrical facilities, odor control facilities and access points to pumps.

7.5.6 Performance Summary

The sizing of the tunnels is based on increased CSO conveyance and treatment, with the pumping capacity at the existing TAPS taken to be upgraded to 65 MGD. It should also be noted that the stored flow will be dewatered to the JMEUC WWTF through a separate CSO treatment train. Given this increased conveyance, the cost analysis for this control program includes capital and operating costs for a CSO wet weather treatment facility at the JMEUC WWTF. As such, elements of Control Program 3 are incorporated into the performance of this control program. This represents a significant volume of additional flow to be treated annually at the WWTF, and the associated O&M costs have not yet been estimated.

The tunnel storage volumes and corresponding diameters for the range of control levels is presented in Table 7-38.

Table 7-38: Program Storage Volumes for Range of Control Levels

Control Level Overflows per year	0	4	8	12	20
Deep Tunnel Storage	78.8	37.8	22.8	19.7	9.4
Outfall 001A Tank	12.5	4.93	2.35	2.15	1.03
Outfall 002A Tank	4.67	1.96	1.21	1.21	0.50
Total Storage Volume (MG)	**95.9**	**44.7**	**26.4**	**23.1**	**10.9**
Tunnel Diameter (ft)	26	18	14	11	9
Tunnel Dewatering Pumping Rate (mgd)	35	25	15	11	9

The performance of under this control program is summarized in Table 7-39 through Table 7-43 which present the results for the equivalent treatment for 0, 4, 8, 12, and 20 overflows per year.

Table 7-39: Control Program 5 - Tunnel Storage, Performance Summary – 0 Overflows

	Baseline 2015			Control Program			Change		
Outfall No.	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
001A	42	86.3	432	0	0.0	0	-42	-86.3	-432
002A	35	32.4	224	0	0.0	0	-35	-32.4	-224
037A	44	64.6	463	0	0.0	0	-44	-64.6	-463
Tunnel Outfall	54	885.3	613	0	0.0	0	-54	-885.3	-613
Total	**-**	**1068.5**	**-**	**0**	**0.0**	**0**	**-**	**-1068.5**	**-**

Table 7-40: Control Program 5 - Tunnel Storage, Performance Summary – 4 Overflows

	Baseline 2015			Control Program			Change		
Outfall No.	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
001A	42	86.3	432	2	8.2	11	-40	-78.2	-421
002A	35	32.4	224	5	3.0	17	-30	-29.3	-206
037A	44	64.6	463	0	0.0	0	-44	-64.6	-463
Tunnel Outfall	54	885.3	613	2	52.3	9	-52	-833.0	-604

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
Total	**-**	**1068.5**	**-**	**-**	**63.5**	**-**	**-**	**-1005.0**	**-**

Table 7-41: Control Program 5 - Tunnel Storage, Performance Summary – 8 Overflows

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
001A	42	86.3	432	8	23.7	31	-34	-62.7	-401
002A	35	32.4	224	8	8.1	24	-27	-24.3	-199
037A	44	64.6	463	0	0.0	0	-44	-64.6	-463
Tunnel Outfall	54.0	885.3	613	7	131.5	19	-47	-753.8	-594
Total	**-**	**1068.5**	**-**	**-**	**163.2**	**-**	**-**	**-905.3**	**-**

Table 7-42: Control Program 5 - Tunnel Storage, Performance Summary – 12 Overflows

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
001A	42	86.3	432	12	26.1	37	-30	-60.2	-395
002A	35	32.4	224	11	13.6	35	-24	-18.8	-189
037A	44	64.6	463	0	0.0	0	-44	-64.6	-463
Tunnel Outfall	54.0	885.3	613	7	184.0	24	-47	-701.3	-589
Total	**-**	**1068.5**	**-**	**-**	**223.7**	**-**	**-**	**-844.8**	**-**

Table 7-43: Control Program 5 - Tunnel Storage, Performance Summary – 20 Overflows

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
001A	42	86.3	432	17	42.4	68	-25	-44.0	-365
002A	35	32.4	224	15	15.6	43	-20	-16.8	-181
037A	44	64.6	463				-44	-64.6	-463
Tunnel Outfall	54	885.3	613	14	275.4	50	-40	-609.8	-562
Total	**-**	**1068.5**	**-**	**-**	**333.4**	**-**	**-**	**-735.1**	**-**

7.5.7 Cost Summary

The Class 5 (+100%, -50%) cost estimate for Control Program 5 for the range of control levels is summarized in Table 7-44, including the cost per gallon treated. As this control program includes additional CSO conveyance and treatment, a conservative capital cost for a wet weather treatment facility is listed with this estimate. The cost for the CSO treatment train at the JMEUC WWTF would be refined during plan selection.

Table 7-44: Control Program 5 – Tunnel Storage, Cost Summary

Control Level, Equivalent to Noted Overflows per Year	0	4	8	12	20
Deep Tunnel Storage ($ Million)	$546.0	$433.0	$367.0	$326.0	$288.0
Treatment Plant Facility ($ Million)	$16.3	$16.3	$16.3	$16.3	$16.3
TAPS Upgrade ($ Million)	$7.2	$7.2	$7.2	$7.2	$7.2
Storage Tank Outfall 001A ($ Million)	$69.9	$31.5	$17.3	$16.2	$9.7
Storage Tank Outfall 002A ($ Million)	$30.1	$15.0	$10.8	$10.8	$5.3
Basin 037 Separation ($ Million)	$24.4	$24.4	$24.4	$24.4	$24.4
Total Construction Cost ($ Million)	*$694.0*	*$527.0*	*$443.0*	*$401.0*	*$351.0*
Land Acquisition Costs ($ Million)	$27.9	$20.6	$18.2	$18.1	$16.8
Non-Construction Costs ($ Million)	$180.0	$137.0	$115.0	$105.0	$92.0
Total Capital Cost ($ Million)	***$901.9***	***$684.6***	***$576.2***	***$524.1***	***$459.8***
Annual O&M Costs ($ Million)	*$4.0*	*$3.0*	*$2.4*	*$2.2*	*$1.9*
Total Present Value ($ Million)	**$962.9**	**$730.6**	**$613.2**	**$558.1**	**$488.8**
Overflow Volume Cap2tured (MG)	1069	1005	905	845	735
Cost per Gallon Treated ($/gal)	$0.90	$0.73	$0.68	$0.66	$0.66

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7.6 Control Program 6: Green Infrastructure

7.6.1 Description

This control program consists of 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 of 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 refinement of types and specific locations of GI will be determined in future planning stages.

7.6.2 Analysis

For purposes of evaluation, directing 2.5%, 5%, 7.5%, 10%, and 15% of the impervious area within the combined sewer area to green stormwater infrastructure was evaluated. This approach was taken rather than evaluating control levels based on the number of overflows because green infrastructure has a minimal impact on overflow volume reduction when compared to all of the other control program alternatives. The range of 2.5% to 15% impervious area capture was selected in order to evaluate the marginal CSO volume reduction produced by different levels of GI implementation, as well as reflect the range of implementation that has also been selected for analysis by the NJ CSO Group. It is noted however that evaluating fixed amounts of impervious area diverted to green stormwater infrastructure ignores whether such an approach is practical or technically feasible.

Using the guidance documents previously discussed, an attempt was made to determine the maximum amount of impervious area that could practically be directed from impervious areas to green infrastructure. It is noted that experience from New York City has shown that the vast majority of sites identified through a desktop GIS study are deemed unsuitable once field investigations and geotechnical (infiltration) testing are conducted. For example, an analysis conducted on sites in one New York City basin showed that of the sites identified at the planning level, only 17% were found suitable to proceed to construction.

The available data on soils and groundwater levels in Elizabeth indicate that the majority of the City is classified as “urban land” as such the infiltration potential of the soil is not defined. Field studies have also been inconclusive regarding the infiltration potential of Elizabeth’s soil. As such, this planning-level assessment did not consider limitations of soil properties on areas suitable for GI. The sites’ soil properties will be considered at a later stage of the process, when individual sites are under consideration. For the purpose of this assessment, bioswales were conservatively 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 7-28), areas of impervious cover (Figure 7-29), publicly owned land (Figure 7-30), and soil infiltration potential (Figure 7-31).

As discussed in Section 5.2.5, the public right-of-way likely 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 in order to install green stormwater infrastructure. Accordingly, it was assumed that only one bioswale could be installed on each side of the street segment (see Figure 7-32). As such, it was assumed that a typical street segment would have two bioswales, one on each side.

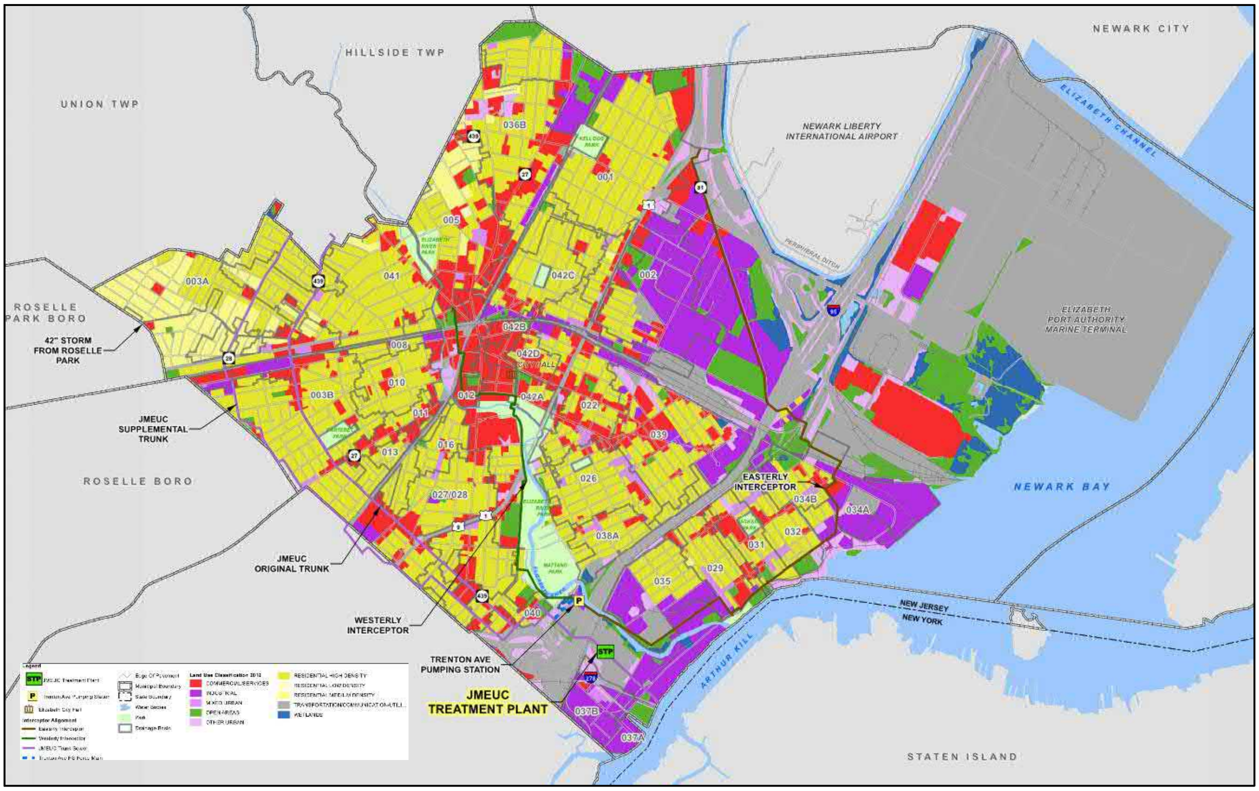

Figure 7-28: Elizabeth Land Use Map

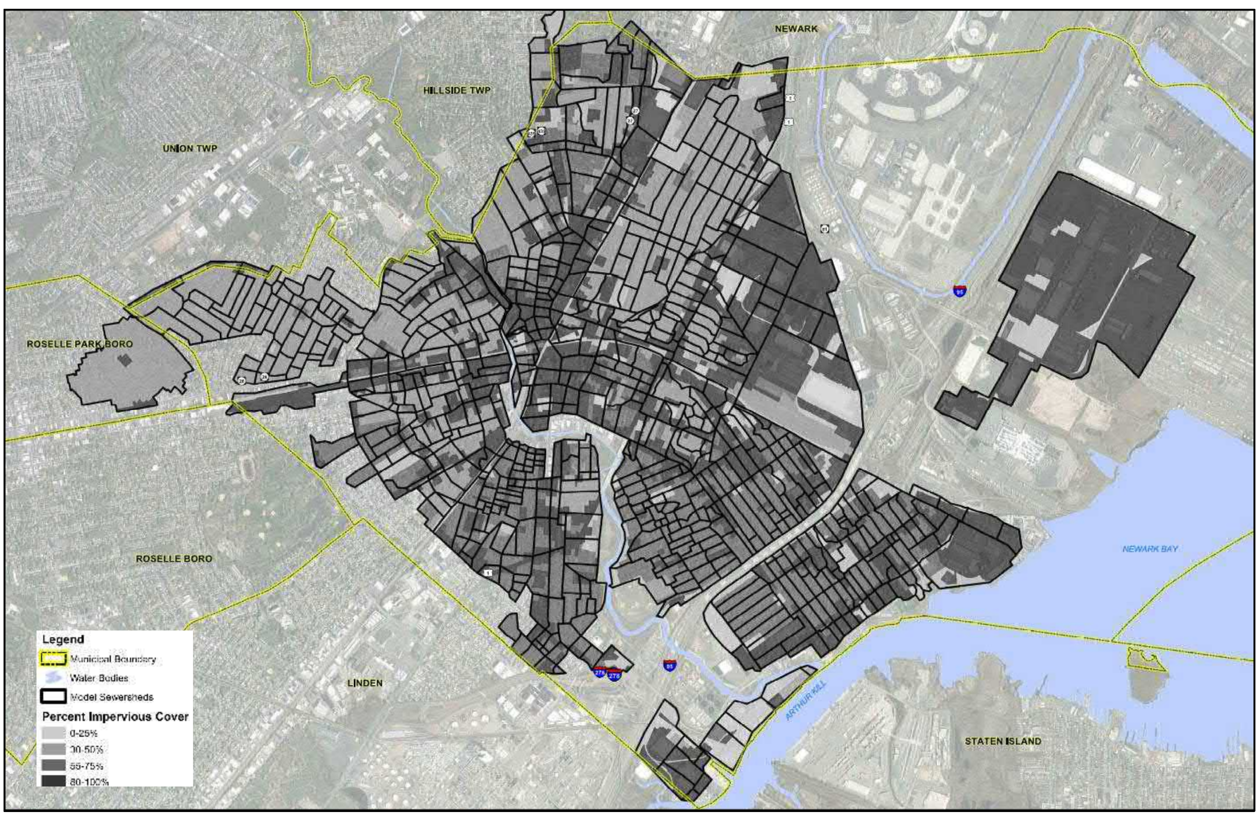

Figure 7-29: Elizabeth Impervious Cover Map

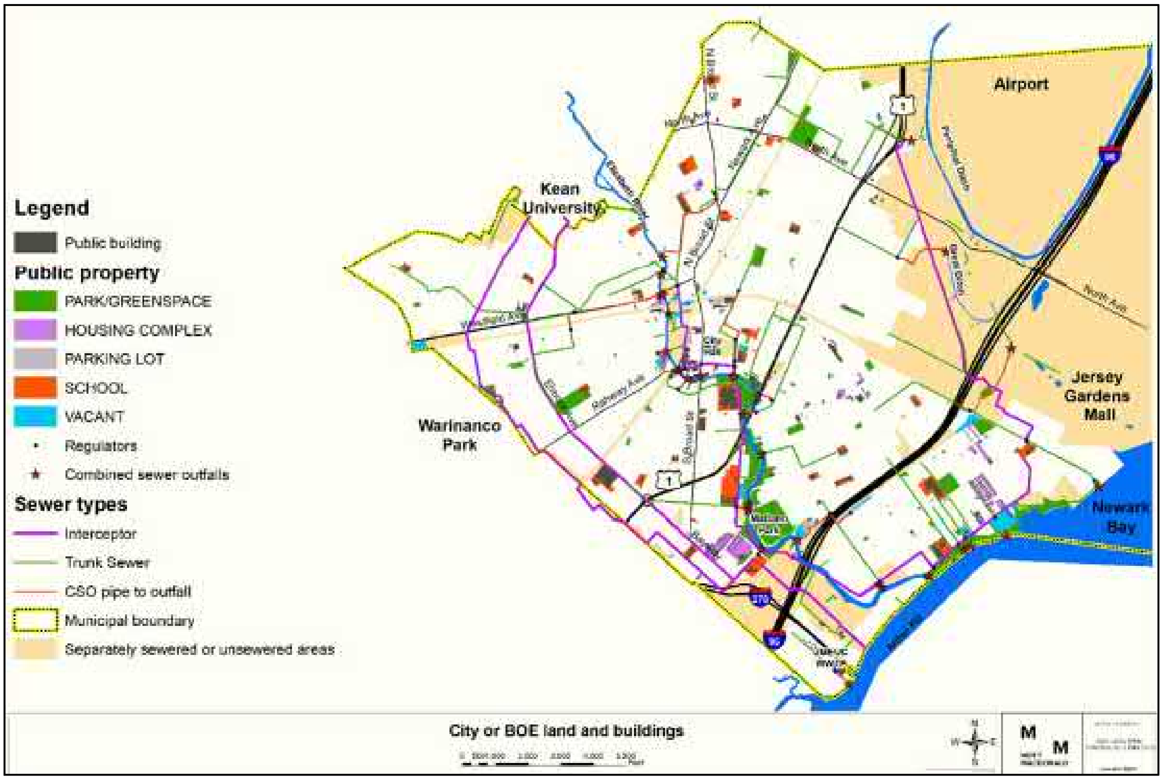

Figure 7-30: Elizabeth Publicly Owned Land

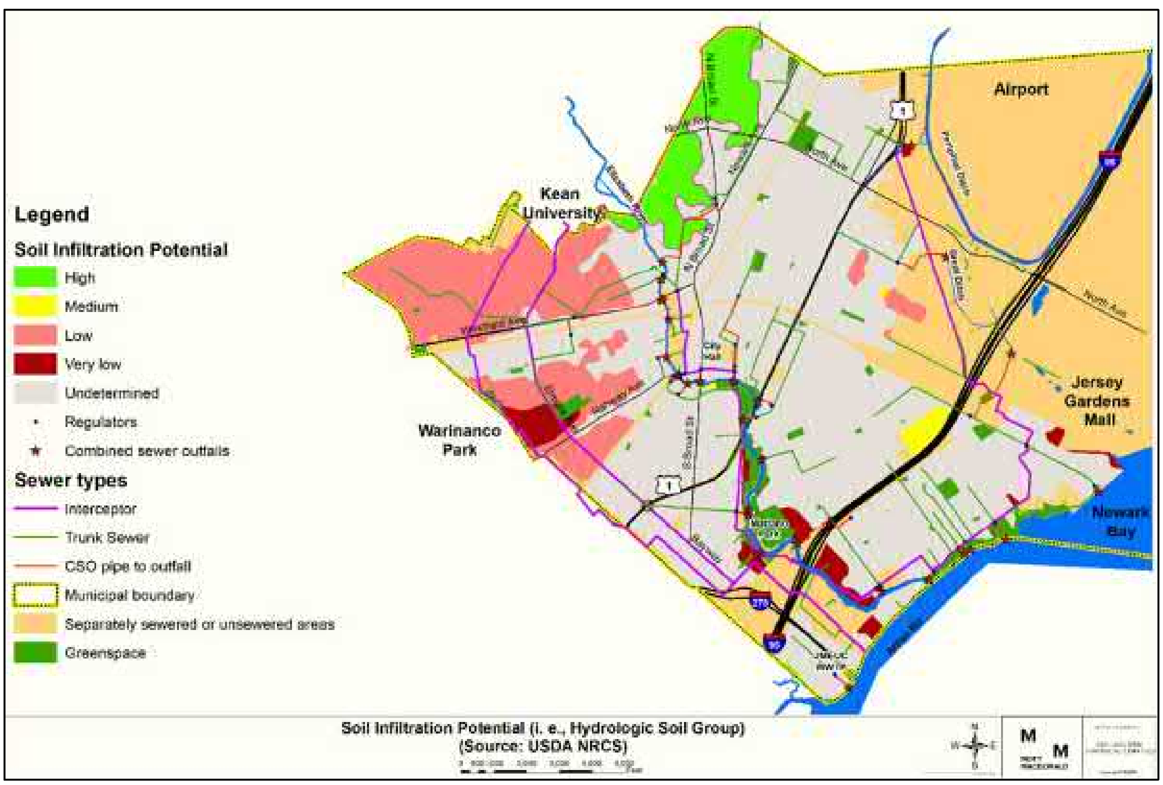

Figure 7-31: Elizabeth Soil Infiltration Potential

Figure 7-32: Typical street segment with green stormwater infrastructure

The typical bioswale was considered to 20 feet by 3 feet and using a 15:1 loading ratio, it would treat 900 square feet (sf) of impervious area. Through the GIS analysis, it was estimated that the City has approximately 1,740 street segments, which result in 3,500 bioswales. Conservatively, applying a planning level installation rate of 25% (versus 17% from New York City) results in 875 bioswales with a treatment area of 787,500 sf, or 18.1 acres, of impervious area treated.

The other considered green stormwater infrastructure practice is permeable pavement. The recommended practice is to apply the permeable paving to parking lanes. A typical street segment in Elizabeth is approximately 390 feet long. It is assumed that the last 50 feet at either end of the block would be reserved for turning lanes, resulting in 290 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 3480 square feet (sf) per block. Given the groundwater and soil conditions, it was assumed that only 10% of the City is suitable for installation of permeable pavement resulting in 348 sf per street segment. This results in a maximum of 609,000 sf of permeable paving in the City. Applying the recommended loading ratio of 4:1 results in a total of 2,436,000 sf or approximately 56 acres of impervious area to be treated.

With both bioswales and permeable paving, a total of approximately 74 acres of impervious area out of a total of 2542 acres of impervious area in the existing combined area, or a maximum of 2.9% of the total impervious area, could be directed to green stormwater infrastructure.

Bioswales were selected as the representative type of GI to be modelled in InfoWorksICM, with site-specific GI types to be selected later in the planning process. The equivalent GI area as bioswales was modelled in the InfoWorksICM as a representative 20'x3' unit with 18" soil depth and 3.5' storage layer. This was input in the InfoWorksICM Sustainable Urban Drainage Systems (SUDS) module (Figure 7-33) 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 7-34) 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.

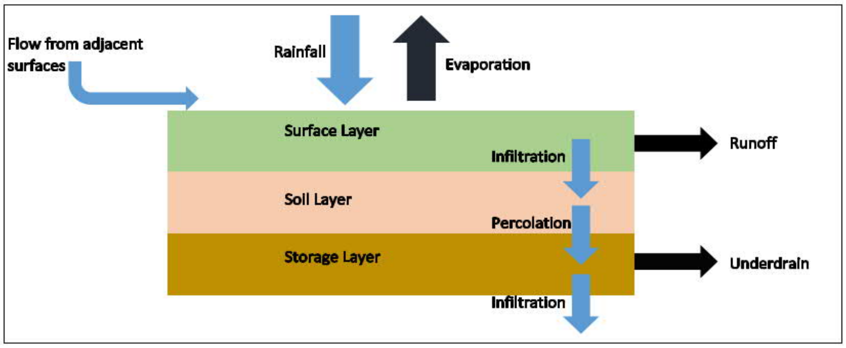

Figure 7-33: InfoWorks SUDS diagram

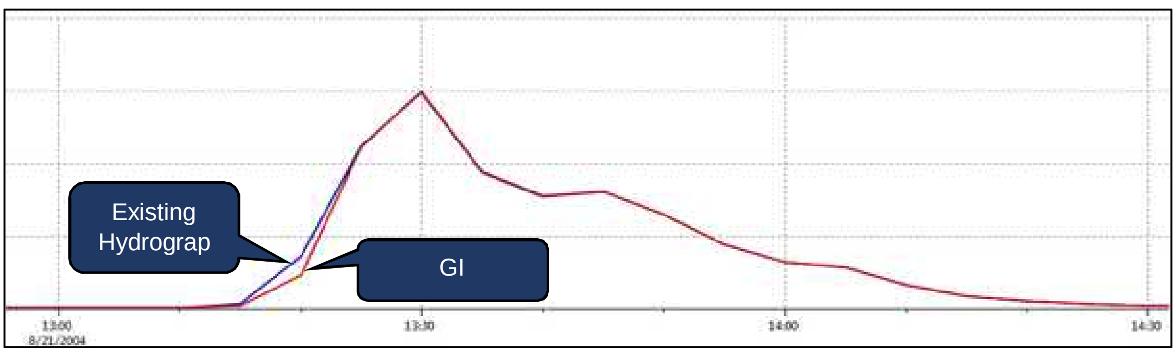

Figure 7-34: Representative green infrastructure hydrograph

7.6.3 Institutional Issues

Typically, the institutional issues associated with green stormwater infrastructure are minimal. Their construction would generally fall within the overall goals of the City'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 City
- NJDEP Treatment Works Approval
- Soil Erosion and Sediment Control Approval

Additional permits and coordination may be required if green stormwater infrastructure is implemented on State or County property.

7.6.4 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 City. Accordingly, no land acquisition would be required. However, there are other implementation challenges associated with green stormwater infrastructure to be considered. As has been experienced by other entities such as New York City, there are numerous 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. This high level of attrition has been reflected in the estimate of green stormwater infrastructure proposed in an effort to realistically reflect these siting and construction implementation constraints.

7.6.5 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 relies heavily 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 is considered to be high.

7.6.6 Performance Summary

The performance of Control Program 6 is summarized in Table 7-45 through Table 7-49 for 2.5%, 5%, 7.5%, 10%, and 15% directly connected impervious area. The performance under the control program includes the future baseline increased base sanitary flows and the impacts of current and planned projects, in addition to the green infrastructure control measures. As noted previously, it is estimated that 5% of the directly connected impervious area is the upper bound of what could reasonably be directed to green infrastructure.

The system-wide annual average CSO volume reduction predicted with green stormwater infrastructure for 2.5% impervious area managed is 16.2 million gallons per year (MG/year) compared against the existing conditions baseline. This value increases nominally to 22.6, 26.6, 31.3, and 36.0 MG/year for 5%, 7.5%, 10%, and 15% impervious area managed, respectively. The low CSO volume reductions reflect the poor infiltration associated with the general soil conditions within the City.

Table 7-45: Control Program 6 - Green Stormwater Infrastructure, Performance Summary, Treatment of 2.5% Impervious

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
001A	42	86.3	432	43	87.3	437	1	0.9	4
002A	35	32.4	224	35	32.4	224	0	0.1	0
003A	43	60.7	285	42	60.9	285	-1	0.2	0
005A	54	96.6	593	54	94.1	601	0	-2.5	8
008A	36	9.6	302	38	9.9	303	2	0.3	1
010A	42	17.2	271	41	17.2	284	-1	0.0	13
012A	44	5.8	355	43	6.0	383	-1	0.2	28
013A	42	16.9	313	42	16.8	330	0	0.0	17

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
014A	13	1.1	16	13	1.1	16	0	0.0	0
016A	46	16.7	367	45	15.9	346	-1	-0.8	-21
021A	19	1.4	32	16	1.1	31	-3	-0.3	-1
022A	46	71.3	591	45	67.1	556	-1	-4.2	-35
026A	53	53.2	613	53	51.3	627	0	-1.8	14
027A	25	27.7	378	25	28.0	316	0	0.3	-62
028A	35	35.4	514	37	35.8	486	2	0.4	-27
029A	39	44.7	474	39	45.5	474	0	0.8	0
030A	11	2.2	19	11	2.2	19	0	0.0	0
031A	35	15.4	266	35	15.5	266	0	0.1	0
032A	26	7.4	83	26	7.3	83	0	0.0	0
034A	44	77.7	404	44	76.5	427	0	-1.2	23
035A	35	42.6	307	33	40.7	265	-2	-1.9	-42
036A	30	43.6	240	30	43.8	241	0	0.3	1
037A	44	64.6	463	44	66.9	494	0	2.3	31
038A	30	8.6	224	3	0.0	7	-27	-8.6	-217
039A	27	9.9	88	27	9.1	88	0	-0.8	-1
040A	42	16.3	262	44	16.7	302	2	0.4	40
041A	53	191.9	591	53	191.5	600	0	-0.4	9
042A	19	11.5	54	19	11.6	49	0	0.0	-5
043A	3	0.2	1	3	0.0	1	0	-0.1	-1
Total	**-**	**1068.5**	**-**	**-**	**1052.3**	**-**	**-**	**-16.2**	**-**

Table 7-46: Control Program 6 - Green Stormwater Infrastructure, Performance Summary, Treatment of 5% Impervious

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
001A	42	86.3	432	43	85.6	436	1	-0.8	4
002A	35	32.4	224	35	32.4	224	0	0.1	0
003A	43	60.7	285	39	60.8	271	-4	0.0	-14
005A	54	96.6	593	54	92.4	600	0	-4.2	7
008A	36	9.6	302	38	9.9	303	2	0.3	1
010A	42	17.2	271	41	17.1	283	-1	-0.1	12
012A	44	5.8	355	43	6.0	383	-1	0.2	28
013A	42	16.9	313	42	16.8	330	0	0.0	17
014A	13	1.1	16	12	1.1	16	-1	0.0	0
016A	46	16.7	367	45	15.9	346	-1	-0.8	-21
021A	19	1.4	32	16	1.1	31	-3	-0.3	-1
022A	46	71.3	591	45	67.0	556	-1	-4.3	-35
026A	53	53.2	613	53	51.1	627	0	-2.1	14
027A	25	27.7	378	25	27.9	316	0	0.2	-62
028A	35	35.4	514	37	35.7	486	2	0.3	-27
029A	39	44.7	474	39	45.4	449	0	0.8	-25
030A	11	2.2	19	11	2.2	19	0	0.0	0
031A	35	15.4	266	35	15.4	266	0	0.1	0
032A	26	7.4	83	26	7.3	83	0	-0.1	0
034A	44	77.7	404	44	76.3	427	0	-1.4	23
035A	35	42.6	307	33	40.7	265	-2	-1.9	-42

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
036A	30	43.6	240	30	43.6	236	0	0.0	-3
037A	44	64.6	463	44	66.8	489	0	2.2	26
038A	30	8.6	224	3	0.0	7	-27	-8.6	-217
039A	27	9.9	88	27	9.0	88	0	-0.9	-1
040A	42	16.3	262	45	16.7	302	3	0.4	41
041A	53	191.9	591	53	190.1	604	0	-1.8	12
042A	19	11.5	54	18	11.5	49	-1	0.0	-5
043A	3	0.2	1	3	0.0	1	0	-0.1	-1
Total	**-**	**1068.5**	**-**	**-**	**1045.9**	**-**	**-**	**-22.6**	**-**

Table 7-47: Control Program 6 - Green Stormwater Infrastructure, Performance Summary, Treatment of 7.5% Impervious

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
001A	42	86.3	432	43	85.0	436	1	-1.3	4
002A	35	32.4	224	35	32.3	223	0	-0.1	-1
003A	43	60.7	285	38	60.6	270	-5	-0.1	-14
005A	54	96.6	593	54	92.4	567	0	-4.2	-25
008A	36	9.6	302	37	9.9	302	1	0.2	1
010A	42	17.2	271	38	17.0	282	-4	-0.2	11
012A	44	5.8	355	42	6.0	382	-2	0.1	26
013A	42	16.9	313	41	16.7	313	-1	-0.2	0
014A	13	1.1	16	12	1.1	16	-1	0.0	0
016A	46	16.7	367	43	15.7	328	-3	-1.0	-39
021A	19	1.4	32	16	1.1	31	-3	-0.3	-1
022A	46	71.3	591	45	66.4	515	-1	-4.9	-76
026A	53	53.2	613	54	50.7	626	1	-2.4	13
027A	25	27.7	378	25	27.8	315	0	0.1	-63
028A	35	35.4	514	37	35.5	511	2	0.1	-3
029A	39	44.7	474	38	45.3	441	-1	0.7	-32
030A	11	2.2	19	11	2.2	19	0	0.0	0
031A	35	15.4	266	35	15.4	266	0	0.0	0
032A	26	7.4	83	26	7.3	83	0	-0.1	0
034A	44	77.7	404	43	75.9	425	-1	-1.8	21
035A	35	42.6	307	33	40.6	264	-2	-2.0	-43
036A	30	43.6	240	30	43.4	233	0	-0.2	-7
037A	44	64.6	463	44	66.5	484	0	2.0	21
038A	30	8.6	224	3	0.0	7	-27	-8.6	-217
039A	27	9.9	88	26	9.0	87	-1	-0.9	-1
040A	42	16.3	262	45	16.6	302	3	0.4	40
041A	53	191.9	591	49	190.1	568	-4	-1.8	-24
042A	19	11.5	54	18	11.5	49	-1	0.0	-5
043A	3	0.2	1	3	0.0	1	0	-0.1	-1
Total	**-**	**1068.5**	**-**	**-**	**1041.9**	**-**	**-**	**-26.6**	**-**

Table 7-48: Control Program 6 - Green Stormwater Infrastructure, Performance Summary, Treatment of 10% Impervious

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
001A	42	86.3	432	43	85.6	436	1	-0.8	4
002A	35	32.4	224	35	32.3	223	0	0.0	0
003A	43	60.7	285	38	60.5	270	-5	-0.3	-15
005A	54	96.6	593	54	91.2	599	0	-5.4	6
008A	36	9.6	302	38	9.8	273	2	0.2	-29
010A	42	17.2	271	38	16.9	282	-4	-0.3	10
012A	44	5.8	355	43	6.0	384	-1	0.1	29
013A	42	16.9	313	42	16.7	329	0	-0.2	16
014A	13	1.1	16	12	1.1	16	-1	0.0	0
016A	46	16.7	367	42	16.0	331	-4	-0.7	-36
021A	19	1.4	32	16	1.1	31	-3	-0.3	-1
022A	46	71.3	591	45	66.2	555	-1	-5.1	-36
026A	53	53.2	613	54	50.4	627	1	-2.7	14
027A	25	27.7	378	25	27.7	311	0	0.0	-67
028A	35	35.4	514	37	35.4	510	2	0.0	-4
029A	39	44.7	474	37	45.2	449	-2	0.5	-25
030A	11	2.2	19	11	2.2	19	0	0.0	0
031A	35	15.4	266	35	15.3	265	0	0.0	-1
032A	26	7.4	83	26	7.2	82	0	-0.1	0
034A	44	77.7	404	44	75.4	425	0	-2.2	21
035A	35	42.6	307	33	40.5	264	-2	-2.1	-42
036A	30	43.6	240	29	43.2	233	-1	-0.4	-7
037A	44	64.6	463	44	66.3	492	0	1.8	30
038A	30	8.6	224	3	0.0	7	-27	-8.6	-217
039A	27	9.9	88	26	8.9	87	-1	-1.0	-1
040A	42	16.3	262	44	16.6	301	2	0.4	40
041A	53	191.9	591	53	188.0	603	0	-3.9	12
042A	19	11.5	54	18	11.4	49	-1	-0.1	-6
043A	3	0.2	1	3	0.0	1	0	-0.1	-1
Total	**-**	**1068.5**	**-**	**-**	**1037.2**	**-**	**-**	**-31.3**	**-**

Table 7-49: Control Program 6 - Green Stormwater Infrastructure, Performance Summary, Treatment of 15% Impervious

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
001A	42	86.3	432	42	85.1	436	0	-1.2	4
002A	35	32.4	224	34	32.2	223	-1	-0.1	-1
003A	43	60.7	285	37	60.2	256	-6	-0.6	-29
005A	54	96.6	593	52	92.4	561	-2	-4.2	-32
008A	36	9.6	302	38	9.7	269	2	0.1	-33
010A	42	17.2	271	38	16.7	265	-4	-0.5	-6
012A	44	5.8	355	42	5.9	380	-2	0.0	24
013A	42	16.9	313	41	16.4	311	-1	-0.5	-2
014A	13	1.1	16	12	1.0	16	-1	0.0	0
016A	46	16.7	367	39	15.8	328	-7	-0.9	-39
021A	19	1.4	32	16	1.1	30	-3	-0.3	-2

Outfall No.	Baseline 2015			Control Program			Change		
	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)	No. of Events	Volume (MG)	Duration (hours)
022A	46	71.3	591	44	65.3	553	-2	-6.0	-38
026A	53	53.2	613	54	49.8	610	1	-3.4	-3
027A	25	27.7	378	24	27.5	272	-1	-0.2	-106
028A	35	35.4	514	36	35.1	457	1	-0.3	-56
029A	39	44.7	474	37	45.0	466	-2	0.3	-8
030A	11	2.2	19	11	2.2	19	0	0.0	0
031A	35	15.4	266	35	15.2	230	0	-0.1	-36
032A	26	7.4	83	26	7.2	82	0	-0.2	0
034A	44	77.7	404	43	74.7	394	-1	-3.0	-10
035A	35	42.6	307	33	40.4	264	-2	-2.2	-42
036A	30	43.6	240	29	42.8	232	-1	-0.8	-8
037A	44	64.6	463	43	65.9	473	-1	1.4	11
038A	30	8.6	224	3	0.0	4	-27	-8.6	-220
039A	27	9.9	88	26	8.8	87	-1	-1.0	-1
040A	42	16.3	262	43	16.6	300	1	0.3	39
041A	53	191.9	591	51	188.0	562	-2	-3.9	-29
042A	19	11.5	54	18	11.4	47	-1	-0.2	-7
043A	3	0.2	1	3	0.0	1	0	-0.1	-1
Total	**-**	**1068.5**	**-**	**-**	**1032.5**	**-**	**-**	**-36.0**	**-**

7.6.7 Cost Summary

The Class 5 (+100%, -50%) cost estimate for the green stormwater infrastructure control program is summarized in Table 7-50.

Table 7-50: Control Program 6 – Green Infrastructure, Cost Summary

Control Level, Percent of Impervious Area Managed, System-Wide	2.5%	5%	7.5%	10%	15%
Bioswale Area (acres)	1.42	2.76	4.16	5.53	8.29
Permeable Pavement Area (acres)	7.97	15.5	23.4	31.1	46.6
Bioswale Cost ($ Million)	$77.10	$150.5	$226.3	$301.0	$451.5
Permeable Pavement Cost	$6.511	$12.71	$19.11	$25.42	$38.13
Total Construction Costs ($ Million)	*$83.61*	*$163.24*	*$245.39*	*$326.40*	*$489.63*
Land Acquisition Costs ($ Million)	$0	$0	$0	$0	$0
Non-Construction Costs ($ Million)	$21.0	$41.0	$61.0	$82.0	$122.0
Total Capital Cost ($ Million)	**$104.6**	**$204.2**	**$306.4**	**$408.4**	**$611.6**
Annual O&M Cost ($ Million)	$0.08	$0.15	$0.22	$0.29	$0.44
Total Present Worth ($ Million)	**$105.6**	**$206.2**	**$309.4**	**$412.4**	**$618.6**
CSO Volume Abated (MG)	16.2	22.6	26.6	31.3	36.0
Cost per Gallon Abated ($/gal)	$6.52	$9.13	$11.63	$13.18	$17.18

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7.7 Control Program 7: Inflow/Infiltration Reduction

Excessive infiltration and inflow (I/I) has the potential to cause conveyance related issues during wet weather flow (WWF), as well as operational issues at treatment facilities. Conveyance-related issues include surcharging of the sewer system, which can cause surface flooding if sufficiently severe. In a combined sewer system, I/I can potentially increase system flows during wet weather to levels that restrict the capture and/or treatment of combined sewage.

As demonstrated by the collection system modeling results presented in JMEUC's System Characterization Report (SCR; see Section 7), JMEUC experiences neither capacity nor treatment related issues during wet weather flow for the Typical Year precipitation record. Of particular importance to the CSO LTCP, the JMEUC trunk sewers and WWTF can capture and treat all flow from the JMEUC service area during the Typical Year, including all flow from TAPS for both the existing TAPS capacity (36 mgd) and the proposed interim TAPS expansion (55 mgd). I/I, while of sufficient magnitude to cause surcharging in some reaches of the JMEUC trunk sewer system, does not cause measurable flooding in the system, and does not restrict the capture of combined sewage from Elizabeth.

Although I/I originating from upstream member municipalities does not cause conveyance or treatment related issues, I/I reduction has the potential to effectively increase the conveyance capacity downstream of the Trenton Avenue Pump Station (TAPS) and through the JMEUC WWTF available for capture and treatment of additional combined sewage flow from Elizabeth during wet weather. Because the existing JMEUC trunk sewers and WWTF can handle current and future TAPS flows (at 55 mgd) during wet weather, the primary benefit to reducing I/I rates would be to reduce the capacity of additional facilities that would be constructed to provide treatment of additional flows from an expanded TAPS pump station and new force main, as described in Section 7.3. In this case, additional wet weather combined sewage from Elizabeth could be directed to the existing JMEUC trunk sewers and WWTF at rates equal to the reduction in I/I rates, which would reduce by the same amount the flow rates used in sizing of a new force main and CSO treatment facilities.

In this section of the report, existing I/I rates and volumes are quantified on a community and metershed basis within the JMEUC service area, and these I/I levels are evaluated using the definitions of non-excessive I/I as defined in NJAC 7:14A-1.2. These I/I levels and potential I/I reduction program effectiveness are also evaluated for similar Northeast and Midwest sewer systems. The calibrated Baseline Merged Model developed as part of the SCR has been used to determine the impact I/I reduction could have on reducing peak flows in the JMEUC trunk sewers, and the associated increase in potential combined sewage capture rates. A cost analysis was performed to establish estimated total and unit costs for I/I removal on a municipality and system-wide basis. These costs are then compared to the potential savings in CSO capture and treatment costs that I/I reduction could potentially provide.

7.7.1 JMEUC's Existing I/I Reduction Program

JMEUC encourages member municipalities to reduce I/I and provides significant resources to them in support of their I/I reduction program. Billing rates (developed annually) are based largely on measured I/I from the prior year, which provides incentive to communities to reduce I/I to lower costs which are passed on to their residents. Significant I/I reduction has been achieved since 1983 when Phase II-B Sewer System Evaluation Survey (SSES) studies were completed for each member municipality. These studies identified locations of excessive I/I and provided a basis for the I/I reduction that has subsequently been completed by member municipalities since. Table 7-51 provides a summary of the I/I reduction achieved between 1983 and 2017 on a member municipality basis.

From Table 7-51, an estimated 40% of infiltration and 34% of inflow have been removed from upstream member municipalities since 1983. As discussed in Section 7.7.2, a comprehensive I/I reduction program can expect to achieve up to 50% I/I reduction from a system-wide standpoint, indicating significant I/I reduction has already been achieved by JMEUC member municipalities. It should be noted that the terms used in the Annual Assessment Report (AAR) for inflow and infiltration align with the terms used in the JMEUC System Characterization Report (SCR) as follows: the term “inflow” is used the AAR to represent rainfall-dependent I/I (RDII), and the term “infiltration” is used in the AAR to represent groundwater infiltration (GWI). For a more detailed summary on JMEUC’s I/I reduction program and associated SSES studies, the reader is directed to Section 9.5.1 of JMEUC’s SCR.

Table 7-51: Estimated I/I reduction of JMEUC member municipalities between 1983 and 2017

	Infiltration (gpd)[1]				Inflow (gpd)[1]			
Municipality	Identified	Removed	% Removed	Balance	Identified	Removed	% Removed	Balance
East Orange	70,747	26,725	38%	44,022	3,007,440	2,789,280	93%	218,160
Hillside	79,012	46,032	58%	32,980	1,185,120	0	0%	1,185,120
Irvington	1,115,672	325,649	29%	790,023	8,612,640	1,830,195	21%	6,782,445
Maplewood	389,078	191,845	49%	197,233	5,449,680	3,551,760	65%	1,897,920
Millburn	191,609	39,369	21%	152,240	2,729,520	396,000	15%	2,333,520
Newark	234,484	46,978	20%	187,506	1,959,540	43,665	2%	1,915,875
Roselle Park	106,187	41,040	39%	65,147	1,576,080	0	0%	1,576,080
South Orange	410,876	326,970	80%	83,906	2,183,760	339,120	16%	1,844,640
Summit	171,657	106,741	62%	64,916	3,651,120	2,046,240	56%	1,604,880
Union	329,127	94,604	29%	234,523	14,534,640	6,202,800	43%	8,331,840
West Orange	250,811	109,664	44%	141,147	7,097,040	683,640	10%	6,413,400
Total	**3,349,260**	**1,355,617**	**40%**	**1,993,643**	**51,986,580**	**17,882,700**	**34%**	**34,103,880**

1. Values adopted from JMEUC’s 2018 Annual Assessment Report

7.7.2 I/I Reduction Strategies and Attainable Reduction Percentages

I/I reduction is a strategy often implemented by municipalities with separate sewers systems experiencing capacity and treatment related issues during wet weather events. I/I can originate from both public (sewer mains) and private sources (service laterals) which can make it difficult to isolate and mitigate.

Private I/I sources include:

- Foundation drains connected to the service lateral or sewer,
- Area drains connected to the service lateral or sewer,
- Damaged or deteriorated pipe sections or joints in the service laterals,
- Directly (or indirectly) connected roof drains and
- Sump pumps connected to the sanitary system.

Public I/I sources include:

- Cross-connections between the storm and sanitary sewer systems,
- Damage or deterioration to the sanitary sewer pipeline and joints,
- Offset or separated joints in the sanitary sewer and
- Deteriorated manholes.

Reduction techniques for private source I/I can involve:

- Disconnecting downspouts,
- Foundation drains or sump pumps,
- Replacing or lining damaged or deteriorated service laterals.

Reduction techniques for public source I/I can include the following:

- Lining or replacing (depending on structural condition) sanitary sewer mainlines and manholes

- Eliminating any sanitary/storm cross-connections
- Chemically grouting non-watertight joints along the mainline.

National experience has shown that the effectiveness of I/I reduction programs can vary greatly. Various factors influence effectiveness:

1. The scale of the program can affect the results. A high level of effectiveness that may be possible on a small scale (e.g. one or two neighborhoods) is not likely to be achieved on a larger scale (e.g. a full sewer system covering several square miles).
2. The nature of the program and techniques used can affect the results. A program that is comprehensive in nature (i.e. addressing all I/I sources, both public and private) can achieve higher levels of reduction than more limited programs. Also, comprehensive lining or replacement programs will generally achieve greater reductions than point repair programs.
3. The monitoring approach and evaluation methods can influence the results (or the reported results). Significant attention must be paid to the collection and analysis of flow monitoring data pre-I/I reduction and post-I/I reduction in order to accurately estimate program effectiveness. Difference in precipitation conditions during the pre- and post- monitoring periods must be properly accounted for to ensure appropriate comparisons of I/I rates and reliable estimates of effectiveness.

The national experience with I/I reduction programs has shown that levels of effectiveness up to roughly a 50% reduction in peak I/I rates and volumes can be achieved from a system-wide standpoint when compared to baseline system conditions (i.e. pre-program, before any I/I reduction efforts have taken place). Hartford, Connecticut MDC and MWRA (in greater Boston) are two sewerage authorities which have extensively addressed I/I in their systems by implementing some or all of the reduction techniques listed above. JMEUC used available data from these authorities to quantify attainable I/I reduction and validate the attainable reduction level of 50% system-wide for costing and modeling purpose. These and other data also were used to provide useful comparisons of the existing I/I rates in the JMEUC service area to those of similar collection systems, to supplement the definitions of non-excessive I/I as defined in NJAC 7:14A-1.2 as a basis for evaluating the I/I conditions in the JMEUC service area.

7.7.2.1 II/I Program Experience – MWRA

Table 7-52 summarizes I/I reduction from 2006 to 2017 of individual municipalities with separate sewer systems serviced by MWRA. The average infiltration volumes in Table 7-52 were obtained directly from Annual Infiltration and Inflow Reduction Reports submitted to the Massachusetts Department of Environmental Protection (MADEP), while average inflow volumes were first normalized to the Typical Year (EWR 2004) rainfall assuming a linear relationship between inflow and precipitation. (Source: MWRA Annual Infiltration and Inflow (I/I) Reduction Report for Fiscal Year 2018, August 28, 2018, Massachusetts Water Resources Authority; and MWRA Annual Infiltration and Inflow (I/I) Reduction Report for Fiscal Year 2007, August 30, 2007, Massachusetts Water Resources Authority.)

Table 7-52 indicates that from 2006 to 2017, MWRA reduced their infiltration levels by 26.66% and their inflow by 40.60% from a system-wide standpoint. The majority of individual municipalities realized reductions of infiltration and inflow of less than 50%, while a select few municipalities saw increases in their I/I volumes.

7.7.2.2 I/I Program Experience – Hartford MDC

As part of requirements of a 2006 Consent Decree issued to Hartford MDC by the United States Environmental Protection Agency (USEPA), Hartford MDC implemented a Clean Water Project to address, among other things, sanitary sewer overflows (SSOs). To assess the effectiveness of I/I reduction in their system, Hartford MDC developed a pilot study program to evaluate the effectiveness of different types of I/I reduction techniques. Table 7-53 summaries the annualized results of this study. Table 7-53 footnotes describe the assumptions and normalization of data that were necessary in order to

Table 7-52: Reduction of I/I Volumes by MWRA Member Municipalities between 2006 and 2017

	Infiltration [2]			Inflow [1,2]			Total I/I		
Municipality	2006 average infiltration (MG)	2017 average infiltration (MG)	% Reduction	2006 average inflow (MG)	2017 average inflow (MG)	% Reduction	2006 I/I (MG)	2017 I/I (MG)	% Reduction
Arlington	857.75	686.2	20.00%	283.74	170.66	39.85%	1141.49	856.86	24.93%
Ashland	120.45	186.15	-54.55%	40.06	24.38	39.14%	160.51	210.53	-31.17%
Bedford	730	401.5	45.00%	66.76	60.95	8.71%	796.76	462.45	41.96%
Belmont	613.2	383.25	37.50%	226.99	150.34	33.77%	840.19	533.59	36.49%
Braintree	1299.4	959.95	26.12%	360.51	231.61	35.76%	1659.91	1191.56	28.22%
Burlington	689.85	496.4	28.04%	120.17	60.95	49.28%	810.02	557.35	31.19%
Canton	598.6	507.35	15.24%	186.93	77.20	58.70%	785.53	584.55	25.59%
Dedham	996.45	602.25	39.56%	240.34	109.71	54.35%	1236.79	711.96	42.43%
Everett	660.65	715.4	-8.29%	210.30	199.10	5.32%	870.95	914.50	-5.00%
Framingham	470.85	708.1	-50.39%	303.76	121.90	59.87%	774.61	830.00	-7.15%
Hingham	211.7	262.8	-24.14%	86.79	36.57	57.86%	298.49	299.37	-0.29%
Holbrook	124.1	113.15	8.82%	20.03	20.32	-1.44%	144.13	133.47	7.40%
Lexington	1043.9	894.25	14.34%	206.96	130.03	37.17%	1250.86	1024.28	18.11%
Malden	1726.45	1138.8	34.04%	277.06	251.92	9.07%	2003.51	1390.72	30.59%
Medford	1613.3	941.7	41.63%	537.43	341.32	36.49%	2150.73	1283.02	40.35%
Melrose	744.6	744.6	0.00%	290.41	203.17	30.04%	1035.01	947.77	8.43%
Milton	733.65	543.85	25.87%	320.45	138.15	56.89%	1054.10	682.00	35.30%
Natick	324.85	270.1	16.85%	80.11	52.82	34.07%	404.96	322.92	20.26%
Needham	813.95	540.2	33.63%	226.99	109.71	51.67%	1040.94	649.91	37.57%
Newton	3365.3	2036.7	39.48%	727.70	459.15	36.90%	4093.00	2495.85	39.02%
Norwood	938.05	1076.75	-14.79%	337.15	162.53	51.79%	1275.20	1239.28	2.82%
Quincy	2507.55	1423.5	43.23%	580.82	353.51	39.14%	3088.37	1777.01	42.46%
Randolph	584	671.6	-15.00%	176.92	105.65	40.29%	760.92	777.25	-2.15%
Reading	558.45	529.25	5.23%	136.86	81.27	40.62%	695.31	610.52	12.20%
Revere	1022	693.5	32.14%	360.51	300.68	16.60%	1382.51	994.18	28.09%
Stoneham	532.9	448.95	15.75%	170.24	73.14	57.04%	703.14	522.09	25.75%

Municipality	Infiltration [2]			Inflow [1,2]			Total I/I		
	2006 average infiltration (MG)	2017 average infiltration (MG)	% Reduction	2006 average inflow (MG)	2017 average inflow (MG)	% Reduction	2006 I/I (MG)	2017 I/I (MG)	% Reduction
Stoughton	708.1	511	27.84%	120.17	60.95	49.28%	828.27	571.95	30.95%
Wakefield	1025.65	846.8	17.44%	206.96	121.90	41.10%	1232.61	968.70	21.41%
Walpole	514.65	277.4	46.10%	73.44	40.63	44.67%	588.09	318.03	45.92%
Waltham	2124.3	1011.05	52.41%	293.75	174.72	40.52%	2418.05	1185.77	50.96%
Watertown	587.65	365	37.89%	150.21	109.71	26.96%	737.86	474.71	35.66%
Wellesley	602.25	470.85	21.82%	250.36	97.52	61.05%	852.61	568.37	33.34%
Westwood	197.1	284.7	-44.44%	70.10	44.70	36.24%	267.20	329.40	-23.28%
Weymouth	1408.9	1489.2	-5.70%	387.22	227.54	41.24%	1796.12	1716.74	4.42%
Wilmington	76.65	219	-185.71%	63.42	20.32	67.97%	140.07	239.32	-70.85%
Winchester	397.85	386.9	2.75%	103.48	73.14	29.32%	501.33	460.04	8.24%
Winthrop	346.75	310.25	10.53%	123.51	89.39	27.62%	470.26	399.64	15.02%
Woburn	2233.8	865.05	61.27%	447.30	178.79	60.03%	2681.10	1043.84	61.07%
Total	**34105.6**	**25013.45**	**26.66%**	**8865.91**	**5266.04**	**40.60%**	**42971.51**	**30279.49**	**29.54%**

1. Normalized to EWR 2004 rainfall assuming linear relationship between inflow and precipitation.
2. Values from MWRA's Annual Infiltration and Inflow Reduction Reports submitted to MADEP.

Table 7-53: I/I Volume Reduction Measures and Results of Hartford MDC Pilot Study

Subarea	I/I Reduction Measures	Inflow [4]			Infiltration [3]			Total I/I					
		Annualized Baseline Inflow (MG) [1]	Annualized Post I/I Reduction Inflow (MG) [2]	% Reduction	Annualized Baseline Infiltration (MG) [1]	Annualized Post I/I Reduction Infiltration (MG) [2]	% Reduction	Annualized Baseline Inflow and Infiltration Volume (MG) [1]	Annualized Post I/I Reduction I/I Volume (MG) [2]	% Reduction	Volumetric I/I Reduction (MG)	I/I Reduction Cost (Million $)	Normalized Cost ($/gal)
WI3A	Private I/I Removal.	8.25	4.33	48%	6.64	9.95	-50%	14.89	14.28	4%	0.60	2.24	$3.72
WI8	CIPP lining of sewer mains and manholes, replacement of manhole frames and covers. CIPP lining of laterals up to property line.	60.20	29.22	51%	209.10	135.82	35%	269.30	165.03	39%	104.27	4.35	$0.04
FB2	Private I/I Removal and CIPP lining of sewer mains.	31.50	28.91	8%	86.27	69.68	19%	117.77	98.59	16%	19.18	2.89	$0.15
RH2AN	CIPP lining of sewer mains and CIPP lining/replacement of laterals.	69.85	41.76	40%	96.23	43.14	55%	166.08	84.90	49%	81.18	3.90	$0.05
RH2AS	CIPP lining of sewer mains and CIPP lining/replacement of laterals.	32.87	22.49	32%	29.86	43.14	-44%	62.73	65.62	-5%	-2.89	3.16	N/A
N4	CIPP lining of sewer mains and manholes, replacement of manhole frames and covers. CIPP lining of laterals up to property line.	111.74	88.85	20%	185.82	217.82	-17%	297.56	306.66	-3%	-9.10	0.65	N/A
N6	CIPP lining of sewer mains and manholes, replacement of manhole frames and covers. CIPP lining of laterals up to property line.	39.94	23.09	42%	46.45	63.05	-36%	86.39	86.13	0%	0.26	0.66	$2.52
N8	CIPP lining of sewer mains and manholes, replacement of manhole frames and covers. CIPP lining of laterals up to property line.	161.83	199.01	-23%	428.05	234.41	45%	589.88	433.42	27%	156.46	1.61	$0.01
WH8	CIPP lining of sewer mains and manholes, replacement of manhole frames and covers. CIPP lining of laterals up to property line.	31.82	33.79	-6%	53.09	19.91	63%	84.91	53.70	37%	31.21	1.11	$0.04
WH9N	CIPP lining of sewer mains, replacement of manhole frames and covers.	33.69	36.86	-9%	86.27	43.14	50%	119.97	80.00	33%	39.97	0.53	$0.01
WH9S	CIPP lining of sewer mains and manholes, replacement of manhole frames and covers. CIPP lining of laterals up to property line.	18.43	15.36	17%	56.41	49.77	12%	74.84	65.13	13%	9.70	1.90	$0.20
WH34	CIPP lining of sewer mains, replacement of manhole frames and covers. CIPP lining of laterals up to cleanout.	56.01	49.15	12%	50.47	69.68	-38%	106.48	118.83	-12%	-12.35	2.51	N/A
Total		**656.14**	**572.81**	**13%**	**1334.66**	**999.50**	**25%**	**1,990.80**	**1,572.30**	**21%**	**418.50**	**25.52**	**$0.06**

1. Metering period occurred in either 2005 or 2011 depending on when I/I reduction began. 2005 flow meter and precipitation data was collected between 3/10/2015 and 6/6/2015 and 2011 flow meter and precipitation data was collected between 2/20/2011 and 5/31/2011
2. Post I/I reduction metering was completed for all Subareas in 2014. 2014 flow meter and precipitation data was collected between 2/20/2011 and 5/31/2011.
3. To approximate annual infiltration volume and account for seasonality observed in GWI, the following adjustments were made to reported infiltration volumes to arrive at tabulated values above: Annualized Infiltration = Reported Infiltration * 365 days/metered days * 1.1 where 1.1 = (Average infiltration in March through May)/(Average infiltration over entire year) for JMEUC.
4. To approximate annual inflow volume and account for seasonality observed in rainfall derived inflow, the following adjustments were made to reported inflow volumes to arrive at tabulated values above:
  a. Normalize reported inflow to EWR 2004 rainfall occurring during same period of flow meter data collection (ex: Subarea WI3A experienced 16.17" of rainfall from 2/20/2011 to 5/31/2011. From 2/20/2004 to 5/31/2004 EWR reported 12.5" of rainfall. Assuming linear relationship between precipitation and inflow, multiply reported inflow volume (3.03 MG) by 12.5"/16.17" to arrive at normalized inflow volume of 2.05 MG)
  b. Assuming similar inflow seasonality between JMEUC and Hartford MDC, extrapolate inflow volume from a. to entire Typical Year (ex: 47% of JMEUC's Typical Year inflow (from Calibrated Baseline Merged Model output) occurs between 2/20/2004 and 5/31/2004. Multiply inflow volume from a. (2.05 MG) by 1/0.47 to arrive at 4.33 MG of annual inflow occurring within Subarea WI3A.

arrive at annualized I/I volumes due to only spring flow meter data being available. (Source: Sewer Rehabilitation Pilot Study (FINAL REPORT), October 2016, CDM Smith.)

Table 7-53 indicates that on the whole, Hartford MDC was able to reduce I/I by an estimated 21% (13% inflow, 25% infiltration) in pilot study areas where I/I reduction techniques were implemented. Because these pilot study areas were generally smaller than the service areas of entire municipalities, variations in percent reduction values were greater. Table 7-54 below summarizes metershed/municipality characteristics of MWRA and Hartford MDC systems, along with Cincinnati MSD and JMEUC (for reference). The parameters found in this table were used as normalization parameters during I/I analysis, discussed in the subsequent sections.

Table 7-54: Community and Metershed Characteristics of JMEUC, Cincinnati MSD, MWRA, and Hartford MDC Collections Systems

Count	Dataset	Community/Metershed	Population	Area (ac)	IDM	Miles of Local Sewer
1	JMEUC	Meter04 - Roselle Park	11,652	683	63.24	3.45
2		Meter05/05A - Union	55,221	5,143	1,013.84	122.05
3		Meter06 - Hillside	17,714	1,294	283.13	34.41
4		Meter10 - Newark	4,907	194	43.50	5.20
5		Meter12 - Newark	2,723	79	19.26	2.40
6		Meter13 - East Orange	6,659	218	77.09	8.72
7		Meter14 - East Orange	10,755	348	104.84	11.11
8		Meter15 - South Orange	1,915	161	35.28	4.41
9		Meter16 - Irvington	2,845	79	24.93	2.98
10		Meter17/17E - Newark	11,601	468	113.65	13.45
11		Meter18 - Newark	23,727	467	114.70	14.12
12		Meter21 - Maplewood	8,840	505	98.03	11.87
13		Meter22 - Maplewood	3,771	228	53.65	6.70
14		Meter24 – Summit/New Providence	31,978	5,702	675.52	82.39
15		Meter25 - Maplewood	6,131	527	119.54	14.64
16		Meter26/31 - Maplewood	4,873	635	128.35	15.82
17		Meter27 - South Orange	6,696	517	103.36	12.85
18		Meter28 - South Orange	7,760	1,152	200.93	24.95
19		Meter29/30 - West Orange	45,578	5,444	940.53	111.53
20		Meter32C - Millburn	6,610	1,308	207.70	25.72
21		Meter32D - Millburn	6,982	2,002	284.05	34.95
22		Meter32E - Millburn	3,722	534	102.30	12.49
23		Meter34 - Hillside	1,990	272	31.66	3.69
24		Meter9 - Irvington	18,386	547	203.37	24.74
25		Meter9A/9A-Up - Irvington	33,481	1,272	239.26	28.44
1	Cincinnati MSD	7413024-7413025		480		
2		7413003-7414002		265		
3		49406007-49405001		1,146	161.07	14.18
4		49704009-49704010		898	211.34	18.78
5		49806014-49806015		265	50.37	4.39
6		49806021-49806022		393	85.65	7.44

Count	Dataset	Community/Metershed	Population	Area (ac)	IDM	Miles of Local Sewer
7		53002006-53002008		573	89.33	8.86
8		53214006-53214004		243	52.14	4.94
9		53302007-53302008		274	58.45	5.45
10		53406011-53406013		232	49.79	4.28
11		53513011-53513010		269	39.27	3.55
12		54605013-54605007		223	33.25	3.33
13		14610007-14610006		160		
14		18709001-18716008		367		
15		21315001-21315002		219		
16		21402002-21402003		272		
17		22907008-22907009		265		
18		22915001-22910006		240		
1	MWRA	Arlington	44,028		954.08	106.00
2		Ashland	17,150		594.10	66.00
3		Bedford	13,975		738.06	78.00
4		Belmont	25,332		708.01	78.00
5		Braintree	36,727		1,300.02	140.00
6		Burlington	25,463		1,150.07	115.00
7		Canton	22,221		566.97	62.00
8		Dedham	25,299		832.03	95.00
9		Everett	42,935		686.01	57.00
10		Framingham	70,441		2,750.39	275.00
11		Hingham	7,350		296.98	33.00
12		Holbrook	10,952		312.05	31.00
13		Lexington	32,650		1,763.04	170.00
14		Malden	60,509		1,000.00	100.00
15		Medford	57,170		1,130.06	113.00
16		Melrose	27,690		641.01	74.00
17		Milton	27,270		746.99	83.00
18		Natick	35,214		1,179.85	135.00
19		Needham	29,736		1,231.86	132.00
20		Newton	87,971		2,710.12	271.00
21		Norwood	28,951		1,091.08	108.00
22		Quincy	93,494		2,019.93	202.00
23		Randolph	33,456		1,137.91	101.00
24		Reading	25,327		863.94	96.00
25		Revere	53,756		1,433.91	98.00
26		Stoneham	21,734		566.99	63.00
27		Stoughton	28,106		887.95	88.00
28		Wakefield	26,080		887.93	93.00
29		Walpole	24,818		577.03	59.00

Count	Dataset	Community/Metershed	Population	Area (ac)	IDM	Miles of Local Sewer
30		Waltham	62,227		1,379.96	138.00
31		Watertown	32,996		674.97	75.00
32		Wellesley	29,090		1,340.25	134.00
33		Westwood	14,876		692.99	77.00
34		Weymouth	55,419		2,380.27	238.00
35		Wilmington	23,147		279.99	20.00
36		Winchester	22,079		746.89	83.00
37		Winthrop	18,111		323.98	36.00
38		Woburn	39,083		1,410.01	141.00
1	MDC	WI3A		156	25.50	3.06
2		WI8		1,342	196.90	21.37
3		FB2		395	61.98	6.03
4		RH2AN		209	44.76	5.30
5		RH2AS		75	15.10	1.81
6		N4		1,677	239.28	18.87
7		N6		350	43.80	5.20
8		N8		2,793	603.45	66.79
9		WH8		163	38.80	4.61
10		WH9N		206	32.30	3.55
11		WH9S		74	22.40	2.51
12		WH34		371	58.30	5.90

Note: Blank fields were not readily available for analysis purposes.

7.7.3 JMEUC Infiltration Analysis

Using model output from the calibrated Baseline Merged Model in InfoWorks ICM developed as part of system characterization, JMEUC developed normalized infiltration rates on a metershed basis and compared these infiltration rates to “non-excessive infiltration” rates defined in the New Jersey Administrative Code (N.J.A.C). JMEUC also compared the infiltration rates in the JMEUC service area to those of other large sewerage authorities (Cincinnati MSD, Hartford MDC, and the MWRA) with separate sewer service areas. Infiltration rates for Cincinnati MSD were obtained directly from calibrated model output, while infiltration rates for MWRA were obtained from annual reports submitted to MADEP. As discussed in Section 7.7.2.2, infiltration rates for Hartford MDC were limited to only spring data and as a result were adjusted to account for seasonal variation.

“Non-excessive infiltration” and “Excessive inflow/infiltration” are defined by N.J.A.C as follows:

“Non-excessive infiltration” - The quantity of flow which is less than 120 gallons per capita per day (domestic base flow and infiltration) or the quantity of infiltration which cannot be economically and effectively eliminated from a sewer system as determined in a cost-effectiveness analysis. For domestic treatment works receiving wastewater from combined sewers, non-excessive infiltration means the quantity of flow attributable to infiltration during dry weather shall be less than 40 gallons per capita per day (gpcd) or 1,500 gallons per day per inch diameter per mile of sewer.

"Excessive inflow/infiltration" - The quantities of infiltration/inflow (I/I) which can be economically eliminated from a sewer system as determined in a cost effectiveness analysis that compares the cost for correcting the I/I conditions to the total costs for transportation and treatment of the I/I.

Using output from the calibrated Baseline Merged Model allowed for explicit representation of domestic base flow, infiltration, and inflow from each modeled metershed. Simulated flows were calibrated using meter data collected over multiple years (2015-2017). Using the calibrated Baseline Merged Model allowed JMEUC to assess system performance using Typical Year (2004) rainfall. During model development, it was assumed that infiltration made up roughly 80% of the minimum nighttime flow during dry weather (when sewer use is assumed to be minimal), while the remaining 20% was (by default) base sanitary flow.

7.7.3.1 Per Capita Infiltration Analysis

Table 7-55 presents normalized (on a per capita basis) average dry weather flow (ADWF) rates on both a metershed and monthly basis for metersheds making up JMEUC's service area.

As indicated in Table 7-55, 40% (10 out of 25) of metersheds making up the JMEUC service area have annual ADWF exceeding 120 gpcd. A composite ADWF of 107 gpcd was calculated for all JMEUC member municipalities using the population of each metershed as a weighting factor. This was done to better quantify infiltration from a system-wide standpoint. This composite value indicates that system-wide, infiltration from upstream member municipalities falls within the non-excessive threshold of less than 120 gpcd based on N.J.A.C.

For comparison purposes, JMEUC per capita DWF rates were compared to the per capita DWF rates of MWRA's 38 member municipalities with separate sewer systems. Population data was not readily available for the portions of Cincinnati MSD and Hartford MDC collection systems used in analysis. Figure 7-35 shows box and whisker plots of ADWF on a per capita basis for JMEUC metersheds and MWRA member municipalities. It is noted that significant I/I reduction was performed throughout MWRA member municipalities between 2006 and 2017 (See Section 7.7.2.1), with infiltration from a system-wide standpoint decreasing by 26.7% between 2006 and 2017. The composite ADWF values for MWRA from a system-wide standpoint were calculated to be 135.7 gpcd before I/I reduction, and 121.2 gpcd after I/I reduction.

7.7.3.2 JMEUC Infiltration on an IDM Basis

In addition to normalizing ADWF on a per capita basis, JMEUC normalized infiltration on an inch-diameter per mile of sewer (IDM) basis to compare infiltration for each metershed to N.J.A.C's "non-excessive infiltration" threshold of 1,500 gpd/IDM.

From Table 7-56, only two of 25 metersheds in the JMEUC service area have an average annual infiltration rate of less than 1,500 gpd/IDM. The average annual composite infiltration rate of the entire JMEUC system (weighted based on the IDM of each individual metershed) is 2,959 gpd/IDM.

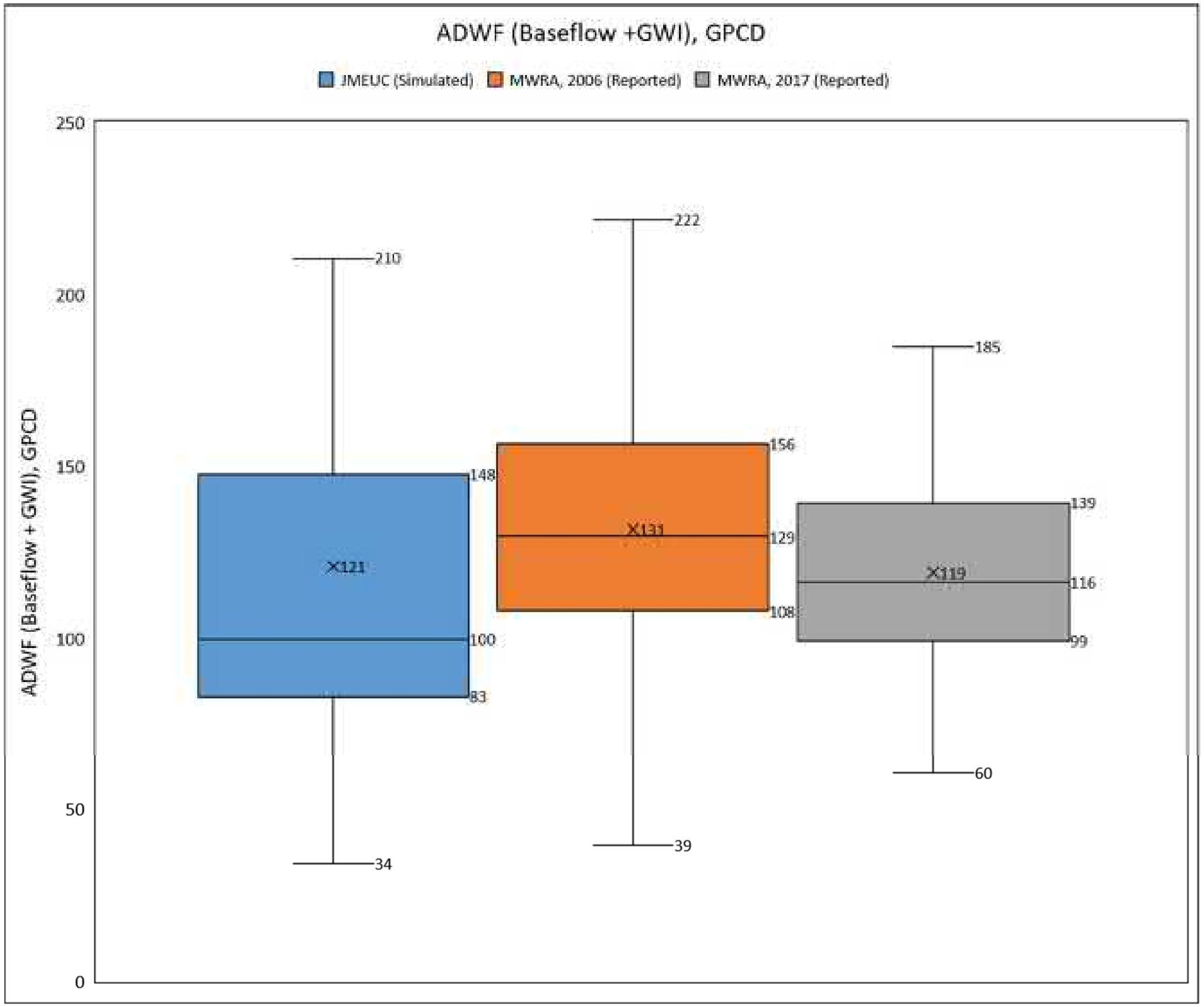
Figure 7-36 presents box and whisker plots of JMEUC average annual infiltration rates alongside those of Hartford MDC and MWRA. The JMEUC infiltration rates normalized by IDM are comparable to those of Hartford MDC, but generally greater than MWRA's. All Sewerage Authorities mean and median infiltration rates on a gpd/IDM basis exceed the N.J.A.C "non-excessive" threshold of 1,500 gpd/IDM.

Disaggregating infiltration from baseflow (as was done above) for large metersheds can often present challenges. Additionally, normalizing infiltration by IDM can present complications, as different sewered areas with the same population could differ significantly in their respective IDM values. Because of these factors, using ADWF on a per capita basis to assess infiltration is generally a more reliable way to identify excessive infiltration, as was done in Section 7.7.3.1.

Table 7-55: Per Capita ADWF rates of JMEUC Metersheds

Metershed	Municipality	Population [2]	Service Area (ac)	Average DWF (Domestic base flow + Infiltration) gcpd, Disaggregated by month [4]												
				1	2	3	4	5	6	7	8	9	10	11	12	Annual Avg.
Meter04	Roselle Park	11,652	683	108	111	117	117	110	93	80	69	70	74	74	87	92
Meter05/05A	Union	55,221	5,143	87	89	90	91	88	83	79	73	74	74	80	84	83
Meter06	Hillside	17,714	1,294	152	153	152	153	152	150	150	149	146	145	147	150	150
Meter10	Newark	4,907	194	103	108	108	111	109	103	91	90	92	92	94	98	100
Meter12	Newark	2,723	79	105	105	106	106	97	94	94	94	92	97	97	101	99
Meter13	East Orange	6,659	218	87	88	87	86	85	82	82	82	81	77	79	84	83
Meter14	East Orange	10,755	348	123	134	133	132	132	128	127	121	112	106	109	118	123
Meter15	South Orange	1,915	161	147	158	171	171	160	139	130	121	119	99	105	115	136
Meter16	Irvington	2,845	79	101	101	101	100	98	96	91	88	87	86	86	91	94
Meter17/17E	Newark	11,601	468	35	36	36	34	34	34	33	33	33	34	34	34	34
Meter18	Newark	23,727	467	87	94	95	95	95	80	74	71	64	66	69	76	80
Meter21	Maplewood	8,840	505	77	77	77	75	74	73	72	71	71	71	72	74	74
Meter22	Maplewood	3,771	228	74	74	76	75	72	71	70	68	67	68	70	73	71
Meter24 1	Summit/New Providence	31,978	5,702	128	138	143	138	120	110	108	108	109	105	106	120	119
Meter25	Maplewood	6,131	527	110	116	123	123	114	100	77	70	74	75	79	107	97
Meter26/31	Maplewood	4,873	635	185	190	201	193	191	182	150	150	146	150	156	175	172
Meter27	South Orange	6,696	517	146	149	154	144	140	126	108	103	103	108	111	130	127
Meter28	South Orange	7,760	1,152	191	191	190	191	191	185	178	174	171	174	176	187	183
Meter29/30	West Orange/Orange	45,578	5,444	104	107	114	114	114	104	96	89	83	88	87	97	100
Meter32C	Millburn	6,610	1,308	98	109	109	106	94	80	72	72	72	76	81	98	89
Meter32D	Millburn/Livingston	6,982	2,002	221	233	244	227	227	207	197	185	183	183	197	221	210
Meter32E	Millburn	3,722	534	196	227	227	217	185	146	125	125	125	138	150	196	171
Meter34	Hillside	1,990	272	321	320	321	324	315	324	324	319	327	319	319	325	322
Meter9	Irvington	18,386	547	66	68	69	70	66	62	59	58	58	57	57	62	63
Meter9A/9A-Up	Irvington	33,481	1,272	146	148	149	148	144	142	143	141	141	146	146	147	145
Composite 3	All	336,517	29,779	113	117	120	118	114	107	101	97	96	97	99	108	107

1. Based on 2017 American Community Survey estimates.
2. Average DWF was weighted using each metershed's population.
3. Values highlighted exceed N.J.A.C's 120 gcpd "nonexcessive infiltration" threshold



Notes:

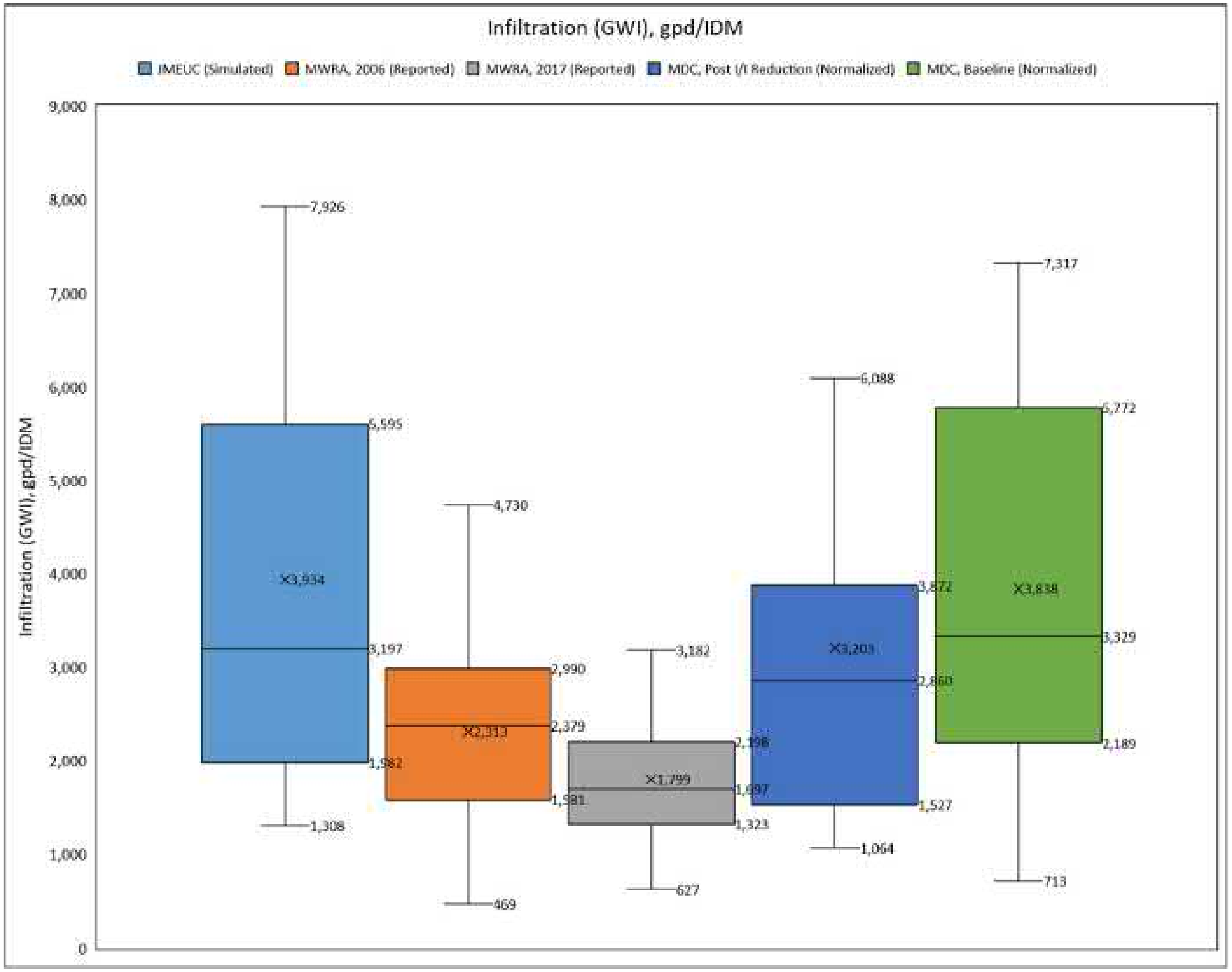
1. Outliers excluded from plots.
2. 25% and 75% quartiles were used for plotting purposes.
3. Mean values in each plot do not represent composite ADWF values described above, rather they represent the average of each dataset using equal weighting regardless of population of metershed/municipality.

Figure 7-35: Per capita ADWF for JMEUC metersheds and MWRA municipalities

Table 7-56: Monthly Infiltration Rates of JMEUC Metersheds on an IDM Basis

			Infiltration (gpd/IDM), Disaggregated by month [3]												
Metershed	Municipality	IDM [1]	1	2	3	4	5	6	7	8	9	10	11	12	Annual Avg.
Meter04	Roselle Park	63.24	9,357	10,459	11,398	11,459	10,353	7,374	4,953	2,829	2,907	3,501	3,541	5,767	6,992
Meter05/05A	Union	1013.84	1,509	1,636	1,703	1,766	1,594	1,370	1,122	802	814	852	1,132	1,392	1,308
Meter06	Hillside	283.13	2,526	2,603	2,562	2,627	2,541	2,444	2,437	2,369	2,182	2,101	2,255	2,449	2,424
Meter10	Newark	43.5	6,006	6,588	6,626	6,986	6,829	6,119	4,832	4,602	4,876	4,897	5,047	5,514	5,743
Meter12	Newark	19.26	8,423	8,489	8,708	8,650	7,479	6,963	7,004	6,936	6,713	7,426	7,404	7,909	7,675
Meter13	East Orange	77.09	3,903	4,072	3,940	3,859	3,782	3,574	3,498	3,487	3,422	3,095	3,263	3,677	3,631
Meter14	East Orange	104.84	6,045	7,145	7,147	7,085	7,001	6,643	6,520	5,902	5,057	4,417	4,679	5,557	6,100
Meter15	South Orange	35.28	4,333	5,135	5,802	5,845	5,288	4,202	3,670	3,160	3,027	2,018	2,267	2,769	3,960
Meter16	Irvington	24.93	4,668	4,813	4,813	4,664	4,465	4,190	3,684	3,313	3,214	3,021	3,008	3,571	3,952
Meter17/17E	Newark	113.65	1,643	1,750	1,758	1,551	1,566	1,553	1,463	1,502	1,475	1,517	1,520	1,609	1,576
Meter18	Newark	114.7	9,012	10,545	10,929	10,949	10,949	8,024	6,642	6,034	4,628	4,953	5,441	7,003	7,926
Meter21	Maplewood	98.03	2,471	2,544	2,541	2,373	2,277	2,175	2,128	2,041	2,002	1,985	2,107	2,251	2,241
Meter22	Maplewood	53.65	2,227	2,216	2,329	2,261	2,064	2,012	1,913	1,773	1,706	1,798	1,919	2,157	2,031
Meter24	Summit	675.52	2,004	2,467	2,733	2,515	1,720	1,230	1,077	1,068	1,126	959	1,001	1,623	1,627
Meter25	Maplewood	119.54	3,137	3,449	3,781	3,802	3,410	2,706	1,561	1,141	1,276	1,377	1,570	2,917	2,511
Meter26/31	Maplewood	128.35	4,172	4,400	4,808	4,542	4,458	4,140	2,963	2,883	2,737	2,873	3,114	3,788	3,740
Meter27	South Orange	103.36	3,065	3,383	3,715	3,128	2,775	1,934	767	394	373	670	882	2,098	1,932
Meter28	South Orange	200.93	5,728	5,748	5,697	5,745	5,748	5,544	5,273	5,102	4,989	5,084	5,142	5,556	5,446
Meter29/30	West Orange	940.53	2,535	2,711	3,033	3,053	3,057	2,601	2,205	1,863	1,598	1,808	1,769	2,221	2,371
Meter32C	Millburn	207.7	1,727	2,069	2,093	1,985	1,627	1,180	920	904	904	1,039	1,183	1,693	1,444
Meter32D	Millburn	284.05	1,980	2,271	2,527	2,156	2,129	1,668	1,417	1,127	1,069	1,066	1,380	1,943	1,728
Meter32E	Millburn	102.3	5,378	6,441	6,516	6,180	5,066	3,673	2,864	2,815	2,815	3,236	3,684	5,271	4,495
Meter34	Hillside	31.66	2,518	2,527	2,527	2,527	2,503	2,525	2,598	2,579	2,601	2,508	2,502	2,454	2,531
Meter9	Irvington	203.369	3,417	3,700	3,807	3,858	3,562	3,124	2,914	2,783	2,774	2,670	2,683	3,067	3,197
Meter9A/9A-Up	Irvington	239.26	11,961	12,167	12,294	12,191	11,630	11,371	11,466	11,141	11,118	11,893	11,946	12,057	11,770
Composite [2]	All	5281.709	3,270	3,564	3,738	3,674	3,413	2,947	2,596	2,346	2,240	2,311	2,447	2,957	2,959

1. Tabulated from SSES Phase II-A Reports.
2. Infiltration was weighted using each metershed's IDM.
3. Values highlighted exceed N.J.A.C's 1,500 gpd/IDK "nonexcessive infiltration" threshold.



Notes:
1. Outliers excluded from plots.
2. 25% and 75% quartiles were used for plotting purposes.
3. Mean values in each plot do not represent composite ADWF's described above, rather they represent the average of each dataset using equal weighting regardless of population of metershed/municipality.

Figure 7-36: Infiltration per IDM of JMEUC Metersheds, MDC Pilot Study Areas, and MWRA Municipalities

7.7.4 JMEUC Inflow Analysis

As was the case with infiltration, JMEUC used Typical Year model output from the calibrated Baseline Merged Model in InfoWorks ICM to develop normalized inflow rates and volumes on a metershed basis. Inflow rates were compared to the “non-excessive inflow” rate of 275 gpcd defined in the N.J.A.C.

“Non-excessive inflow” is defined by N.J.A.C as follows:

“Non-excessive inflow” - The maximum total flow rate during storm events which does not result in chronic operational problems related to hydraulic overloading of the treatment works or which does not result in a total flow of more than 275 gallons per capita per day (domestic base flow plus infiltration plus inflow) during a significant rainfall event which causes surface ponding and surface runoff. Chronic operational problems may include surcharging, backups, bypasses, and overflows.

The definition of “Excessive inflow/infiltration” found in the N.J.A.C is included above in Section 7.7.3.

In addition to comparing inflow rates to N.J.A.C definitions, JMEUC inflow rates and volumes were also compared to inflow rates and volumes of Cincinnati MSD, Hartford MDC, and the MWRA systems. Cincinnati MSD inflow rates and volumes were obtained directly from calibrated model output, while inflow volumes for the MWRA needed to be normalized by precipitation to account for differences in rainfall volumes between Typical Year rainfall used by JMEUC, and rainfall recorded during the years inflow was analyzed for MWRA’s member municipalities (2006 and 2017). Inflow volumes for Hartford MDC were limited to only spring data and as a result, had to be extrapolated over the entire year to obtain estimated annual inflow volumes. During the extrapolation process, inflow was adjusted to account for seasonality typically observed in rainfall-derived inflow. The normalization, extrapolation, and adjustments mentioned above are described in the footnotes of Table 7-53.

7.7.4.1 JMEUC Peak Inflow Analysis

Table 7-57 presents simulated peak gpcd values (domestic base flow + infiltration + inflow) for 10 rainfall events occurring during the Typical Year on a metershed basis for the JMEUC system. 23 of the 25 metersheds exceed the 275 gpcd non-excessive threshold defined within the N.J.A.C at some point during the Typical Year. Table 7-57 also includes the total hours and percent of time during the Typical Year that flows from individual metersheds are predicted to exceed 275 gpcd. In general, most metersheds exceed 275 gpcd less than 1% of the time over the course of the Typical Year. Metershed Meter34 is a relatively small metershed with significant industrial activity which can explain the relatively large peak flows observed on a per capita basis.

As was done in Table 7-56 and Table 7-57 for infiltration, composite flows were calculated for JMEUC’s system using population for each metershed as a weighting factor. This was done to better quantify peak inflows from a systemwide standpoint. Furthermore, to account for flow attenuation through the 43 miles of trunk sewer in the JMEUC system, simulated flows through the North and South twin barrel trunk sewers directly upstream of TAPS were determined. These flows excluded inflow from the Elmora (combined) and DOT (storm) subcatchments so that only inflow from separate upstream member municipalities was accounted for. Table 7-56 indicates that JMEUC’s twin trunk sewer system significantly attenuates flow from upstream member municipalities which decreases peak flows near the TAPS and at the WWTF. In total, there are five rainfall events during the typical year which cause total flow through the North and South twin barrel trunks to exceed 275 gpcd, totaling 30 hours or 0.34%of the Typical Year.

Comparison of the JMEUC peak inflow rates on a per capita basis to those of other sewerage authorities was not possible during analysis due to the lack of readily available population data from Cincinnati MSD. Additionally, Hartford MDC and MWRA inflow data was only reported on a volumetric basis. To directly compare inflow rates between JMEUC and Cincinnati MSD, wet weather peaking factors (peak flow/ADWF) were calculated from calibrated model output. Both the calibrated Baseline Merged Model (JMEUC) and Cincinnati MSD models were run using Typical Year rainfall data to provide a direct comparison between the two systems. Figure 7-36 shows box and whisker plots of the two systems which summarize the maximum peaking factors predicted by the calibrated models using Typical Year rainfall as model input. The mean peaking factor of the JMEUC metersheds is 50% smaller than the mean peaking factor of the analyzed Cincinnati MSD metersheds. To account for sewered area discrepancy between JMEUC metersheds and Cincinnati MSD metersheds (average sewered area for JMEUC metersheds = 1272 acres, average sewered area for Cincinnati MSD metersheds = 377 acres), JMEUC's largest metersheds (Meter05/05A, Meter06, Meter24, Meter29/30, Meter32C, Meter32D, Meter9A/9A-Up) were removed and the box and whisker plots were regenerated (Figure 7-37). Elimination of JMEUC's largest metersheds decreased the mean sewered area from 1,272 acres to 423 acres, but had virtually no impact on the peaking factor distribution of the JMEUC system, as can be seen comparing Figure 7-36 and Figure 7-37.

While peak inflow data from other sewerage authorities was limited, Figure 7-36 and Figure 7-37 indicate wet weather peaking factors from the JMEUC upstream metersheds are generally smaller than those of Cincinnati MSD metersheds. Additionally, while peak flows on a per capita basis are generally larger than the 275 gpcd non-excessive inflow threshold defined in the N.J.A.C, attenuation of flow through JMEUC's 43 miles of trunk sewer decrease peak flows significantly prior to reaching both the TAPS force main and WWTF.

7.7.4.2 JMEUC Inflow Volume Analysis

To determine if the existing inflow volumes in the JMEUC service area could be deemed excessive, comparisons to annual inflow volumes of other sewerage authorities were generated. Figure 7-38 summarizes annual inflow volumes normalized by IDM on a metershed basis for JMEUC, Hartford MDC, and Cincinnati MSD, and on a municipality basis for MWRA. Figure 7-38 indicates JMEUC inflow volumes are significantly less than Hartford MDC (both pre- and post-I/I reduction) and Cincinnati MSD inflow volumes, and are generally in line with MWRA (both pre- and post-I/I reduction) inflow volumes.

Figure 7-39 summarizes annual inflow volumes normalized by sewered area on a metershed basis for JMEUC, Cincinnati MSD, and Hartford MDC systems. MWRA inflows are not included due to the lack of information pertaining to sewered areas for MWRA member municipalities. Figure 7-36 indicates that on a sewered area basis, JMEUC inflow volumes are less than Harford MDC (pre- and post-I/I reduction) inflow volumes, and in line with Cincinnati MSD inflow volumes. Discrepancies in inflow volume distributions between Figure 7-38 and Figure 7-39 can be explained by population (and pipe network) density. A more sparsely populated area will have fewer pipes and a smaller IDM on a per area basis. These plots indicate that JMEUC member municipalities have a greater population density than both Cincinnati MSD and Hartford MDC areas included in this analysis.

7.7.5 I/I Reduction Cost Analysis

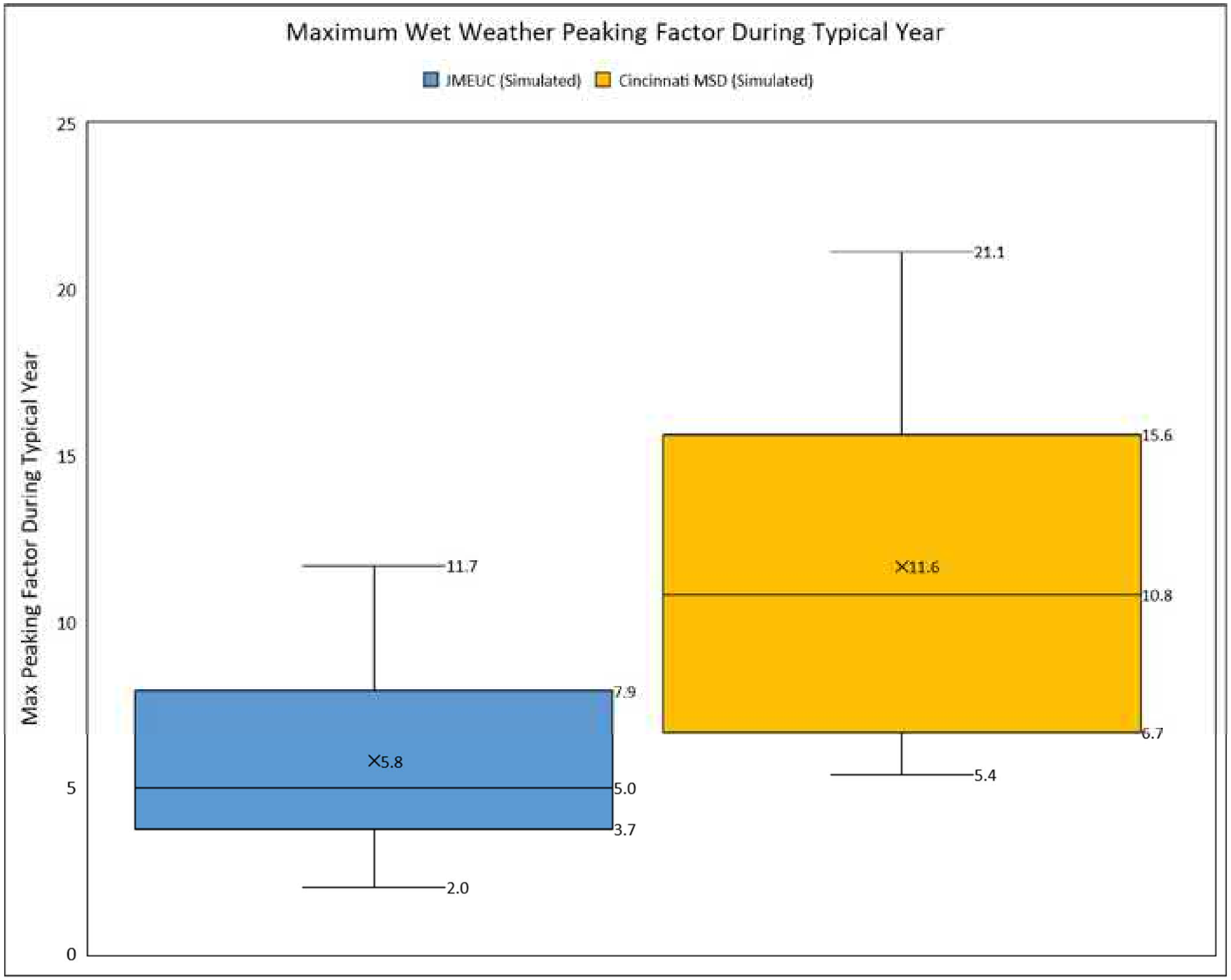
A planning-level cost analysis was performed to estimate the potential costs associated with additional I/I removal from JMEUC's member municipalities. As discussed in Section7.7.2, a maximum attainable reduction in I/I volume of 50% from baseline conditions (no previous I/I removal) conditions was used to estimate I/I reduction benefits. As noted earlier, national experience with comprehensive I/I reduction on a system-wide level indicates this is a reasonable upper limit assumption for estimating I/I reduction benefits, and the results of the MWRA and Hartford MDC programs confirm that higher reduction levels cannot be assumed to evaluate I/I reduction for JMEUC's system. This cost analysis was also conservative (i.e. maximizes the benefit/cost ratio for I/I reduction) in that the cost analysis only priced in CIPP lining of sewer mains and laterals. More detailed definition of I/I reduction program elements could likely include additional work in the form of manhole rehabilitation and lining, downspout and sump pump redirection, and elimination of cross connections would likely need to be performed in some locations.

To begin the cost analysis, a review of the inflow reduction already achieved on a municipality basis was completed. It was assumed at this step for both costing and modeling purposes that any additional reduction in inflow (RDII) would result in an equal percent reduction in infiltration (GWI). Municipalities

Table 7-57: Peak Total Flow Rates on a Per Capita Basis for JMEUC Metersheds

Metershed	Municipality	Hours during Typical Year that flows exceed 275 gpcd	Percent of time during Typical Year that flows exceed 275 gpcd	Peak Typical Year flow rate (gpcd)	2/6/2004	4/12/2004	4/25/2004	5/12/2004	7/12/2004	7/23/2004	9/8/2004	9/18/2004	9/28/2004	11/28/2004
Meter04	Roselle Park	133	1.51%	871	632	499	434	**871**	268	373	527	821	519	682
Meter05/05A	Union	141	1.61%	764	**764**	444	423	738	232	276	365	383	325	408
Meter06	Hillside	38	0.43%	357	338	299	260	327	268	259	350	312	**357**	329
Meter10	Newark	28	0.31%	431	390	275	250	422	210	274	320	**431**	360	388
Meter12	Newark	11	0.13%	507	256	276	242	**507**	224	309	332	464	289	349
Meter13	East Orange	6	0.07%	436	322	221	204	**436**	201	248	243	281	239	314
Meter14	East Orange	0	0.00%	236	205	187	184	236	187	192	199	210	222	207
Meter15	South Orange	108	1.24%	398	361	383	**398**	340	232	323	289	320	306	381
Meter16	Irvington	96	1.10%	857	434	391	303	750	374	377	606	**857**	598	687
Meter17/17E	Newark	1	0.01%	345	218	180	153	**345**	122	136	153	214	130	192
Meter18	Newark	0	0.00%	217	171	143	137	141	137	148	138	121	163	217
Meter21	Maplewood	14	0.16%	360	269	205	195	**360**	167	223	293	282	354	319
Meter22	Maplewood	44	0.51%	762	409	372	317	**762**	258	306	456	490	409	503
Meter24	Summit/New Providence	93	1.06%	502	346	329	326	302	231	265	387	339	**502**	305
Meter25	Maplewood	11	0.13%	397	301	287	278	**397**	158	161	219	181	241	203
Meter26/31	Maplewood	342	3.90%	745	652	543	524	**745**	400	414	633	550	656	670
Meter27	South Orange	142	1.62%	396	357	304	302	300	271	273	361	312	**396**	342
Meter28	South Orange	744	8.50%	846	**846**	650	616	598	312	376	376	451	494	467
Meter29/30	West Orange/Orange	19	0.21%	448	266	214	180	231	203	197	303	289	**448**	311
Meter32C	Millburn	63	0.72%	518	454	419	393	492	189	246	414	294	**518**	353
Meter32D	Millburn/Livingston	1502	17.14%	1025	965	630	612	858	468	652	683	641	**1025**	734
Meter32E	Millburn	163	1.86%	490	**490**	452	430	479	218	259	379	269	438	358
Meter34	Hillside	6498	74.18%	1434	632	942	790	**1434**	499	669	901	975	782	1009
Meter9	Irvington	3	0.03%	470	289	245	249	**470**	167	222	310	401	229	340
Meter9A/9A-Up	Irvington	90	1.02%	1245	481	543	520	**1245**	361	495	763	1120	510	872
Composite [1]	All	NA	NA	604	435	348	326	**532**	237	280	381	428	392	413
Downstream Twin Barrel Trunk Sewer [2]	All	30	0.34%	371	**371**	269	287	309	214	230	294	243	340	279

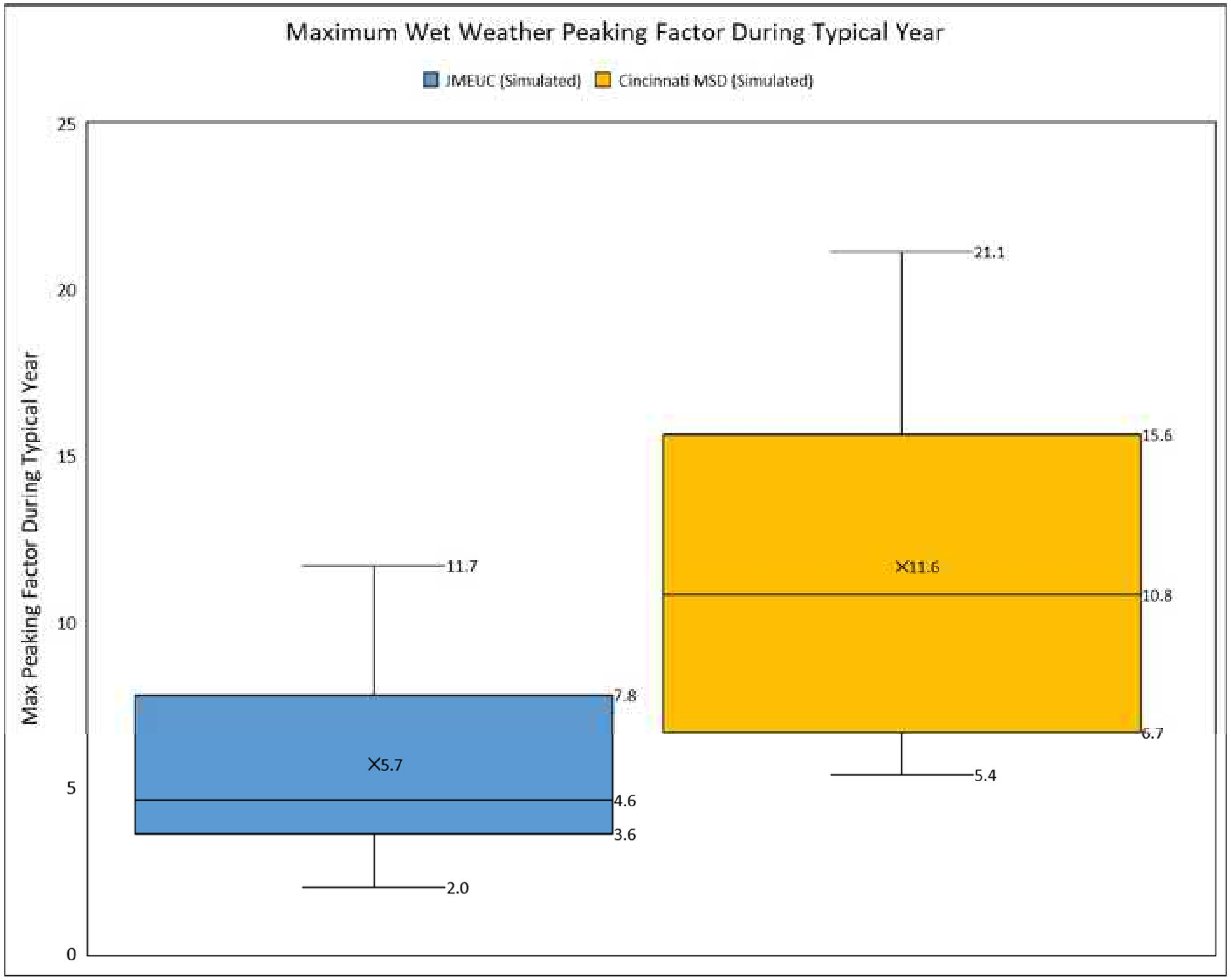
1. Peak flow rates weighted using each metersheds population.
2. Simulated peak flows through North and South Barrels directly upstream of the TAPS. Flows from Elmora and DOT subcatchments were removed to only include separate sewer areas.
3. Values highlighted exceed N.J.A.C's 275 gcpd "nonexcessive inflow" threshold. **Bold** values indicate peak flow during Typical Year.



Notes:

1. Outliers excluded from plots.
2. 25% and 75% quartiles were used for plotting purposes.
3. Mean values in each plot do not represent composite ADWF's described above, rather they represent the average of each dataset using equal weighting regardless of population of metershed/municipality.

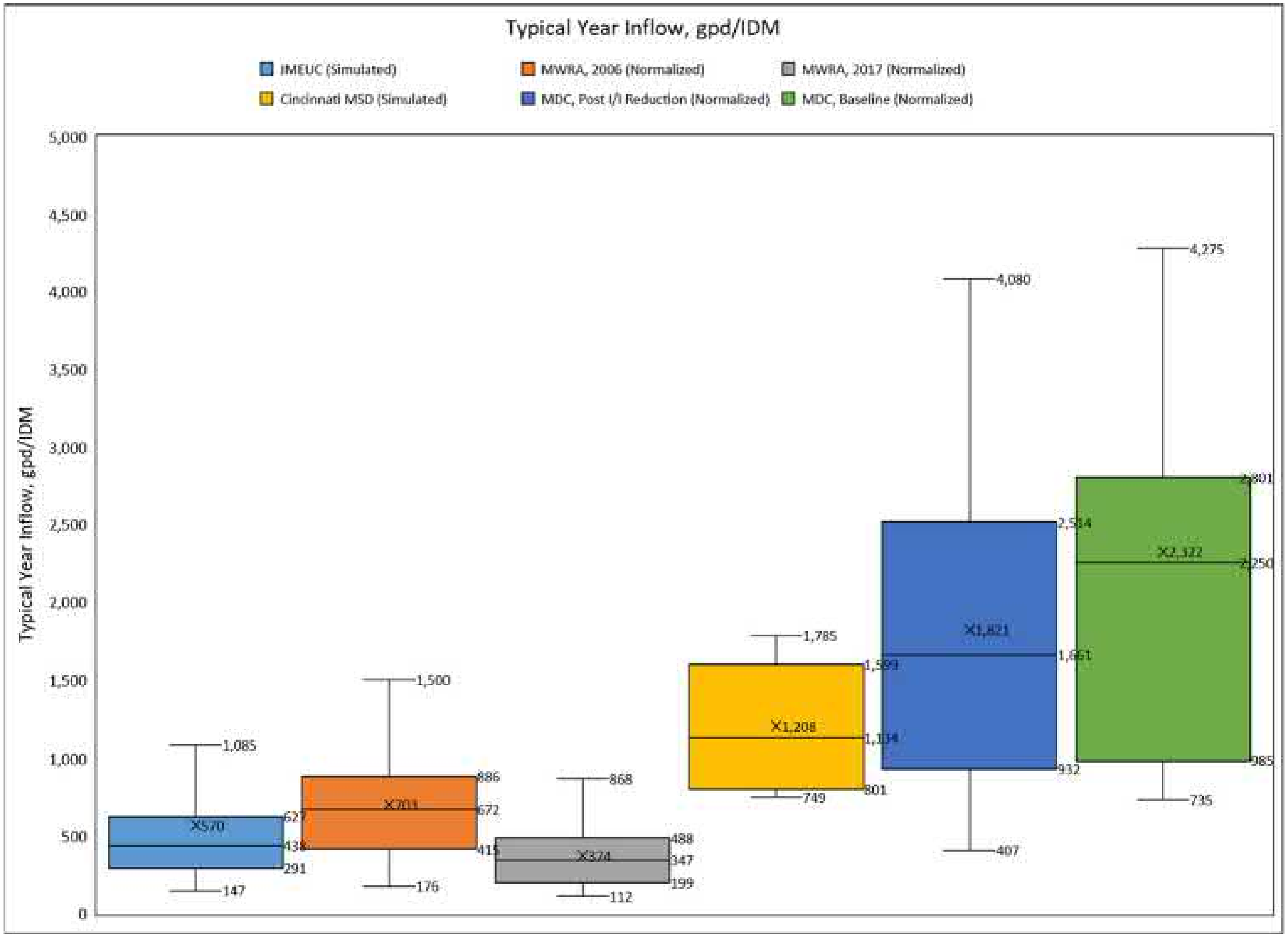
Figure 7-37: Peaking Factors of JMEUC and Cincinnati MSD Metersheds During Typical Year



Notes:

1. Outliers excluded from plots.
2. 25% and 75% quartiles were used for plotting purposes.
3. Mean values in each plot do not represent composite ADWF's described above, rather they represent the average of each dataset using equal weighting regardless of population of metershed/municipality.

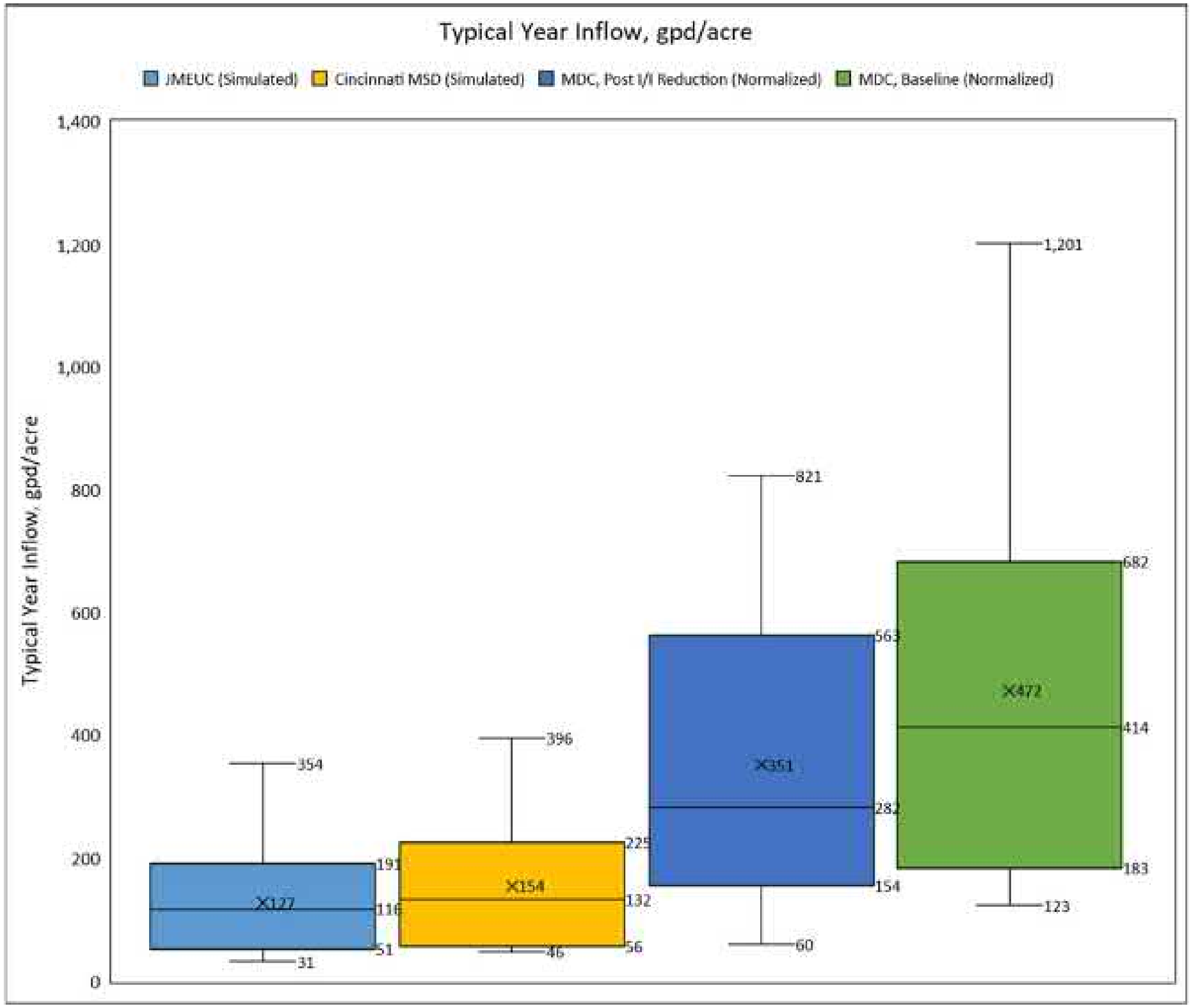
Figure 7-38: Peaking Factors of JMEUC and Cincinnati MSD Metersheds During Typical Year Excluding JMEUC's 7 Largest Metersheds



Notes:

1. Outliers excluded from plots.
2. 25% and 75% quartiles were used for plotting purposes.
3. Mean values in each plot do not represent composite ADWF values described above, rather they represent the average of each dataset using equal weighting regardless of population of metershed/municipality.

Figure 7-39: JMEUC, MWRA, Hartford MDC, and Cincinnati MSD Typical Year Inflow Volumes on IDM Basis



Notes:

1. Outliers excluded from plots.
2. 25% and 75% quartiles were used for plotting purposes.
3. Mean values in each plot do not represent composite ADWF values described above, rather they represent the average of each dataset using equal weighting regardless of population of metershed/municipality.

Figure 7-40: JMEUC, MWRA, Hartford MDC, and Cincinnati MSD Typical Year Inflow Volumes on an Area Basis

were assigned the percentage of inflow reduction already achieved as reported in JMEUC's 2018 Annual Assessment Report (Table 7-51). For example, the community of West Orange has removed a reported 10% of inflow from their system since I/I reduction efforts began in the 1980's. A select few municipalities have actually removed more than 50% of inflow from their systems (Maplewood, East Orange, Summit). For these municipalities, it was assumed that no further I/I reduction could be achieved. The high (>50%) I/I reductions in these areas may be explained by the initial levels of I/I. It is likely in a system such as JMEUC's that some upstream areas may benefit significantly from I/I remediation from a percentage reduction standpoint due to pre-existing excessive I/I. Diversely, there may be other portions of the system which have limited I/I, where even extensive I/I remediation would have a limited impact on removal percentage. It is likely that Maplewood, East Orange, and Summit experienced significant RDII rates prior to any reduction efforts, and that the removal of inflow sources had a significant impact from a percentage reduction standpoint.

After assigning existing inflow reduction percentages to each municipality, the percent of additional reduction necessary to achieve 50% reduction from the baseline year (1983) was determined. Table 7-58 summarizes the existing inflow reduction already achieved, and the inflow reduction for each municipality still necessary in order to achieve a 50% reduction in inflow.

Table 7-58: Estimated Percent of Additional Inflow Reduction Still Attainable for JMEUC Municipalities

Municipality	% of Inflow Already Removed (2018 Annual Report)	% of Additional Inflow Reduction Needed to Achieve 50% Reduction from Baseline Year (1983) [1]
Union	43%	12.28%
Roselle Park	0%	50.00%
Irvington	21%	36.71%
South Orange	16%	40.48%
Newark	2%	48.98%
East Orange	93%	0.00%
Hillside	0%	50.00%
Maplewood	65%	0.00%
Millburn	15%	41.18%
Summit	56%	0.00%
West Orange	10%	44.44%

1. Calculated using the following equation: % Of Additional Inflow Reduction = 1-0.5/(1-% of Inflow Already Removed)

Because most of the municipalities listed in Table 7-58 are made up of multiple metersheds, assumptions needed to be made regarding where existing I/I reduction had already taken place. This was necessary in order to accurately model and cost any future I/I reduction. For municipalities with either 0% reduction or greater than 50% reduction, this exercise was straight forward. For municipalities having between 0% and 50% existing inflow reduction and multiple metersheds, assumptions on which metershed(s) existing I/I reduction had already taken place in were necessary. Municipalities which fell into this category were Irvington, Millburn, Newark, and South Orange. Using Typical Year Baseline Merged Model output, inflow volumes on a per area basis were calculated for each metershed within these municipalities. It was assumed that the metershed(s) with the smallest inflow volume per area had undergone the already achieved I/I reduction associated with the municipality. For example, Irvington, made up of three metersheds, has reduced inflow by 21%. Of Irvington's three metersheds, Metershed Meter 9A/9A-Up has the smallest observed Inflow volume during the Typical Year on a per area basis (49,341 gallons/acre). Because of this, it was assumed all inflow reduction in Irvington occurred in this metershed, while the remaining two metersheds (Meter9 and Meter16) have not experienced any inflow reduction. By assuming no inflow reduction in these two metersheds, it fixes their attainable inflow reduction at 50%, allowing an iterative calculation to determine the percent reduction in inflow for metershed Meter9A/9A-Up necessary to achieve an inflow reduction of 50% from a municipality standpoint. Table 7-59 summarizes the calculations described above. Following completion of this step, estimated attainable inflow reduction percentages for all metersheds were known. Using these percentages, the percent of each metershed needing to undergo comprehensive I/I reduction was determined. For this analysis comprehensive I/I reduction is defined as CIPP lining of the public sewers and service laterals. Total sewer main lengths in each metershed were known from previous SSES Phase II-A studies, while dwelling counts (and their laterals) were known on a municipality level from JMEUC's 2018 Annual Assessment Report. Dwelling counts were divided among metersheds using each metershed's area and the assumption of a uniform dwelling density throughout each municipality. Multiplying the total sewer main length and dwelling count of each metershed by the percent of the metershed needing to undergo inflow reduction in the form of CIPP lining provided the total length of sewer main and number of laterals to be lined.

Table 7-59: Estimated Attainable % Reduction in Inflow for Metersheds Within Municipalities with Multiple Metersheds and Reported Inflow Reduction of Between 0% and 50%

Metershed/ Location	Municipality	Municipality's existing Typical Year Inflow Volume (MG)	Municipality's % Attainable I/I Reduction	Municipality's Inflow Volume with I/I Reduction (MG)	Typical Year Inflow Volume (MG)	% Inflow Reduction Attainable	Target Typical Year Inflow Volume (Gal/Acre)
Meter16 [1]	Irvington	116.91	36.71%	73.99	10.15	50.00%	5.08
Meter9 [1]					43.98	50.00%	21.99
Meter9A/9A-Up [2]					62.77	25.25%	46.92
Meter32D [1]	Millburn	56.83	41.18%	33.43	35.73	50.00%	17.86
Meter32C [2]					14.99	26.25%	11.05
Meter32E [2]					6.12	26.25%	4.51
Meter10 [1]	Newark	45.83	48.98%	23.38	8.79	50.00%	4.39
Meter17/17E [1]					20.89	50.00%	10.44
Meter12 [1]					3.43	50.00%	1.72
Meter18 [2]					12.73	46.30%	6.83
Meter27 [1]	South Orange	109.10	40.48%	64.94	44.83	50.00%	22.41
Meter28 [2]					57.43	33.85%	37.99
Meter15 [2]					6.84	33.85%	4.53

1. Assumption made that no inflow reduction has been completed in metershed
2. % Inflow Reduction Attainable calculated iteratively so that Municipality Inflow Volume with I/I Reduction equals sum of Target Typical Year Inflow Volume for metersheds making up Municipality

Cost estimates for the required I/I remediation programs to achieve the target (50%) reduction levels were generated using the data described above. Unit costs for CIPP lining of sewer mains and laterals were obtained from the Hartford MDC study (see Section 7.7.2.2 above). Applying the unit cost of $35 per linear foot for the lining of sewer mains, and the cost of $7,000 each to line a lateral produced the total cost of I/I removal for each metershed. The calibrated Baseline Merged Model was used to estimate I/I volume reductions (both RDII and GWI) by simulating the Typical Year I/I volumes using the 50% I/I reduction factors. A measurement of cost effectiveness for I/I removal (gallons I/I removed per $ cost incurred) was then determined for each metershed and Table 7-61 presents these results in tabular format.

Table 7-60: Peak Inflow Reduction During Typical Year WWF Events at WWTF

Event	Peak WWTF Influent Rate (mgd) With Existing I/I	Peak WWTF Influent Rate (mgd) With Attainable I/I	Difference (mgd)	% Reduction
2/6/2004	162.72	140.82	21.90	13.46%
4/12/2004	125.22	108.95	16.27	12.99%
4/25/2004	125.95	110.75	15.20	12.07%
5/12/2004	142.26	122.60	19.66	13.82%
7/12/2004	108.29	95.34	12.95	11.96%
7/23/2004	113.24	99.67	13.57	11.98%
9/8/2004	137.05	118.00	19.06	13.91%
9/18/2004	117.91	102.41	15.51	13.15%
9/28/2004	160.50	139.12	21.38	13.32%
11/28/2004	128.05	111.08	16.96	13.25%

As shown on Table 7-61, the projected cost effectiveness of I/I removal varies significantly across the metersheds in the JMEUC service area. The metersheds were therefore ranked by cost effectiveness and the results plotted to create a cumulative cost curve as shown on Figure 7-41. If I/I removal was implemented on a metershed basis sequentially across the service area in order of cost effectiveness, the cost curve shows the I/I reduction volumes that would be achieved as a function of incurred cost. It should be noted, however, that the actual I/I reductions that would be achieved with a comprehensive I/I reduction program are difficult to predict. National experience has shown that the actual reduction levels achieved are often very different than those predicted, and often significantly less than predicted, due to the uncertainties inherent the actual local sources of I/I and the various factors influencing the occurrence of I/I in the specific project areas.

7.7.6 Impact of I/I Reduction on JMEUC System Flows

To assess the impact that I/I reduction would have on JMEUC system performance, the percent reduction values calculated during cost analysis were applied to the R values (RDII) and infiltration rates (GWI) in the calibrated Baseline Merged Model in InfoWorks ICM for each metershed. The model was then used to simulate the Typical Year rainfall with the reduced I/I model parameters. Model results for I/I volumes were compared to the Typical Year results for the existing I/I conditions to determine the impact of system-wide I/I reduction on both peak flow rates and flow volumes near the TAPS and at the WWTF. The reduction of peak flow rates near the TAPS and at the WWTF offers the potential to increase flow rates pumped at the TAPS, while a reduction in peak hourly flow rates and volume to the WWTF offers the potential to minimize any necessary future WWTF expansion.

Model results indicate that reducing I/I to attainable levels throughout JMEUC's member municipalities without any changes to the TAPS capacity would reduce Typical Year flow volumes though the WWTF by 9% (from 21,700 MG to 19,750 MG, or 1950 MG). The majority of this reduction in volume (1,650 MG) would be observed during DWF due to removal of groundwater infiltration (GWI) not directly influenced by rainfall. The remaining reduction in Typical Year WWTF volume (300 MG) would be associated with RDII.

Table 7-60 shows the predicted reduction in peak inflow (peak hourly rates) to the WWTF during the largest rainfall events in the Typical Year. Generally, attainable I/I reduction is predicted to reduce peak influent flow rate to the WWTF by between 12%-14%, with the largest absolute differences (21-22 mgd) occurring during the largest flow events (2/6/2004 and 9/28/2004).

7.7.7 Cost Effectiveness of RDII Reduction for CSO Control

As shown on Table 7-60 above, the largest reduction in peak hourly flow rate at the WWTF during the Typical Year that would be achieved with full system I/I reduction is 22 mgd. This represents the greatest attainable reduction in peak flow rate at the WWTF with full implementation of RDII reduction across the entire JMEUC sanitary sewer service area. This also assumes the predicted I/I reduction levels would actually be achieved, and as noted earlier, national experience has shown that there is considerable risk that the I/I reduction improvements could under-perform versus those levels.

As shown on Table 7-61, the cost to achieve the predicted level of RDII reduction is estimated to be almost $600 million. As noted previously in this section, the primary benefit to reducing I/I rates would be to reduce the capacity of additional facilities that would be constructed to provide treatment of additional flows from an expanded TAPS pump station and new force main, as described in Section 7.3. Those potential CSO treatment facilities would be sized to provide up to 85 mgd in treatment capacity and are estimated to cost on the order of $16-47 million to construct. Therefore I/I reduction would potentially reduce the sizing of those facilities by 22 mgd, or about 25%. If the cost of those facilities is reduced proportionally, the savings would be on the order of $4-12 million. Given that the system-wide I/I reduction costs required to achieve that cost savings would be at least 50 times greater than the savings, it is clear that system-wide I/I reduction is not a cost-effective approach for CSO control in the JMEUC service area. Even if only Metershed 28 where I/I reduction is most cost effective was addressed, the $16 million cost of I/I reduction in that metershed may be as costly as the proposed WWTF expansion cost itself, and the cost reduction would be a small fraction of the 25% reduction for the full system.

While RDII is not cost effective as a CSO control approach in the JMEUC service area, JMEUC will continue to encourage its members and customers to implement RDII reduction under existing JMEUC programs. Any further reductions in RDII that are achieved through these programs will serve to reduce wet weather operating costs at the WWTF and improve performance during extremely large storm events (larger than the largest events in the Typical Year).

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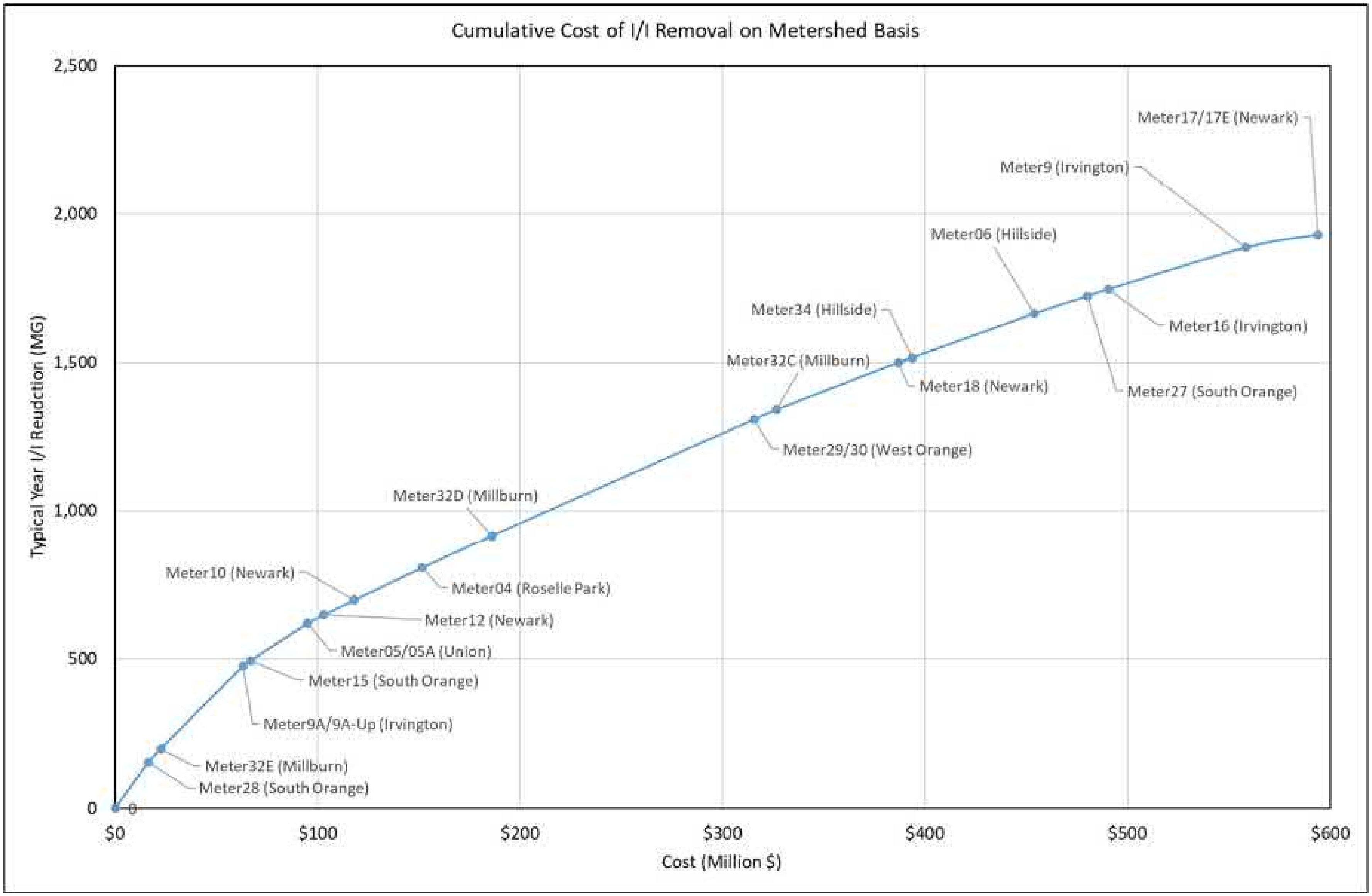


Figure 7-41: Cumulative Cost Effectiveness of I/I Removal on Metershed Basis

Table 7-61: Cost Summary Table for I/I Reduction for JMEUC on Metershed Basis

Metershed	Municipality	% Reduction In Inflow Necessary to Achieve 50% Reduction for Associated Municipality	% of Metershed Needing to Undergo CIPP Lining to Achieve Required Inflow Reduction	Sewer Main Length in Metershed(lf) [5]	Estimated Sewer Main Length to be CIPP Lined to Achieve Inflow Reduction % (lf)	Estimated Dwellings In Metershed [4]	Estimated Dwellings In Metershed To Be Lined to Achieve Inflow Reduction %	Estimated Cost of CIPP Lining Sewer Mains [1]	Estimated Cost of CIPP Lining Laterals [2]	Total Estimated CIPP Lining Cost	Estimated Inflow During Typical Year (MG) [3]	Estimated Attainable Inflow Reduction During Typical Year (MG)	Estimated Infiltration During Typical Year (MG) [3]	Estimated Attainable Infiltration Reduction During Typical Year (MG)	Estimated Attainable I/I Reduction During Typical Year (MG)	Estimated gallons of RDII removed per $ spent
Meter16	Irvington	50.00%	100.00%	15,722	15,722	1,398	1,398	$550,270	$9,788,630	$10,338,900	10.15	5.08	35.92	17.96	23.03	2.23
Meter05/05A	Union	12.28%	14.00%	644,418	90,213	25,109	3,515	$3,157,443	$24,605,217	$27,762,660	524.91	64.46	483.09	59.32	123.78	4.46
Meter04	Roselle Park	50.00%	100.00%	18,237	18,237	4,752	4,752	$638,295	$33,264,000	$33,902,295	60.30	30.15	160.90	80.45	110.60	3.26
Meter27	South Orange	50.00%	100.00%	67,823	67,823	3,400	3,400	$2,373,805	$23,798,412	$26,172,217	44.83	22.41	72.58	36.29	58.71	2.24
Meter9	Irvington	50.00%	100.00%	130,642	130,642	9,039	9,039	$4,572,470	$63,269,685	$67,842,155	43.98	21.99	237.04	118.52	140.51	2.07
Meter13	East Orange	0.00%	0.00%	46,038	0	2,759	0	$0	$0	$0	12.86	0.00	102.10	0.00	0.00	NA
Meter22	Maplewood	0.00%	0.00%	35,368	0	1,528	0	$0	$0	$0	12.01	0.00	39.76	0.00	0.00	NA
Meter28	South Orange	33.85%	51.17%	131,757	67,422	3,940	2,016	$2,359,775	$14,113,118	$16,472,893	57.43	19.44	399.33	135.17	154.61	9.39
Meter9A/9A-Up	Irvington	25.25%	33.78%	150,174	50,728	16,459	5,560	$1,775,469	$38,918,335	$40,693,804	62.77	15.85	1027.65	259.48	275.33	6.77
Meter10	Newark	50.00%	100.00%	27,454	27,454	1,991	1,991	$960,890	$13,934,851	$14,895,741	8.79	4.39	91.08	45.54	49.93	3.35
Meter17/17E	Newark	50.00%	100.00%	71,028	71,028	4,706	4,706	$2,485,980	$32,943,284	$35,429,264	20.89	10.44	65.33	32.66	43.11	1.22
Meter12	Newark	50.00%	100.00%	12,652	12,652	1,104	1,104	$442,820	$7,731,462	$8,174,282	3.43	1.72	53.93	26.96	28.68	3.51
Meter15	South Orange	33.85%	51.17%	23,278	11,912	972	498	$416,910	$3,482,531	$3,899,441	6.84	2.32	50.88	17.22	19.54	5.01
Meter21	Maplewood	0.00%	0.00%	62,675	0	3,582	0	$0	$0	$0	18.32	0.00	80.13	0.00	0.00	NA
Meter06	Hillside	50.00%	100.00%	181,685	181,685	7,700	7,700	$6,358,975	$53,899,930	$60,258,905	45.43	22.72	250.46	125.23	147.94	2.46
Meter26/31	Maplewood	0.00%	0.00%	83,512	0	1,974	0	$0	$0	$0	18.93	0.00	175.00	0.00	0.00	NA
Meter18	Newark	46.30%	86.22%	74,528	64,258	9,626	8,299	$2,249,025	$58,094,346	$60,343,371	12.73	5.89	331.22	153.35	159.25	2.64
Meter34	Hillside	50.00%	100.00%	19,475	19,475	865	865	$681,625	$6,055,070	$6,736,695	5.19	2.60	29.24	14.62	17.22	2.56
Meter14	East Orange	0.00%	0.00%	58,637	0	4,456	0	$0	$0	$0	6.48	0.00	233.19	0.00	0.00	NA
Meter32D	Millburn	50.00%	100.00%	184,553	184,553	3,966	3,966	$6,459,355	$27,762,427	$34,221,782	35.73	17.86	178.85	89.43	107.29	3.14
Meter25	Maplewood	0.00%	0.00%	77,294	0	2,484	0	$0	$0	$0	8.02	0.00	109.29	0.00	0.00	NA
Meter24	Summit	0.00%	0.00%	321,745	0	14,385	0	$0	$0	$0	107.24	0.00	399.83	0.00	0.00	NA
Meter29/30	West Orange	44.44%	79.99%	588,882	471,021	20,179	16,140	$16,485,728	$112,982,061	$129,467,789	69.06	30.69	813.52	361.53	392.22	3.03
Meter32C	Millburn	26.25%	35.59%	135,812	48,340	3,755	1,336	$1,691,895	$9,355,270	$11,047,165	14.99	3.93	109.18	28.66	32.59	2.95
Meter32E	Millburn	26.25%	35.59%	65,923	23,464	2,114	752	$821,244	$5,267,341	$6,088,585	6.12	1.61	167.41	43.94	45.55	7.48
Total				**3,229,312**	**1,556,628**	**152,243**	**77,038**	**$54,481,973**	**$539,265,968**	**$593,747,942**	**1,217**	**284**	**5,697**	**1,646**	**1,930**	

1. Unit cost of $35/lf from Hartford MDC 2016 Sewer Rehabilitation Pilot Study
2. Unit cost of $7,000/lateral from Hartford MDC 2016 Sewer Rehabilitation Pilot Study
3. Volume from Typical Year calibrated Baseline Merged Model
4. Dwelling counts from JMEUC's 2018 Annual Assessment Report
5. Sewer main length from SSES Phase II-A Reports

7.8 Consideration of Significant Indirect Users

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. JMEUC monitors SIUs for compliance with the pretreatment requirements.

Section 2 noted that of the eight (8) SIUs located in the City of Elizabeth, only three (3) of these facilities contribute flow to a sewer that is tributary to a CSO regulator / diversion structure, as tabulated in Table 7-62.

Table 7-62: Significant Indirect Users Discharging to Combined Sewer System

SIU Name Address Standard Industrial Class.	CSO Basin	Contributing Flow	Description
Mastercraft Metal Finishing 801 Magnolia Avenue 3471 Manufacturing of phonographic masters	039A	Process wastewater flow rate is approximately **80 gallons per day (gpd)**. Pre-treatment consists of chemical precipitation, filtration, neutralization and pH correction.	The facility electroplates vinyl record masters. The vinyl record masters are silver and nickel plated to form record stampers to make the production vinyl records.
Michael Foods, Inc. - Jersey Pride 1 Papetti Plaza 2015 - Egg processing	039A	Process wastewater flow rate is approximately **110,000 gpd**. Pre-treatment includes flow equalization, settled solids removal, neutralization and pH correction.	The egg processing performed at the site includes liquid-egg pasteurization, homogenization, storage, and distribution and hard cook eggs washing, boiling, peeling, and packaging.
Wakefern Food Corporation 600 York Street 5140 - Food Warehousing and distribution	002A	Reported average daily process wastewater flow rate is approximately **13,300 gpd**. Pre-treatment includes flow equalization, sedimentation, grease/sludge removal and pH neutralization.	The facility warehouses and distributes various food items to supermarkets and seafood cleaning/packaging.

The discharge from these three (3) SIUs were analyzed to assess whether, during overflow events, the discharge would negatively affect water quality, focusing on toxic metals and organics. Their discharge is sampled under existing industrial pretreatment permits issued by JMEUC. It is noted that the permits require only certain parameters to be measured, and Wakefern Food Corporation is required to measure only BOD and TSS and not any toxic parameters, so no quantitative analysis was conducted regarding this SIU. As such, the analysis was only completed for Mastercraft Metal Finishing and Michael Foods/Jersey Pride. Both of these SIUs discharge to Outfall 039A. The flow characteristics of this outfall are provided in Table 7-63 below:

Table 7-63: CSO 039A Flow Characteristics

CSO Basin 039A	Overflow statistics for Typical Year, 2015 Baseline
Number of overflows	35
Annual volume (MG)	32.35
Annual duration (hrs)	224
Average flow rate (mgd)	3.471

Sampling reports were provided by JMEUC for Mastercraft Metal Finishing and Michael Foods - Jersey Pride. The reports provided the average flow from the facilities and the average concentration for a series of contaminants measured as total concentration, i.e. dissolved plus particulate.

Based on the concentration and the discharge flow rate from each SIU, the annual mass load was calculated for each measured contaminant over the annual duration of overflow events at Outfall 039A. Under existing conditions for the Typical Year, the total duration of overflow events is 224 hours. The analysis conservatively assumed that 100% of the load during an overflow event was discharged through the overflow (i.e. no portion of the contaminants were conveyed to the treatment plant), which is the 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 is to assess the effect of the SIUs, concentrations in the combined sewer flow without SIUs was not considered. The results are presented in Table 7-64. All concentrations are very low, less than 0.011 mg/L, most less than 0.001 mg/L. This is attributable to dilution, as the average flow rate at this outfall location is approximately 27 times larger than the combined flow from the two SIUs (3.5 mgd vs. 0.13 mgd), per Table 7-65.

To assess whether the determined concentrations were problematic, they have been compared with 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. It is noted that some criteria were not indicated by EPA for the organic compounds and for some metals.

Table 7-64: Concentration of toxic contaminants attributable to SIUs in combined-sewer overflows

	Mastercraft Metal Finishing		Michael Foods – Jersey Pride				
Parameter	Average concentration dissolved + particulate, mg/L	Load, lb/yr	Average concentration dissolved + particulate, mg/L	Load, lb/yr	Combined Load, lb/yr	Average concentration from outfall, mg/L	EPA Aquatic Life Chronic Criteria, mg/L
METALS							
Aluminium		-	0.290	2.857	2.857	0.011	
Antimony		-	-	-	-	-	
Arsenic		-	-	-	-	-	0.036
Barium		-	0.040	0.394	0.394	0.001	
Beryllium		-	-	-	-	-	
Cadmium	0.011	0.000	-	-	0.000	0.000	0.001
Chromium	0.015	0.000	0.013	0.131	0.131	0.000	0.011
Cobalt		-	-	-	-	-	
Copper	0.024	0.000	-	-	0.000	0.000	0.003
Lead	0.022	0.000	-	-	0.000	0.000	0.003
Manganese		-	0.010	0.099	0.099	0.000	
Mercury		-	-	-	-	-	0.001
Molybdenum		-	-	-	-	-	
Nickel	0.592	0.008	0.010	0.099	0.107	0.000	0.008
Silver	0.014	0.000	-	-	0.000	0.000	0.002
Thallium		-	0.010	0.099	0.099	0.000	
Tin		-	-	-	-	-	
Titanium		-	0.010	0.099	0.099	0.000	
Zinc	0.016	0.000	0.080	0.788	0.788	0.003	0.081
NON-METALS		-		-			
Bis(2-ethylhexyl) phthalate		-	0.110	1.084	1.084	0.004	
Boron		-	0.080	0.788	0.788	0.003	
Bromodichloromethane		-	0.002	0.020	0.020	0.000	
BTEX		-	-	-	-	-	
Butlybenzylphthalate		-	0.001	0.010	0.010	0.000	

	Mastercraft Metal Finishing		Michael Foods – Jersey Pride				
Carbon tetrachloride		-	-	-	-	-	
Cyanide, Amenable	0.001	0.000	-	-	0.000	0.000	
Cyanide, total	0.003	0.000	-	-	0.000	0.000	
Dibromochloromethane		-	-	-	-	-	
Dichlorodifluoromethane		-	-	-	-	-	
Fluoride		-	0.060	0.591	0.591	0.002	
Hetachlor epoxide		-	-	-	-	-	
Pehnol		-	-	-	-	-	
Selenium		-	-	-	-	-	
Total Toxic Organics	0.065	0.001	0.150	1.478	1.479	0.005	
Trichlorofluoromethane		-	-	-	-	-	
Xylenes		-	-	-	-	-	

Table 7-65: SIU and Outfall Flow Comparison

	SIU			
	Mastercraft Metal Finishing	**Michael Foods, Inc. -- Jersey Pride**	**SIU Total**	**CSO 039A Total**
Average Flow (mgd)	0.000179	0.127095	0.13	3.5

Section 8
Public Participation Process Update

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 the City of Elizabeth's activities since the Public Participation Process report was submitted in June 2018 and revised in November 2018, with prior activities (including the first four meetings of the Supplemental CSO Team) documented in that report.

8.1 Supplemental CSO Team

In accordance with the NJPDES CSO Permits, the City of Elizabeth and JMEUC have continued to invite members of the affected public to participate in a Supplemental CSO Team, to solicit input and share information on the LTCP development process. While the initial meetings were primarily informative and educational in nature, the latter meetings have involved more participation and feedback from the team members on the CSO LTCP alternatives being considered. The meeting proceedings are summarized below.

8.1.1 Meeting No. 5

The fifth meeting, held on October 26, 2018 was attended by 13 individuals, of which seven were from the permittee team including Elizabeth, JMEUC and consultants, and six were stakeholder representatives from the other invited groups. At this meeting, the status of DEP review of LTCP submittals was presented, as well as an update on the public participation process, initial discussion of the development and evaluation of alternatives, and a presentation on treatment technologies used in the Bayonne Wet Weather demonstration project. Input on the CSO issues and public engagement was requested and the questions and comments from this meeting were as follows:

1. Team members were asked for input in identifying affected groups. The team suggested churches, community centers, senior centers, and the housing authority and also suggested that information could be included with the annual letter from the Mayor and could be sent to Shaping Elizabeth and the Groundworks Elizabeth Free Plant Sale. Information could also be included with emails from the Elizabeth Chamber of Commerce. It was suggested that an update be included in the monthly report to the JMEUC board and an insert be included with the annual recycling calendar mailed to every resident.

2. A team member recommended simplifying the information presented to make it more understandable to non-technical persons. Team members were provided with the handouts given to students at the Future City events, and it was agreed that these provided a simplified message.

3. A team member requested clarification that the objective of this project is to reduce the 56 overflows to 4 overflows per year. It was noted that the level of control required remains to be determined, but the reduction of overflow events to that range is one potential approach. A team member also asked where the wastewater treatment plant discharged to, and the project team responded that the treated waters flow to the Arthur Kill.

Feedback from the Supplemental CSO Team members was also solicited electronically through an interactive web-based survey application. Participants anonymously answered survey questions on a website using their mobile devices during the meeting and the poll results were presented in real-time. Incorporating these live polls was also an effective communication strategy as it encouraged CSO Team members to provide instant feedback and remain engaged throughout the meeting. The survey questions and responses were as follows:

Question / Possible Selections	Response Count
Which social media method would you suggest for effective LTCP messaging?	
City of Elizabeth Twitter feed	1
New Elizabeth/JMEUC CSO LTCP Twitter feed	0
Facebook	5
LinkedIn	0
City of Elizabeth & JMEUC website	1
Total	**7**
How would you like to review key draft submittals?	
Content and summaries presented at CSO Supplemental Team meeting presentations	4
Review full draft submittals	3
Review draft Executive Summary	2
Total	**9**
What are you most interested in discussing at upcoming meetings?	
CSO receiving water quality impacts	2
Approach to financial capability assessment	1
Green infrastructure analysis	4
Presumption vs. Demonstration approach	1
Other	0
Total	**8**

8.1.2 Meeting No. 6

The sixth meeting, held on January 30, 2019, was attended by 15 individuals, of which two were from NJDEP, eight were from the permittee team including Elizabeth, JMEUC and consultants, and five were stakeholder representatives from the other invited groups. At this meeting, the status of DEP review of LTCP submittals was presented, as well as an update on the public participation process. One of the team member organizations presented to the rest of the team on their background, objectives and interest in the LTCP process. This was followed by discussion of some of the alternatives that were being analyzed at that time, including maximizing wet weather treatment capacity at the plant, siting alternatives analysis, and green infrastructure analysis. Questions and comments from this meeting were as follows:

1. A team member requested a copy of the public informational flyer that was produced and requested that the digital copy be re-sent to their organization. NJDEP indicated that it would check whether they could circulate the flyer as well.

2. A team member suggested that the City could have a booth at a kickoff event for a new micro-farm conservation area.

3. A team member asked for further information on the locations of the Peripheral Ditch and Great Ditch, and the project team responded that the Peripheral Ditch is the drainage channel around Newark airport with one City of Elizabeth CSO outfall discharging to it, and the Great Ditch is an open channel and closed pipe drainage system extending from Dowd Avenue and across the

New Jersey Turnpike and the Jersey Gardens mall to Newark Bay, with 2 CSO outfalls discharging to it.

4. A team member asked what type of technologies are used for disinfection. The project team responded that peracetic acid or ultraviolet disinfection could be used, and the current process at the JMEUC plant is chlorination using sodium hypochlorite, followed by dechlorination to remove the residual chlorine from disinfected wastewater prior to discharge into the environment.

5. A team member requested that it would be helpful to present information on which green infrastructure practices are more effective than others. The project team noted that bioswales, planter boxes, and permeable pavement are being considered as representative green stormwater infrastructure technologies and the effectiveness depends greatly on the soil infiltration conditions. In the City, the soils are generally poor for infiltration.

6. NJDEP indicate that they are expecting a discussion as part of the alternatives evaluation report as to how the Project Team sought feedback from the Supplemental CSO Team and how this feedback was received. The team confirmed that this would be done.

7. A team member noted that because the selected projects may be implemented over up to the next 30 years, it would be interesting to include discussion in the evaluation of alternatives as to how advances in technology could be incorporated in the future. The project team noted that real time controls and smart infrastructure, with sensors and automation, is a consideration and that implementation factors can be further addressed with the plan selection.

Feedback from the Supplemental CSO Team members was again solicited through the web-based survey application. The survey questions and responses were as follows:

Question Possible Selections	Response Count
What do you consider the primary benefit of green infrastructure practices?	
Water quality improvements	2
Reduced flooding	5
Water harvesting/conservation	0
Aesthetic, green community spaces	1
Increased property values	0
Job creation for operations & maintenance	0
Total	**8**
What do you consider the primary barrier to green infrastructure implementation in public rights-of-way and open space areas?	
Project site identification	0
Operations & maintenance requirements	4
Cost effectiveness relative to storage (relative to other technologies)	4
Lack of funding/acceptance due to newer technology	1
Total	**9**
What do you consider the primary benefit of grey infrastructure practices?	
Reduced flooding	3
Lower maintenance than green infrastructure	2
Lower cost per gallon captured vs. green infrastructure	6

Question	Response
Less visible	0
Total	**11**
What do you consider the primary barrier to grey infrastructure implementation?	
Capital cost	9
Large site disruption during construction	0
Does not create long term jobs (less maintenance required)	0
Does not contribute to community aesthetics/green spaces	2
Total	**11**
What do you consider more appropriate in selecting CSO control alternatives?	
Low capital costs, higher long-term maintenance cost	0
High capital costs, lower long-term maintenance cost	10
Total	**10**
Please select the indicator most important to your stakeholders in considering the financial capability of the community.	
Median household income	3
Current cost of wastewater/water usage	2
Unemployment rate	0
Cost of living (available disposable income)	1
% of homes owned vs. rented in the City	1
% of population receiving social security benefits	0
% of population below the poverty line	3
Other?	0
Total	**10**

8.1.3 Meeting No. 7

The seventh meeting, held on April 11, 2019 was attended by 16 individuals, of which three were from NJDEP, six were from the permittee team including Elizabeth, JMEUC and consultants, and seven were stakeholder representatives from the other invited groups. This meeting included an update on the public participation process and a recap of the background, context and objectives of the LTCP process. This was followed by discussion of the control objectives, and more detailed discussion on the evaluation of each of the alternatives. Significant discussion and feedback was encouraged at this meeting, and questions and comments were as follows:

1. NJDEP indicated that they would be interested in seeing the drone footage of the Trumbull Street construction site, which could be used as part of a press release for education and awareness about construction of CSO projects.

2. NJDEP asked for clarification on the number of overflow events and whether one storm would produce overflows at all outfalls. The project team responded that because each sewershed would respond to precipitation differently based on characteristics such as land use, size, and shape, one overflow event could represent an active overflow occuring at a range from one to all of the outfall locations.

3. A team member noted that population projections may not take into account the number of undocumented people that would use the system. The project team responded that this is true, however there is no way to quantitatively know what this additional use of the system would be.

4. A team member asked why contamination would be an unfavorable in siting a tank, if it does not rely on infiltration. The project team responded that contamination is not a no-go, but it would be considered in terms of cost to address the contamination, fill, etc.

5. A team member noted that most municipalities across the northeast would also face challenges in siting in urban areas, and asked what others are doing. The project team responded that tunnels, sewer separation, and controls with additional property purchases are considered in other places.

6. A team member asked if Green Acres sites can be used, as that is the only open land available. The project team responded that it is possible but unclear as to what the requirements would be. It would also be important to consider any above ground structures that would be auxiliary to the below-grade tanks and pump station. It will be necessary to coordinate across various planning agencies to address items such as Green Acres. A team member explained that if the City receives any funding from Green Acres, restrictions were placed on all parkland owned by the City.

7. A team member asked whether repurposing the existing sewer as a sanitary sewer and installing new storm sewer had been considered. The project team noted that this may require more cleaning due to septic conditions and that new sanitary sewer construction was selected for planning purposes. The details of the sewer separation implementation would be considered further in the future if and where chosen to be included in the control plan.

8. A team member asked what it means when the system is leased to a private utility company. The project team responded that in the City, the sewer system is not leased to a private company, but a contract service provider operates the system. The City's contract sewer system operator is responsible for operating, cleaning, maintaining and repairing the system, but the City is still responsible for capital planning and improvements.

9. A team member expressed that sewer separation would likely be disruptive and would have to be done with a lot of education and updates to the community on why this is happening, because there is currently minimal connection between residents and such projects.

10. NJDEP asked if lower cost basins for sewer separation are more active. The project team responded that they are not, and they have smaller flows. They also noted that proximity to previously separated areas is also a factor to be considered in determining which areas would make sense to separate in the future.

11. A team member asked what a basin is. The project team responded that this refers to the geographic area contributing flow to an outfall regulator.

12. A team member asked whether the City could buy a site, build a storage tank, then re-sell the site with an easement. The project team responded that a portion of the site would need to be maintained for cleanout and access, and it would be difficult to resell the property unless it was partitioned. This would be best decided with the property owner, but it would likely be easier/lower-risk to just get an easement with the existing owner and not buy the site outright.

13. A team member asked whether it would be possible to have a surface-mounted tank. The project team responded that this would require influent pumping into tank, which needs much larger pumps than effluent pumping, and the above grade facility may be less acceptable to the public. The team member suggested that it could be used as an observation deck or for education. The project team responded that land is so valuable that burying the tank would likely be preferable but this would be considered further if moving forward with this type of alternative.

14. An attendee asked how many years it would take to construct a deep tunnel solution. The project team responded that planning and design would take 5 to 10 years, and construction would take several additional years.

15. A team member asked whether the tunnels would be constructed under existing infrastructure, or whether sites would be relocated. The project team responded that the tunnel would be constructed deep underground in rock, and relocation would only be required at shafts.

16. A team member asked about using peracetic acid for disinfection, and if other treatment methods would be considered as new developments are made. The project team responded that peracetic acid is a newer technology and other current processes could be looked at during facility planning.

17. A team asked what type of green infrastructure would implemented, and the project team responded that it would typically be bioswales but other types would be evaluated. NJDEP clarified whether bioswales were selected as the representative type for the model and the project team confirmed this. The team member noted that it would be helpful to see the other types of green infrastructure and the process for how the type would be selected. They added that this is one of the only alternatives that the community would be able to participate in and suggested reviewing the ordinance conditions and how green infrastructure could be incorporated.

18. NJDEP asked whether a cost per gallon was known for green infrastructure. The project team indicated that this is still being developed but would be forthcoming.

8.1.4 Meeting No. 8

The eighth meeting, held on June 7, 2019 was attended by nine individuals, of which six were from the permittee team including Elizabeth, JMEUC and consultants, and three were stakeholder representatives from the other invited groups. This meeting included an update on the public participation process, a recap of the background and objectives of the LTCP process, and a summary of the population projection. This was followed by a progress update on the screening criteria and process, as well as the evaluation criteria and process for each of the CSO control alternatives. Questions and comments from this meeting were as follows:

1. The team was asked for location suggestions for an open public meeting in the upcoming months and the selection phase. The Mickey Walker Center and Stephen Sampson Senior Center were suggested, and it was noted that that parking is available at Peterstown Community Center. School auditoriums were also suggested as a possible location.

2. A team member asked whether the City still has a responsibility to remove CSOs if its removal from the Elizabeth River does not sufficiently improve water quality. The project team responded that the City is responsible for doing their part such that it does not prevent the attainment of water quality goals, in the event that upstream sources address their load contributions.

3. A team member asked whether water quality samples were taken before and after a wet weather event, and whether there is dilution shown. The project team responded that samples were taken before and after events and that pathogen concentrations were typically higher during the wet weather events.

4. A team member asked whether it is allowable to have different numbers of overflows at different locations. The project team responded that different basins respond differently in wet weather and a group of outfalls can be considered for a certain level of control based on overflow activations, conditional on NJDEP approval of the approach.

5. A team member indicated that there seems to be open space available near Jersey Gardens, and suggested this site may be feasible for CSO controls. They suggested that costs to homeowners may be mitigated by assessing new developments in the City and incorporating CSO controls there. The project team indicated that the Jersey Gardens area is already serviced by a separate sewer, and coordination with the Planning Department would be needed to incorporate current and future redevelopment considerations. A fair and equitable distribution for the CSO controls will be an important factor for public acceptance and is evaluated as part of the community's financial capability to afford the CSO control program. The project team also noted that this may be associated with the development of a stormwater utility fee.

6. A team member asked whether the City meters what enters and exits the combined sewer system. The project team indicated that there is a meter at TAPS, as well as a meter on the connection from Roselle Park to monitor their contribution.

7. A team member asked whether a new forcemain would be constructed to convey flow to TAPS. The project team responded that the 65 mgd TAPS upgrade would maximize the existing forcemain infrastructure.

8. A team member asked whether community development block grant funds could help finance these alternatives. The project team responded that such funding is typically not used for this purpose by the City, but could be investigated. State revolving loan funding for Clean Water Act programs would likely be a funding source, but ultimately the program will be primarily financed through municipal bonds.

9. A team member suggested clarifying the wording for satellite treatment at individual outfalls to indicate that while sites at the outfall are preferable, the sites are not necessarily at the end of the outfalls.

10. A team member asked whether the tanks would be located underground. The project team indicated that the current program is considering below-grade tanks.

11. A team member asked how long the NJDEP allows municipalities to construct and implement their control programs. The project team indicated that it would be about 20-30 years.

12. A team member asked if a combination of alternatives could be looked at. The project team responded that the intent is to combine projects and alternative programs into the selected long term control plan.

13. A team member asked how the percentage of area managed by green infrastructure relates to the volume of overflow. The project team indicated that a 2.5% capture corresponds to about a 20 million gallon annual reduction.

14. A team member asked what the basis of a stormwater utility fee would be. The project team indicated that it likely would be based on impervious area.

Feedback from the Supplemental CSO Team members solicited through survey questions. Rather than responding online as in previous meetings, responses to the survey questions were discussed verbally in order to generate discussion as a group. The questions and responses were as follows:

Question Possible Selections
What are the most important priorities for the community related to wet weather?
Address basement flooding
Community greening (tree planting, green infrastructure, etc.)
Community employment
Affordability
Discussion: The first priority would be addressing basement flooding, followed by affordability.
How would your constituents feel about the acquisition of private property for siting CSO facilities?
Acceptable
Maybe, if considered the best CSO management strategy
Maybe, if well-screened or incorporated into existing landscape/architecture
Not in favor – disruption to community, displace residents, etc.
Discussion: It would depend on the area of the City, as well as the type of property.
What factor would be most important to your constituents in forming a stormwater utility for financing of CSO controls?
Establish rates that are fair and equitable
Credits to rate-payers for reducing runoff through green infrastructure, etc.
Constituents would not be open to establishing a stormwater utility
Other
Discussion: This would require a lot of public education, in order to demonstrate to the community that the rate is fair and equitable. While community members would be very sensitive to cost, it is likely that only a small percentage of the community would look into credits. A comparison of a sewer bill with and without a stormwater fee could be presented to show how it offsets the sewer rate for a typical residential property. It would be critical to forewarn the community that this is coming, through newsletters, meetings, etc.
What would be the preference of your constituents in approach to siting CSO controls?
Centralized solution – longer-term disruption to streets, fewer locations
Satellite sites – smaller, shorter-term disruption, several locations
Discussion: One member expressed a preference for a centralized solution as it would require fewer stakeholders to coordinate with.

8.2 Posters, Flyers, Brochures, and Handouts

The City of Elizabeth developed and circulated a new informational flyer (see appendix) which provides educational information about CSOs, the LTCP process, and some of the projects that the City is currently working on.

This flyer was shared on social media via City of Elizabeth's Twitter and Facebook in mid-December 2018 (shown below). The City of Elizabeth continues to maintain a Twitter page followed by over 2,200 users and a Facebook page followed by over 9,700 users. With such a large following, the permittees may use these two social media platforms to post educational information about CSOs as well as to advertise any education events or opportunities to provide input on the LTCP process and CSO alternatives. The Facebook post linking to the informational flyer reached 988 people, was clicked on 73 times, "liked" 11 times and shared 5 times. The flyer was also distributed at Elizabeth City Hall and a PDF was emailed to the 35 members of the Supplemental CSO Team for distribution through their organizational networks.

Informational handouts describing CSOs, rain gardens, and projects in Elizabeth have been made available to students at the Future City E-Day events, with an estimated 50 handouts distributed to students at each event.

In 2019, the City has initiated a city-wide tree planting program, with a goal to plant up to 2,500 trees on private property upon request by the owners. Over 15,000 copies of an informational brochure on this tree planting program were mailed to City residents to provide information on the initiative as well as describe the value of trees to a community in improving water quality, managing stormwater and reducing flooding.

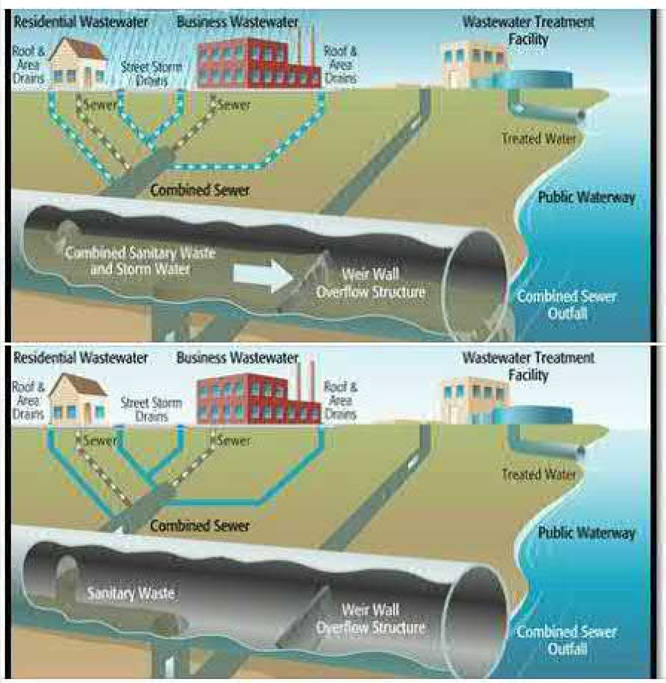
City of Elizabeth NJ @CityofElizabeth · 20 Dec 2018

The City is working with @NewJerseyDEP to improve the quality of our waterways by reducing combined sewer overflows (CSOs). To learn more visit: elizabethnj.org/pdfs/engineer/... and send feedback to the City Engineer at dloomis@elizabethnj.org.

City of Elizabeth NJ - City Hall
December 20, 2018 at 12:22 PM ·

The City of Elizabeth is working with @State of NJ Department of Environmental Protection to improve the quality of our waterways by reducing combined sewer overflows (CSOs). Interested in learning more about CSOs and the City's engineering projects? Check out the City website here: http://www.elizabethnj.org/.../engi.../CSO/ElizabethCSOFlyer.pdf.

We would love to hear your feedback! Please send your comments and questions to the City Engineer, Dan Loomis at dloomis@elizabethnj.org.



City of Elizabeth NJ - City Hall
Government Organization
Send Message
11
5 Shares

8.3 Websites

The City of Elizabeth's new website was launched on June 19, 2019 to provide residents and visitors with new features, upgrades and enhanced, user-friendly experience. Information on the CSO control plan, the municipal stormwater management plan, the stormwater pollution prevention plan, sewer system mapping, informational flyers, and a link to the CSO notification webpage will be posted on this website. Copies of the presentations made at the Supplemental CSO Team meetings and the City's current stormwater management ordinances will also be available through the webpage. .

The JMEUC website continues to include a public outreach section, which has information about water infrastructure, sewer rates, F.R.O.G. (fats, roots, oil, and grease), scheduling of plant tours, and the CSO LTCP Program.

8.4 Community Organization and School Events

The City of Elizabeth has continued to collaborate with Future City on its Environmental Day and Estuary Day activities, attending events on October 5, 2018 and May 3, 2019. At each event, the City made about 8 presentations to over 250 students from different City schools on topics such as combined sewers, rainfall infiltration on different types of land surfaces, and the structure and function of rain gardens. The City will continue to participate in these 2 annual student outreach events as an excellent way to reach many students from various parts of the city.

In addition, during the summer, the City of Elizabeth's Public Works Department offers a Trash and Recycling educational outreach to children at summer camps through an entertainment program. Five sessions were held between August 13, 2018 and August 16, 2018. Four of the sessions were attended by an average of 25-50 students per session, and one session had 10-15 students in attendance. This educational outreach program will be provided again during the summer of 2019.

8.5 News Releases and Media Coverage

Media advisory notices indicating the City of Elizabeth's participation in public education events, such as those organized through Future City, Inc. and Elizabeth River/ Arthur Kill Watershed Association, have been issued. Further news releases are planned to publicize meetings presenting information on LTCP alternatives and plan selection. Media advisories may also be issued to invite other interested groups to participate in the Supplemental CSO Team.

The City of Elizabeth also arranged with the police department to take drone footage of the construction site at the Trumbull Street Stormwater Control Project, with the intention to use this footage in future public awareness videos.

8.6 Regional and Watershed Based Partnerships

The permittees continue to recognize the value in collaboration with regional groups focused on CSO issues and they have and will continue to actively participate in events hosted by the groups such as Jersey Water Works and the NJ CSO Group. Through these meetings, permittees are sharing resources, obtaining feedback from peers on challenges with CSO mitigation and the LTCP process, and reviewing techniques on public messaging.

The City has been meeting with NJDEP on a quarterly basis to provide status updates on LTCP progress, and to obtain regular feedback on project direction and developments. The City also hosted the NJDEP's CSO Public Participation Workshop on March 6, 2019 at the local Peterstown Community Center. This workshop was Organized by NJDEP in order to gather Supplemental Team members and CSO

Permittees from across the State and discussed methods of identifying and effectively engaging with stakeholders.

8.7 CSO Outfall Identification Signs

The City of Elizabeth has continued to maintain signs at each CSO outfall to educate the public of the potential hazards associated with water contact during and following wet weather.

8.8 CSO Notification System

As part of NJ CSO Group, the City of Elizabeth has continued to utilize the online CSO notification system (https://njcso.hdrgateway.com/) as a public information tool advising on the status of CSO occurrences in the City of Elizabeth and certain other communities participating in the NJ CSO Group.

8.9 Green Infrastructure Signage

The City is committed to continuing to install signage for rain gardens explaining the function and purpose of green infrastructure as a strategy in stormwater management. The locations include at Trumbull Street, Kenah Field, and Green Acres Park.

8.10 Combined Sewer Infrastructure and Treatment Plant Tours

JMEUC continues to host several tours each year of its wastewater treatment facilities upon request by interested parties. Additional tours for community, environmental, and media groups of the combined sewer outfall and control facilities, receiving waterways, JMEUC wastewater treatment plant, and green infrastructure installations may be hosted by the permittees to foster understanding of the sewer system, water quality, and CSO issues and control alternatives.

Section 9
Conclusions

In this section, the key findings from the work completed for this Development and Evaluation of Alternatives Report and the implications of the findings on the direction of the combined sewer overflow (CSO) long term control plan (LTCP) process are summarized. The estimated costs and the projected benefits for CSO control are compared for the CSO control programs and performance metrics considered. The main considerations for informing the future selection of the control measures, individually or in combinations, for the CSO LTCP, are discussed, including the varying levels of complexity, effectiveness, public acceptance, and costs.

9.1 Alternatives Cost and Performance Summary

The reductions in CSO volume and the sizes of the facilities required to achieve those reductions were estimated for each control program with a series of modeling analyses. Estimated facility sizing was used to develop the Class 5 (+100%, -50%) cost estimates for total capital costs, 20-year operation and maintenance (O&M) costs, and total present worth (TPW) as present values for the control alternatives analyzed. This information is summarized in Table 9-1 and the total present worth costs normalized by the gallon of CSO abated or controlled in the Typical Year are tabulated in Table 9-2. Where applicable, the alternative program is qualified by the level of CSO control or the extent of implementation considered. For example, the control programs for satellite treatment facilities, satellite storage facilities, and deep tunnel storage have subcategories using the frequency of CSO events for the Typical Year as a performance metric, while the additional conveyance and treatment alternative considers the discharge from the Trenton Avenue Pump Station (TAPS) as the extent of implementation measure.

Table 9-1: Control Alternatives Cost Summary

Control Alternative	Control Level or Extent of Implementation	Estimated Costs ($ Million)		
		Total Capital Cost	20-Year O&M Cost as Present Value	20-Year Total Present Worth
1) Sewer Separation	0 events/yr	$1,244	$151.3	$1,396
2) Satellite Treatment Facilities	0 events/yr	$865.2	$98.0	$963.2
	4 events/yr	$803.0	$93.0	$896.0
	8 events/yr	$714.2	$87.0	$801.2
	12 events/yr	$714.2	$87.0	$801.2
	20 events/yr	$488.8	$70.0	$558.8
3) Additional Conveyance & Treatment	55 mgd-Real Time Control	$9.06	$1.10	$10.16
	140 mgd	$85.69	$15.4	$101.12
4) Satellite Storage Facilities	0 events/yr	$1,175	$130.7	$1,306
	4 events/yr	$638.1	$71.4	$709.5
	8 events/yr	$485.0	$56.2	$541.3
	12 events/yr	$439.9	$50.2	$490.0
	20 events/yr	$297.2	$35.0	$332.2
5) Deep Tunnel Storage	0 events/yr	$901.9	$61.0	$962.9
	4 events/yr	$684.6	$46.0	$730.6
	8 events/yr	$576.2	$37.0	$613.2

Control Alternative	Control Level or Extent of Implementation	Estimated Costs ($ Million) Total Capital Cost	20-Year O&M Cost as Present Value	20-Year Total Present Worth
	12 events/yr	$524.1	$34.0	$558.1
	20 events/yr	$459.8	$29.0	$488.8
6) Green Stormwater Infrastructure (by percent impervious area managed)	2.5%	$104.6	$1.00	$105.6
	5.0%	$204.2	$2.00	$206.2
	7.5%	$306.4	$3.00	$309.4
	10.0%	$408.4	$4.00	$412.4
	15.0%	$611.6	$7.00	$618.6
7) Inflow/Infiltration Reduction	50% I/I volume reduction[1]	$594.0	Not appl.	$594.0

[1] Reduction in JMEUC separate sanitary sewer area I/I rates/volumes with maximum attainable I/I reduction at the sewershed level at 50% of initial condition (1983 SSES results).

Table 9-2: Summary of CSO control program CSO volume reductions

Control Alternative	Control Level/Extent	CSO Volume Abated (MG/yr)	CSO Volume Reduction (%)	Cost (TPW) per Volume Abated ($/gal)
1) Sewer Separation	0 events/yr	1068.5	100.0%	$1.31
2) Satellite Treatment Facilities	0 events/yr	1068.5	100.0%	$0.90
	4 events/yr	1063.6	99.5%	$0.84
	8 events/yr	1055.6	98.8%	$0.76
	12 events/yr	1055.6	98.8%	$0.76
	20 events/yr	956.4	89.5%	$0.58
3) Additional Conveyance & Treatment	55 mgd-Real Time Control	175.8	16.5%	$0.06
	140 mgd	370.3	34.7%	$0.27
4) Satellite Storage Facilities	0 events/yr	1068.5	100.0%	$1.22
	4 events/yr	960.3	89.9%	$0.74
	8 events/yr	867.5	81.2%	$0.62
	12 events/yr	822.9	77.0%	$0.60
	20 events/yr	661.1	61.9%	$0.50
5) Deep Tunnel Storage	0 events/yr	1068.5	100.0%	$0.90
	4 events/yr	1005.0	94.1%	$0.73
	8 events/yr	905.3	84.7%	$0.68
	12 events/yr	844.8	79.1%	$0.66
	20 events/yr	735.1	68.8%	$0.66
6) Green Stormwater Infrastructure (by percent impervious area managed)	2.5%	16.2	1.5%	$6.52
	5.0%	22.6	2.1%	$9.13
	7.5%	26.6	2.5%	$11.63
	10.0%	31.3	2.9%	$13.18
	15.0%	36.0	3.4%	$17.18
7) Inflow/Infiltration Reduction	50% I/I volume reduction[1]	See Note [2]	See Note [2]	See Note [2]

[1] Reduction in JMEUC separate sanitary sewer area I/I rates/volumes with maximum attainable I/I reduction at the sewershed level at 50% of initial condition (1983 SSES results).
[2] Specific value not calculated. See report text for further discussion.

For the additional conveyance and treatment control alternative, the annual average volume of combined sewage captured as a percentage of the total combined sewage generated during wet weather (i.e., the percent capture performance metric) was also determined for increasing conveyance capacities from the TAPS. The estimated CSO volume reduction using real time controls and a 55 million gallons per day (mgd) conveyance capacity improves the system-wide percent capture from 66.5% (for the existing conditions) to 72.0% using the Trenton Avenue Pump Station as the point of analysis, or from 83.1% to 85.9% using the JMEUC Wastewater Treatment Facility (WWTF) as the point of analysis. At the upper limit of the capacities considered, the predicted decrease in the system-wide CSO volume of 370 million gallons (MG) associated with a 140 mgd conveyance rate is estimated to correspond to a percent capture value of 78.1% with a TAPS point of analysis and 89.0% with a JMEUC WWTP point of analysis.

For the inflow/infiltration (I/I) reduction control alternative, a total capital cost of $594 million has been calculated for I/I volumes reductions in the JMEUC separate sanitary sewer area. The analysis is based on a maximum attainable I/I reduction at the sewershed level of 50% from an initial condition taken as the I/I rates identified in a 1983 Sewer System Evaluation Survey (SSES). The I/I reduction to this extent is estimated to correlate to the cured-in-place pipe (CIPP) lining of an additional 1.5 million feet of sewer main and 77,000 feet of building sewer laterals in the system.

The primary benefit of I/I reduction for CSO control in the subject system is in potentially reducing the sizing of conveyance and treatment facilities to handle additional combined sewer flow at the JMEUC WWTF. Results from modeling analyses for the Typical Year indicate that the assumed I/I reduction levels in the separate sanitary sewer area could reduce the peak flow rate at the JMEUC WWTF by approximately 22 mgd. Relative to the capital cost to provide a CSO treatment train at the JMEUC WWTF for additional conveyance and treatment, which is estimated to be on the order of $16 million to $47 million for an 85 mgd capacity, reducing I/I rates to reduce the required CSO treatment train capacity is not considered cost-effective. The cost/benefit tradeoff was considered across the full range of I/I reduction levels and associated CSO treatment capacity reductions, and at no level was I/I reduction found to be cost effective.

9.2 Review of Evaluation Findings

The development and evaluation of CSO control programs is an essential step in the planning process to identify specific projects for implementation under the LTCP. This Development and Evaluation of Alternatives Report fulfils the report objectives outlined in Section 1 and was prepared per EPA and NJDEP regulatory compliance requirements and guidance documents. A broad range of CSO control strategies including source controls, collection system controls, storage systems, and treatment technologies have been considered. Appropriate alternative control programs have been identified and evaluated, including all the remedial controls specified in Part IV Section G.4.e of the NJPDES CSO Permits.

The CSO control programs have been analyzed for their practical and technical feasibility and performance capabilities under future conditions. A clear and comprehensive examination of the alternatives has been presented consistent with the CSO regulatory requirements, including a preliminary assessment of the potential impact of significant indirect user discharges. Public input on the identification and evaluation of alternatives has been an important element of the LTCP process, and an update on public participation has been incorporated in this report.

Extensive data has been compiled and analyzed for the suitable CSO control alternatives by determining the size of facilities or scale of implementation associated with a range of performance criteria. The alternatives evaluation has considered several factors, including:

- Performance capabilities and effectiveness relative to CSO volume reduction, pollutant of concern (i.e., pathogen) removal, and CSO event frequency reduction.
- Estimates of the total capital costs, O&M costs, and total present worth value associated with implementing and operating the control facilities for the level noted. Where applicable, cost estimates for land acquisition have been included due to the absence of available City-owned sites and under-utilized properties within the combined sewer area.
- Public acceptance considerations that reflect the degree to which communities may be impacted, public amenities can be incorporated, and political matters may impact the approval of a control alternative by elected officials, non-governmental organizations, and the general public.
- Institutional issues concerning permitting requirements and associated approval processes and schedule impacts.
- Implementation constraints related to likely environmental issues, subsurface conditions, construction complexity, facility reliability, and scale of operations and maintenance.
- Adaptability for multiple-use facilities to provide other beneficial services in addition to CSO control; grouped outfall applications and facility consolidation; and phased construction.
- Regulatory requirements and any potential compliance risks.

Key findings from the alternatives analysis based on the evaluation criteria above are:

General Considerations for the LTCP

- The City of Elizabeth and JMEUC are currently performing many quantity and quality source control measures considered as best management practices (BMPs) for stormwater management and pollution prevention. As noted in the CSO control technology screening process, it is expected that these current practices will be continued as applicable per regulatory requirements to help optimize system operations, minimize CSO discharges, and reduce pollutants to receiving water bodies.
- The preliminary siting analysis conducted to identify potential open or under-utilized sites for CSO control facilities demonstrated that insufficient City-owned or unoccupied land is available in the areas surrounding the CSO outfalls. Of 85 potential sites identified in an initial screening, only 11, or 12.9%, were rated favorably for CSO control facilities through a preliminary assessment by City representatives given existing land use, property ownership, and current redevelopment programs. As such, the identification of appropriate sites will be a major issue in selecting Long Term Control Plan components. The estimated costs for obtaining land rights to construct CSO facilities based on space requirements have been considered as a capital cost in the control program assessments. However, this high-level treatment will need to be significantly refined with the CSO control approach selection to account for the political, social, environmental, and regulatory impacts of siting the facilities.

Sewer Separation

- Sewer separation has been evaluated as a means of eliminating the combined sewer system completely and the associated sewage component from the discharges at each existing CSO outfall. Cost estimates for full sewer separation indicate that this control alternative is extremely costly and the analysis notes the required construction work will be extensive, affecting over 100 miles of roads and 3,500 acres of tributary area. As such, sewer separation would be highly disruptive to most City residents and would actually increase untreated stormwater discharges, which will likely be subject to additional treatment requirements in the future.

- The costs and potential construction impacts of sewer separation vary widely between basins and some of the CSO basins are relatively small in area or in tributary sewer lengths. Sewer separation may also be effective where a CSO outfall is isolated from other outfall locations or where it is impracticable to acquire a site for CSO control facilities.

- While sewer separation may not be the most practical alternative for the entire City, some more remote areas, such as the basin draining to Outfall 037A, may be more suitable candidates for basin-level sewer separation, and partial separation could also be additive to other control programs.

Satellite CSO Treatment

- Sizing of satellite treatment facilities at each CSO outfall was determined considering primary treatment using the Actiflo® process as a representative technology and disinfection using peracetic acid. At a control level less than 8 CSO events per year, the land requirements for satellite treatment are less than the land needed for satellite storage facilities. However, the estimated TPW of satellite treatment is only less than the TPW of satellite storage for the 0 CSO events per year metric.

- Satellite treatment may not be a desirable alternative due to the cost of land acquisition and challenges of permitting and obtaining easements, as well as access to and maintenance of these facilities. Furthermore, the type and scale of operations for satellite treatment facilities would require staffing resources that the City does not have.

- The satellite treatment alternatives can be adapted for grouping and consolidation of CSO outfalls if a favorable centralized site of sufficient size for the facilities is identified. The facilities would need to be sized for the peak flow rate criteria associated with the consolidated outfalls and cost savings for having fewer CSO control sites would be evaluated against additional costs for consolidation piping from the existing outfalls to the shared facility sites.

Expanded Treatment of Wet-Weather Combined Sewer Flows at the JMEUC WWTF

- Additional conveyance from the Trenton Avenue Pump Station from 36 mgd to 55 mgd (and perhaps up to 65 mgd, pending additional analysis), with real time controls to maintain JMEUC peak flow rates no higher than existing for the largest events in the Typical Year, would decrease total system-wide CSO volume by about 175.8 MG/year, compared to the existing conditions (2015) baseline. Although a major pump station improvements program would be required to replace the pumping equipment and other systems for reliable operation at the higher flow capacity, this control alternative option has a low cost per gallon of CSO volume reduction and is expected to have minimal public impact and permitting constraints. No expansion of the existing WWTP or force main are anticipated to be necessary at these flow rates.

- Additional conveyance during wet weather from the TAPS above 55 mgd (and perhaps up to 65 mgd) would require additional treatment capacity at the JMEUC WWTF. Different CSO flow path scenarios (through/around the existing JMEUC WWTF) were evaluated, all of which include various degrees of bypassing of CSO flow around the existing WWTF treatment train (including the secondary treatment portion; in accordance with an approved NJPDES permit authorizing bypass).

- The flow path with additional CSO treatment selected for further consideration in the LTCP maintains a 153 mgd peak hour flow, including a 55 mgd flow from the TAPS, through the existing JMEUC WWTF and provides a new CSO treatment train for additional conveyance up to 85 mgd from the TAPS, for a potential total future flow of 140 mgd from the TAPS. This configuration simplifies the integration of an expanded TAPS, reduces stress on existing JMEUC WWTF unit processes, and eliminates the need to increase existing unit capacities. Mechanical bar screens followed by fine screens along with a new high-rate disinfection facility will provide treatment of the additional flow (above 55 mgd) from TAPS. New conduits on the WWTF site would also be required to convey flow into the new treatment units and from the new disinfection unit to the new effluent pump station (proposed separately as part of flood control improvements) at the JMEUC WWTF.

Satellite Storage

- The control program for satellite storage facilities considered a below-grade storage tank, with 15-foot side water depth, for each CSO outfall basin. The results for the tank volume and area requirements indicate that extensive land acquisition will be necessary for the storage facilities, with an estimated 25.5 acres required for tank area system-wide for a control metric of 0 overflows per year. With a system-wide TPW of $1.3 billion, the costs and area requirements for satellite storage facilities are excessive at the 0 overflows per year level. However, the satellite storage facilities costs compare more favorably to the other control alternatives at the higher CSO frequency metrics.

- Constraints on finding sufficient suitable sites for the satellite storage facilities appears to have the greatest impact on the ability to implement this control alternative. CSO storage tanks are a familiar and proven control measure and multi-use sites incorporating CSO storage are common. The number of facilities and the scale of operations, even considering outfall consolidation, would add significant complexity and resource demands on the City. Dewatering time requirements for the satellite storage tanks may require additional conveyance and treatment, particularly for the more restrictive performance levels.

- As noted above for satellite CSO treatment, the satellite storage alternatives can be adapted for grouping and consolidation of CSO outfalls if a favorable centralized site of sufficient size for the facilities is identified. The facilities would need to be sized for the storage criteria associated with the consolidated outfalls and cost savings for having fewer CSO control sites would be evaluated against additional costs for consolidation piping from the existing outfalls to the shared facility sites.

Deep Tunnel Storage

- The deep tunnel storage control program includes approximately 19,800 feet of tunnel, servicing 26 CSO outfalls. In combination with the tunnel, this alternative incorporates satellite storage for CSO Basins 001 and 002, and sewer separation for Basin 037, which are all remote basins. It also includes consolidation piping and drop shafts for 7 outfall groups, a tunnel dewatering pump station, expanded wet weather treatment, and increased pumping required from the TAPS.

- Deep tunnel storage is one of the lower-cost alternatives on a cost per gallon basis that achieves the full range of CSO control levels evaluated. In terms of cost per gallon treated, the value is relatively constant for 8 through 20 overflow events per year, then escalates for the more restrictive performance measures.

- Relative to the other control programs, the tunnel storage alternative is anticipated to have a lower impact on the community as the majority of construction takes place below grade. The alignments would primarily be located in public right of way, with less land acquisition requirements relative to satellite storage or treatment alternatives, as land is only required for fewer and smaller shaft sites. A centralized storage tunnel is also beneficial as compared to satellite storage facilities, because it serves to store overflows from outfalls throughout the City during wet weather events.

Green Stormwater Infrastructure

- Through a desktop analysis of available geographic information layers and other resources, the maximum amount of impervious area within the City that could practically be directed to green stormwater infrastructure (GSI) was estimated to be 2.6%. Nonetheless, for evaluation purposes, directing 2.5%, 5%, 7.5%, 10%, and 15% of the impervious area within the combined sewer area to GSI was modeled.

- Results from the modeling analyses indicate that GSI achieves relatively small reductions in CSO volumes. Compared to the existing conditions (i.e., 2015 baseline), control of runoff from 5% of the impervious area, or approximately 127 acres, reduces CSO volumes by about 22.6 MG, or 2.1%. An important factor related to the GSI performance is the generally poor infiltration rates associated with the soil conditions within the City.

- GSI does not achieve the desired level of control in terms of volume reduction or reduction in CSO frequency. As such, GSI can only provide limited support toward meeting the CSO control objectives and it is anticipated that if included in the LTCP, it would be additive to other control programs due to its aesthetic and public value. Green infrastructure has a notably higher TPW cost per gallon relative to other alternatives due to significant operational and maintenance requirements.

I/I Reduction

- The JMEUC System Characterization Report and the I/I reduction control program evaluation in this report document that the JMEUC trunk sewers and WWTF can capture and treat all flow from the JMEUC service area during the Typical Year, including peak flow from the TAPS at the existing contractual limit (36 mgd) and the proposed additional conveyance (up to 55 mgd TAPS discharge) with real time controls to maintain peak flow rates no higher than existing for the largest events in the Typical Year.

- In comparison to I/I levels in other similar systems, the JMEUC sanitary sewer service area I/I levels were found to be relatively low, with significant I/I reductions having been previously achieved across the JMEUC service area. It is estimated that a 30 to 40% reduction has been realized in I/I rates against baseline values identified in a 1983 SSES.

- Assuming a 50% maximum attainable I/I reduction at the sewershed level, planning level estimates noted that a 22 mgd reduction in the peak hourly flow rate at the JMEUC WWTF during the Typical Year under existing conditions. The estimated CIPP and other sewer system rehabilitation costs related to the maximum attainable I/I reduction is approximately $594 million, while the identified CSO control impact is a reduced wet weather treatment train capacity. Considering a maximum CSO treatment train capacity required of 85 mgd with an estimated capital cost of $46.7 million at the upper end of the options considered, the identified I/I reduction costs are not cost effective for a 25% reduction in the CSO treatment train capacity.

- While I/I reduction will not be included in the LTCP as a CSO control approach, JMEUC will continue to encourage its member and customer communities to implement I/I reduction as part of ongoing sewer system management practices.

9.3 Direction for the Selection and Implementation of Alternatives Report

As noted in Section 1, the goal of the CSO LTCP process is to select and implement a CSO control program that is most capable of cost-effectively improving water quality within the impacted receiving waters. The contents of this report provide the information necessary for the City and JMEUC to advance the LTCP process to the Selection and Implementation of Alternatives step. Further assessment and refinement of the control alternatives is expected as a CSO control approach is selected and recommended LTCP components are identified.

In this next step, both standalone programs and combinations of different programs will be considered based on feasibility, site suitability, cost, percent capture and other considerations. As discussed in this report, the City and JMEUC are coordinating with the NJ CSO Group to complete water quality modelling of the receiving waters. The results of the water quality modelling will be used to finalize the control approach selection (demonstration or presumption) to be used to demonstrate fulfilment of the water quality-based requirements of the EPA CSO Policy and the NJPDES CSO permits.

The Selection and Implementation of Alternatives Report will present the target water quality goals and CSO control objectives. The description of the selected control program will include planning level facility descriptions, sizing, cost estimates, implementation program and schedule, anticipated siting requirements and other relevant information. A financial capability assessment will be included in the report to evaluate affordability of the proposed implementation program and schedule, and to identify and evaluate methods of financing the selected control program. The proposed compliance monitoring program plan for the selected control program will also be included.

Appendix A

Map Exhibits

Plate A: City of Elizabeth Overall Sewer System Map

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Legend

- Significant Industrial Users
- JMEUC Treatment Plant
- Pumping Station
- CSO Discharge
- Regulator
- Elizabeth City Hall
- Trunk Sewer Alignments
- CSO Outfall
- Force Main Alignment
- Siphon Main
- Edge Of Pavement

Interceptor Alignment

- Easterly Interceptor
- Westerly Interceptor
- JMEUC Trunk Sewer
- Municipal Boundary
- Water Bodies
- Regulator Basin

Sewer System Drainage Types

- Combined Sewer System
- Seperated Sewer System
- No Contributing Flow

Map Document: (P:\340878\GIS\MXD\Alternatives Analysis\Plate_SewerSystemComponents_Overall.mxd)
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MOTT MACDONALD
111 Wood Avenue South
Iselin, New Jersey 08830-4112

CITY OF ELIZABETH
UNION COUNTY, NEW JERSEY
PLATE A
SEWER SYSTEM OVERALL MAP

Designed	Drawn	Checked	Approved	Date
AG	MM	AG	AG	6/13/2019

Appendix B

Updated Technical Guidance Manual

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Passaic Valley Sewerage Commissioners

CSO Long Term Control Plan

Updated Technical Guidance Manual

January 2018

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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

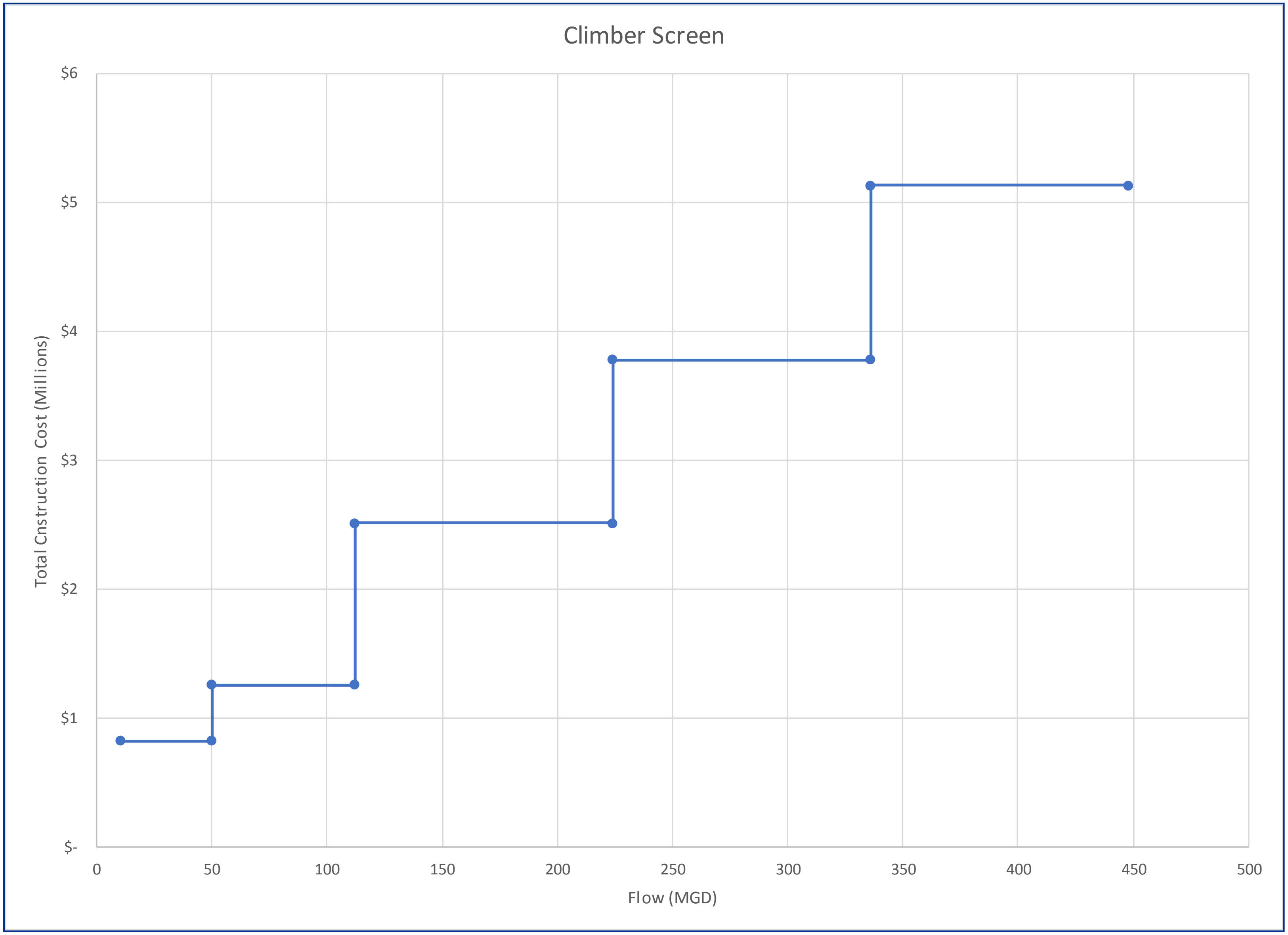


Table 2-2 - Annual Operation Costs of Climber Screens

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

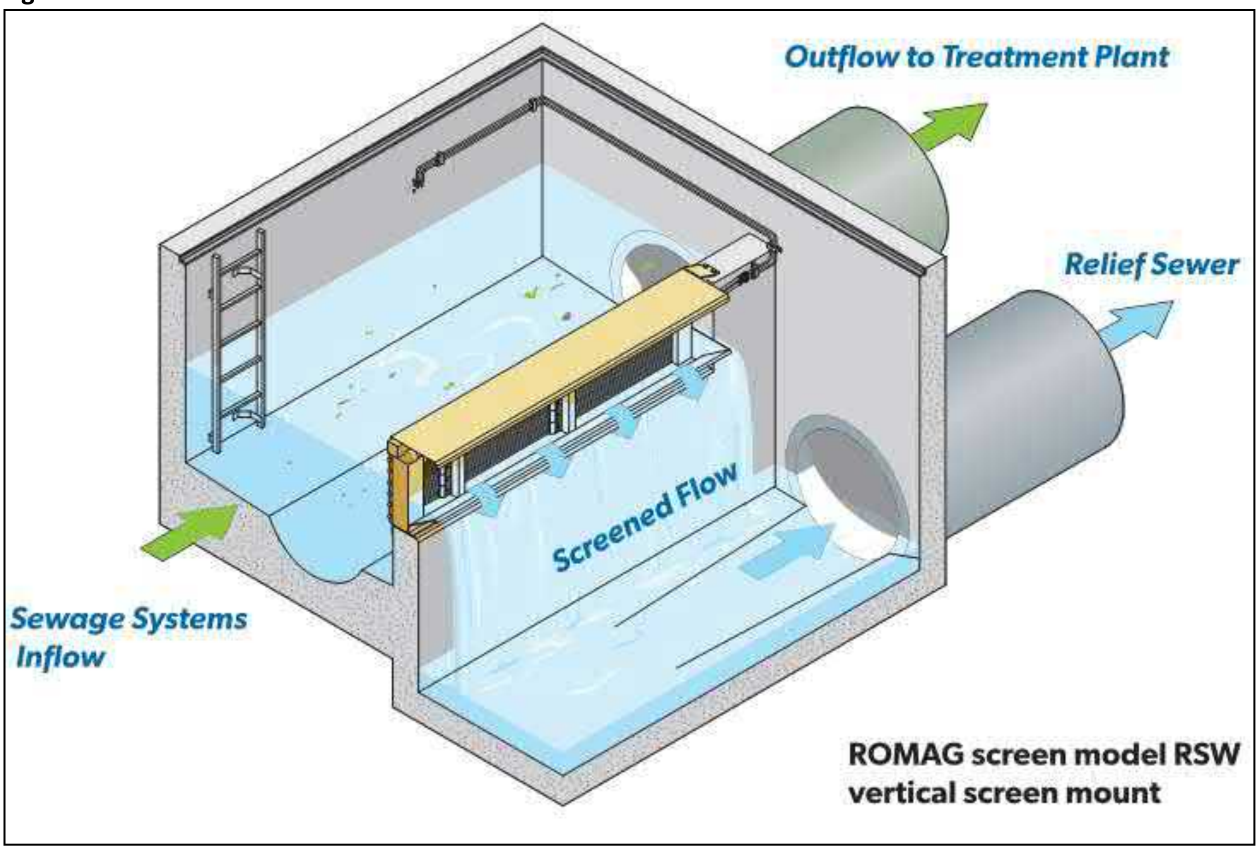
2.2.2 Fine Screens

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

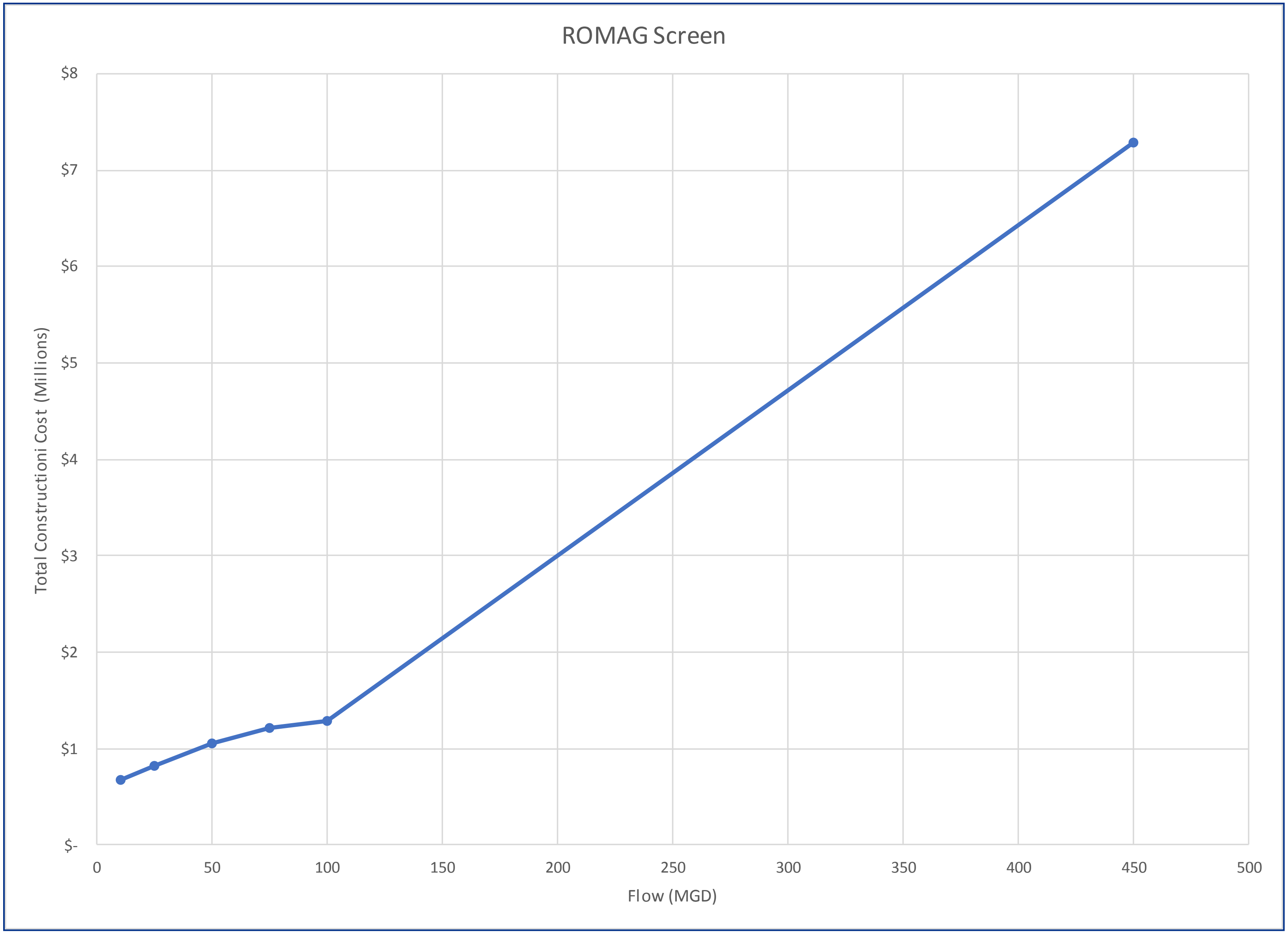


Table 2-5 - Annual Operation Costs of ROMAG Screens

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.

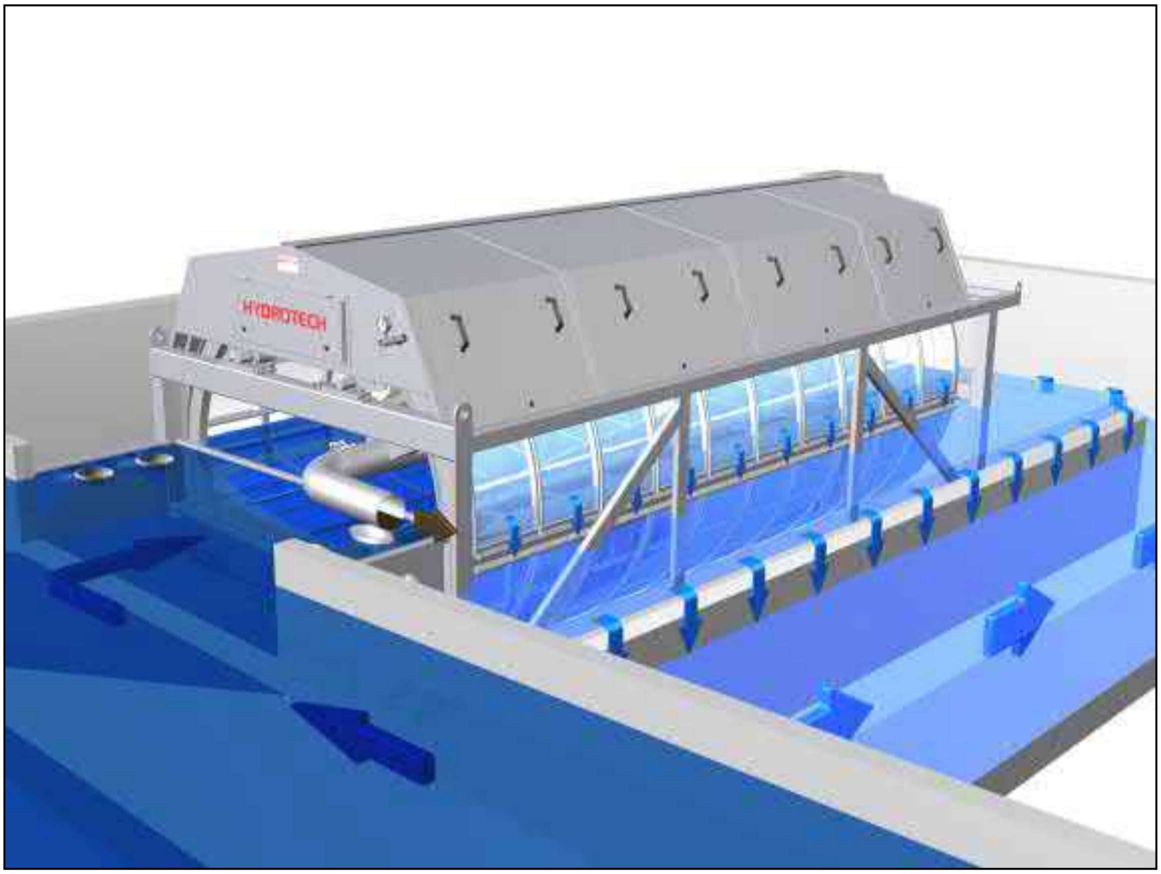
These screens are not considered applicable for CSO treatment and not further evaluated.

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.

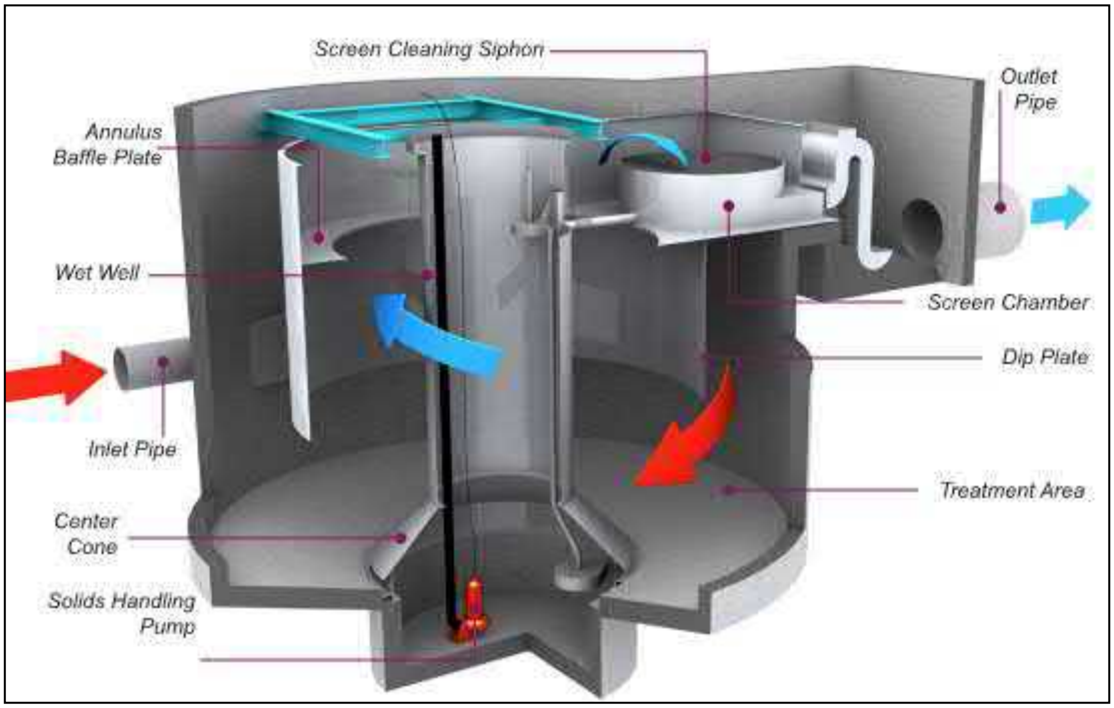
2.3.1.1 Storm King® Vortex Separator

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

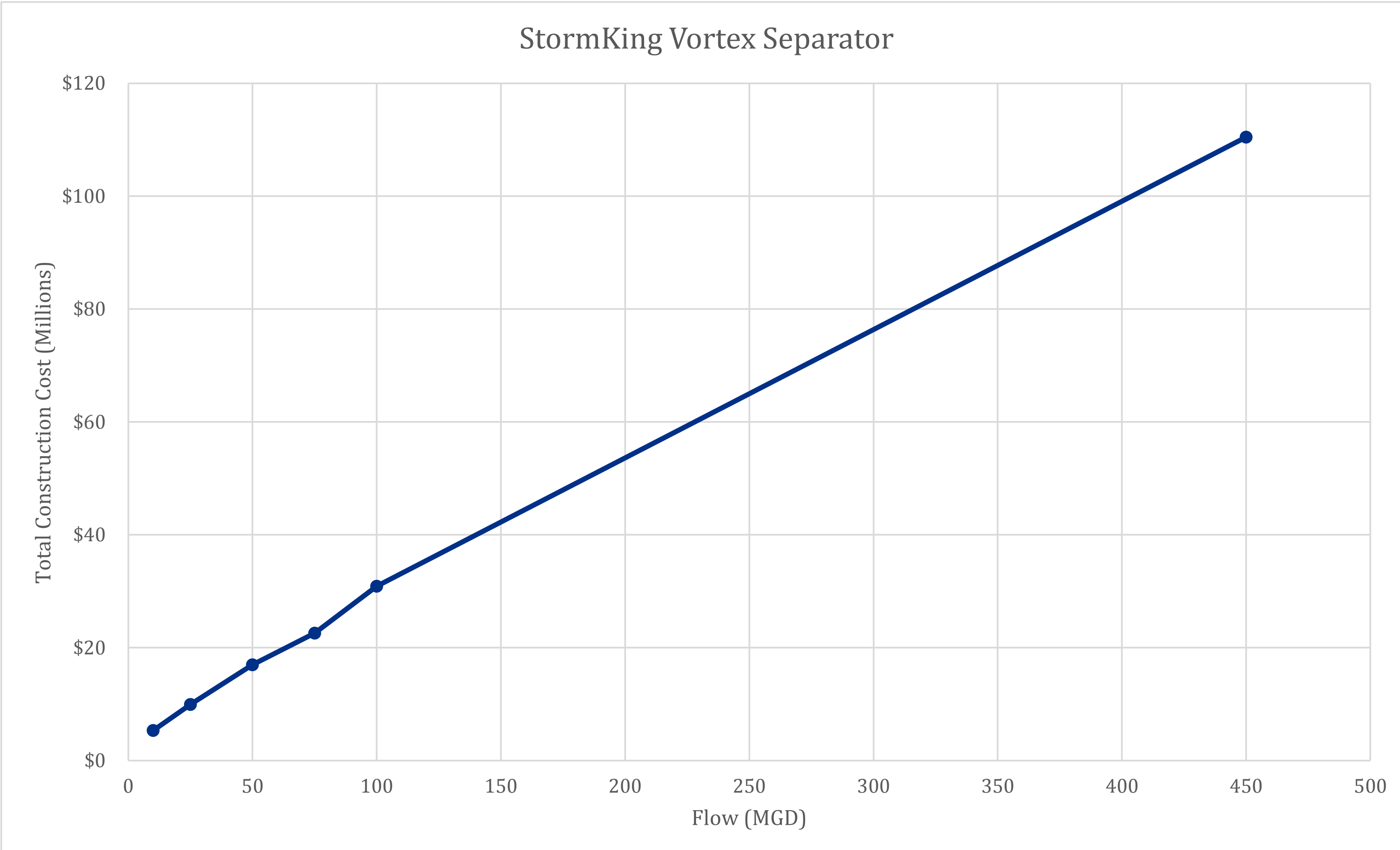


Table 2-9 - Annual Operation Costs of Storm King Vortex Separator

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

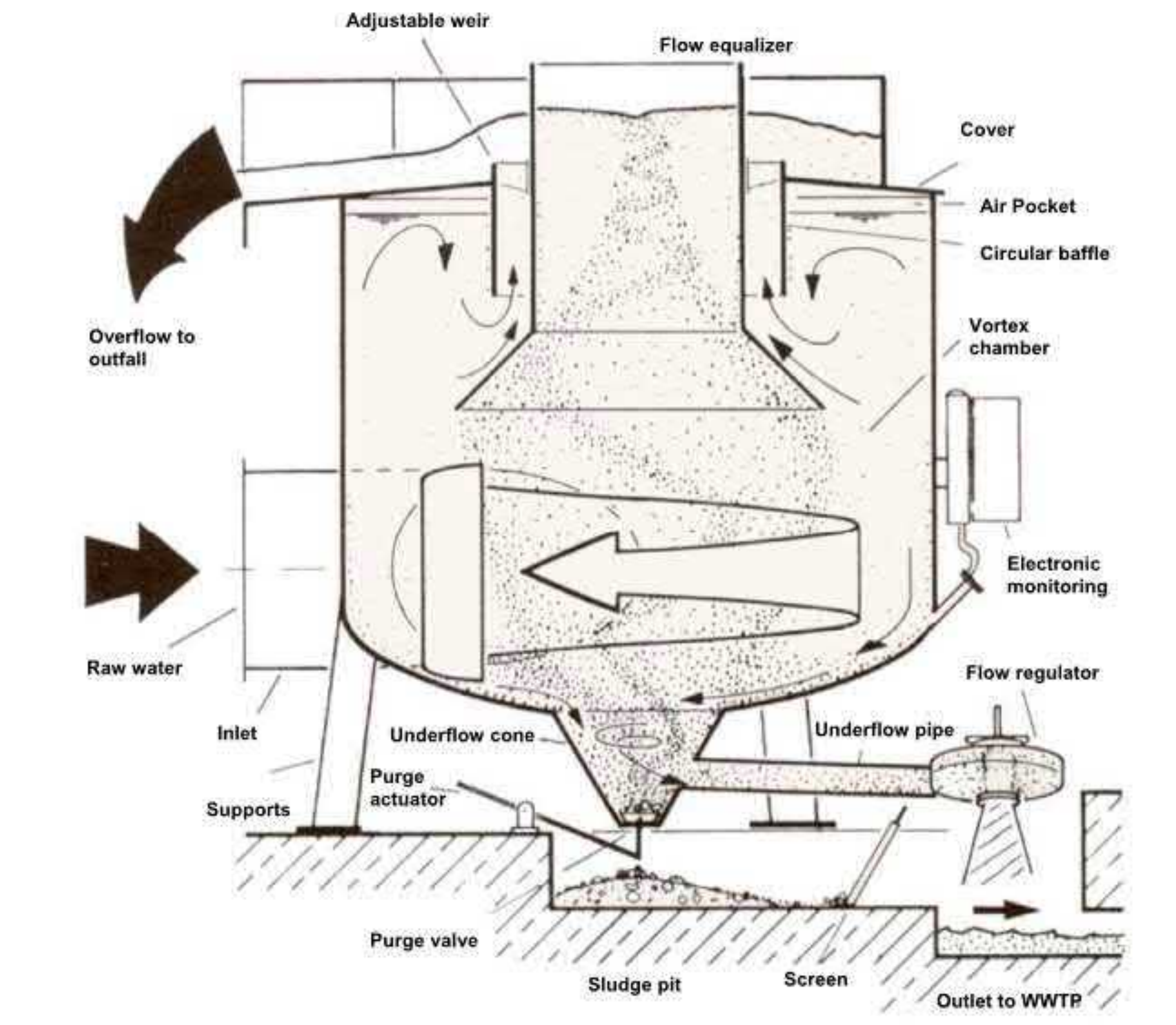
2.3.1.2 HYDROVEX® FluidSep Vortex Separator

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

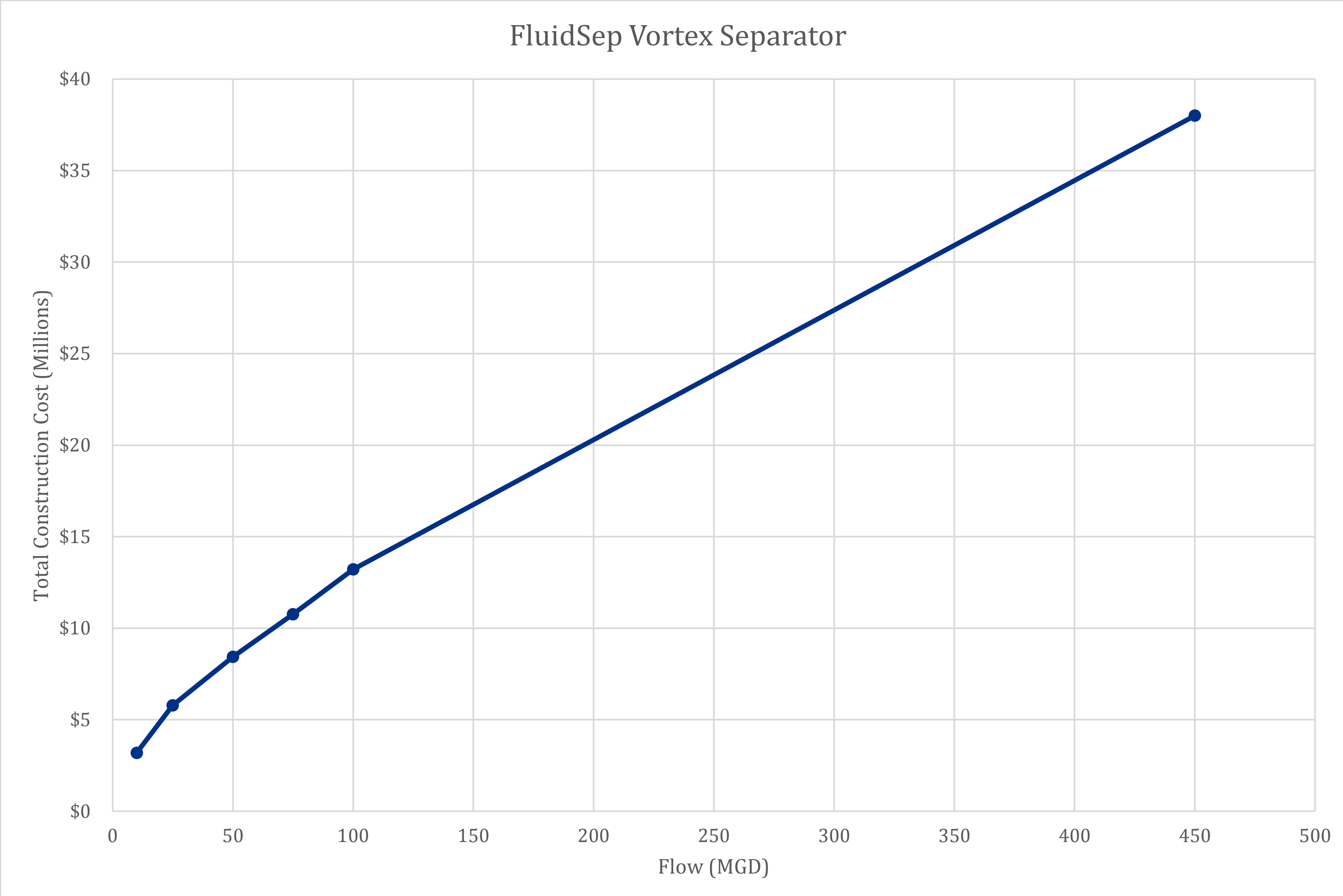


Table 2-12 - Annual Operation Cost of HYDROVEX Fluidsep Vortex Separator

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

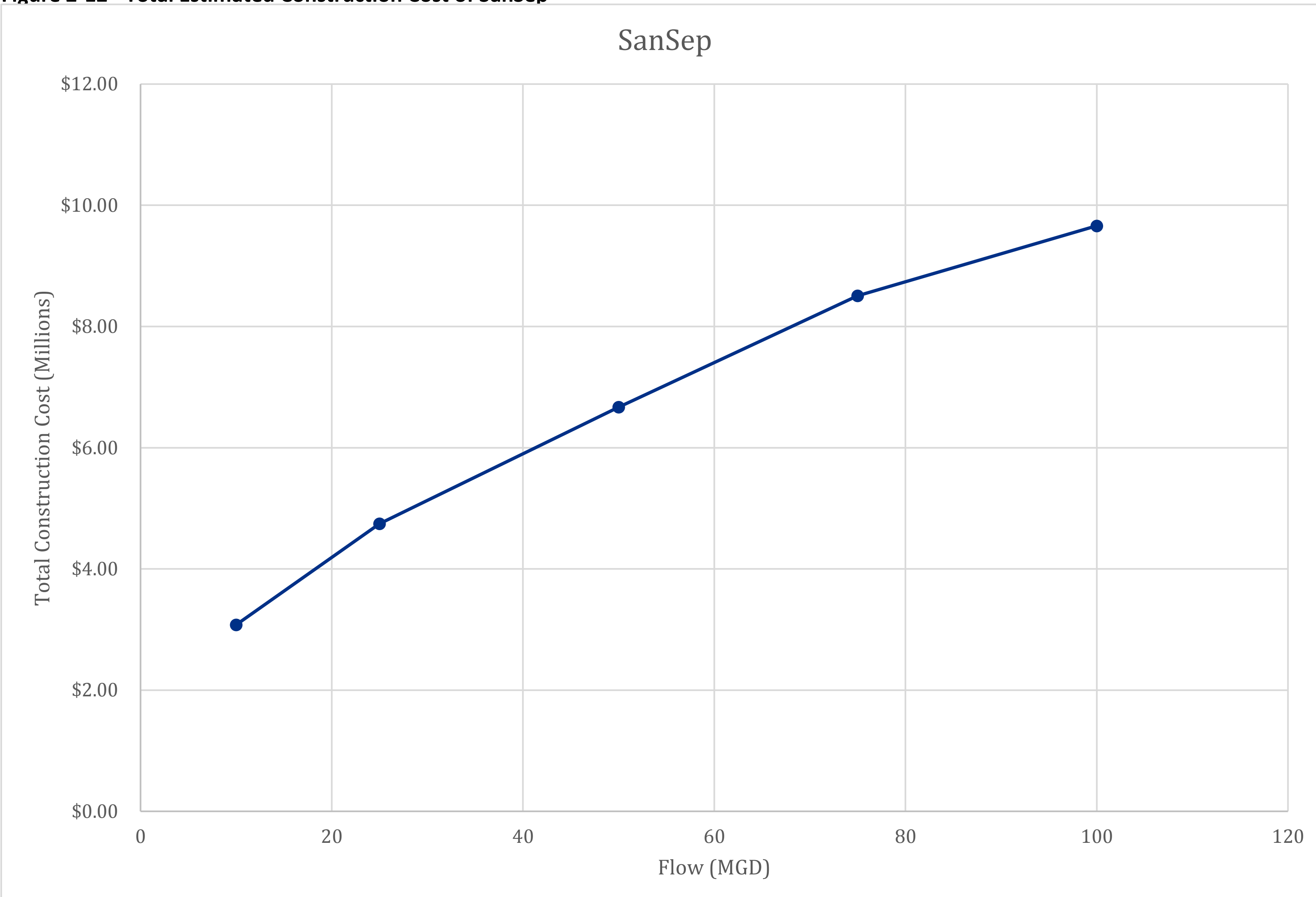


Table 2-15 - Annual Operation Cost of SanSep

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.

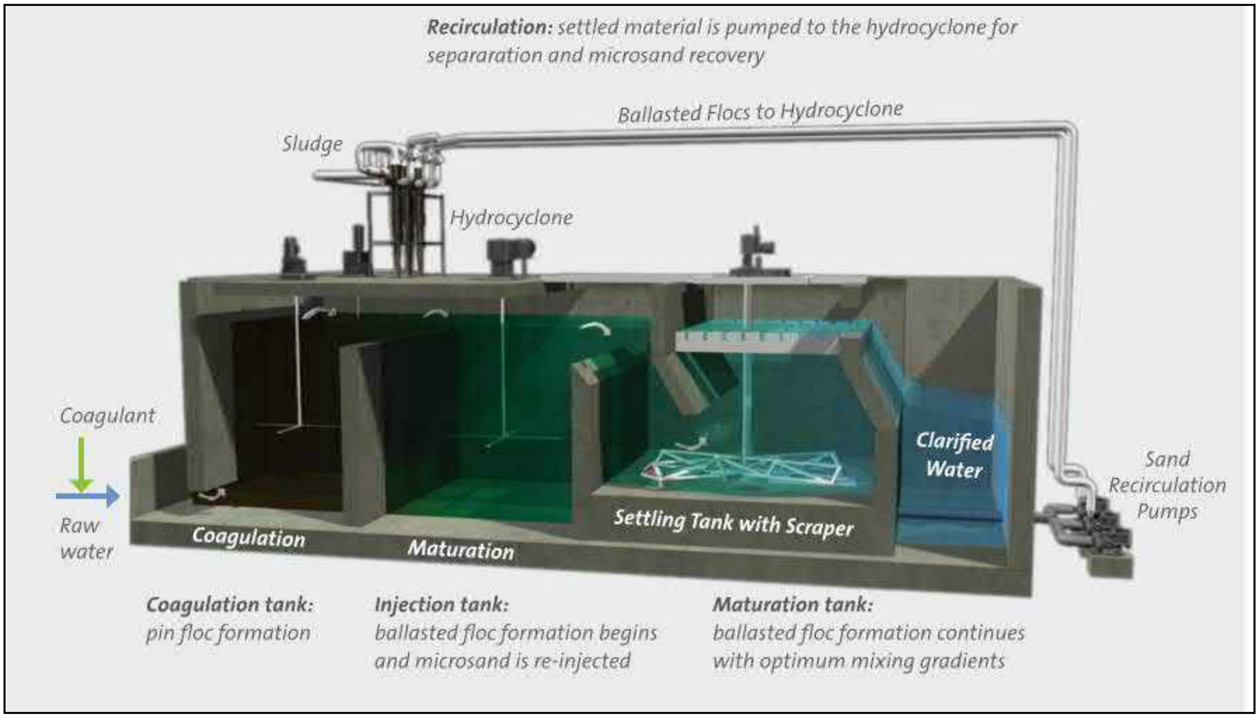
The ballasted flocculation processes, using chemical addition as a critical part of their operation, have higher removal percentages than vortex/swirl separation processes for virtually all the pollutants with the exception of total nitrogen and NH_3. The compact size of ballasted flocculation units 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

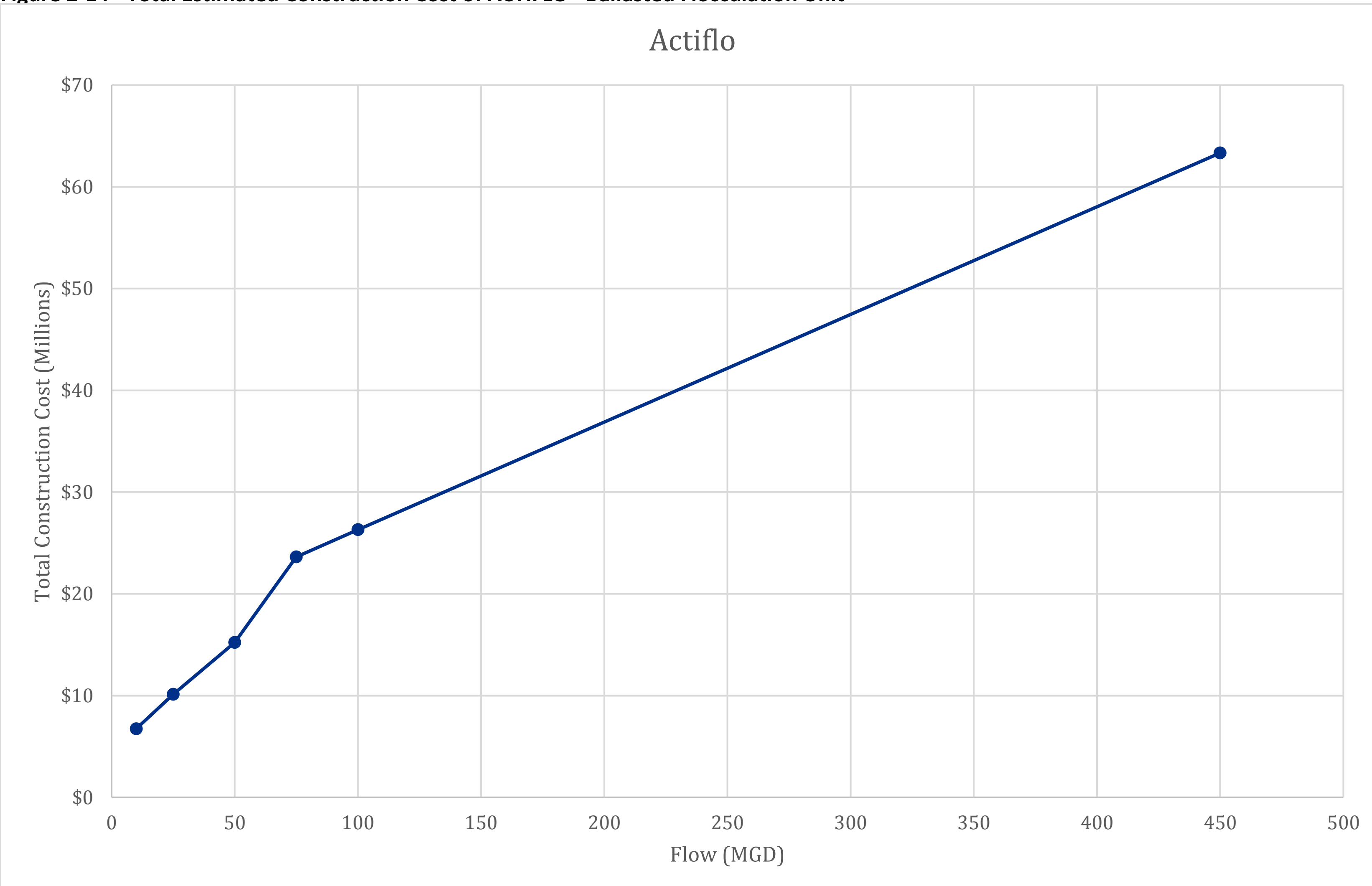


Table 2-19 - Annual Operation Cost of ACTIFLO® Ballasted Flocculation

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.

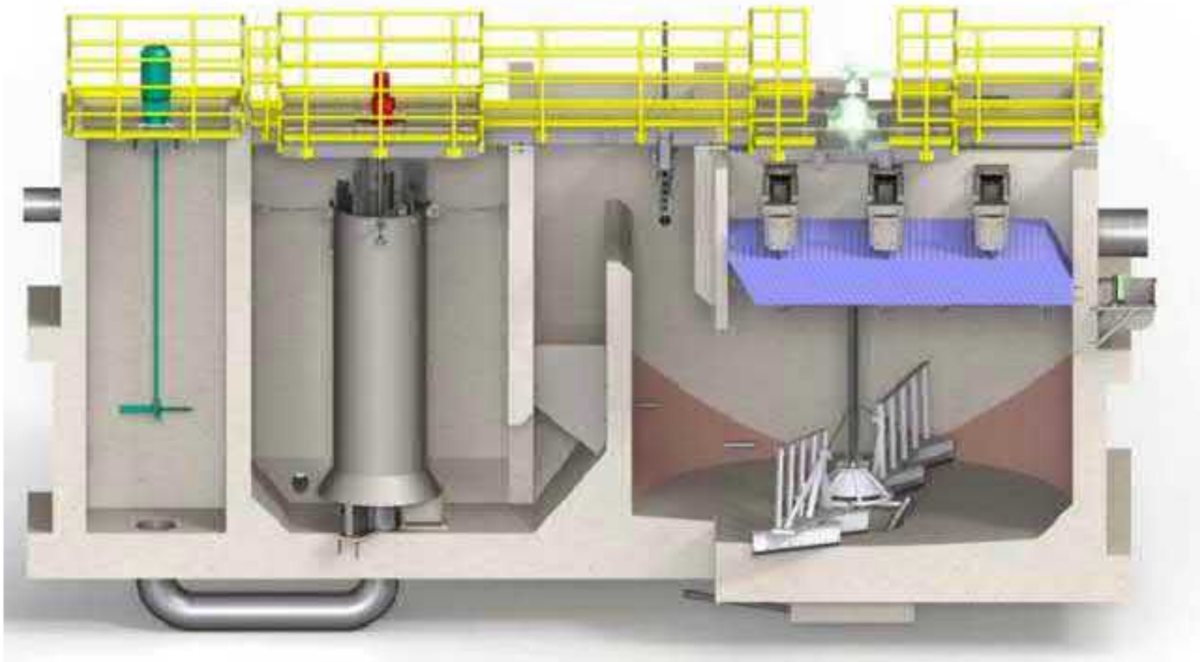
4. Settling & clarification zone: Flocculated solids enter the settling zone, over a submerged weir wall, where dense, suspended matter settles to the bottom of the clarifier. Clarified water is displaced upward from the downward moving slurry, through inclined plate settlers. The plate modules act as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

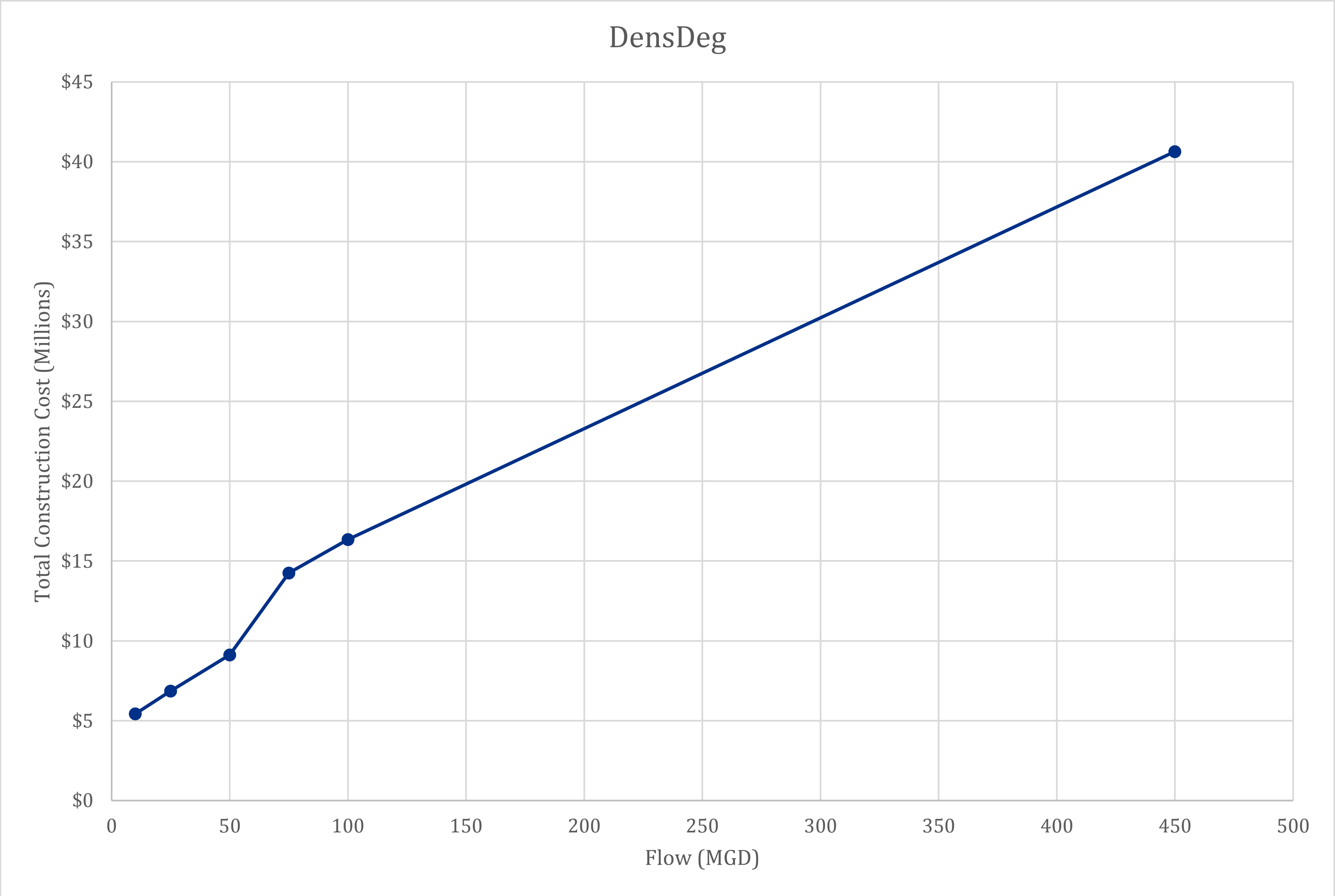


Table 2-22 - Annual Operation Cost of DensaDeg Ballasted Flocculation Unit

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

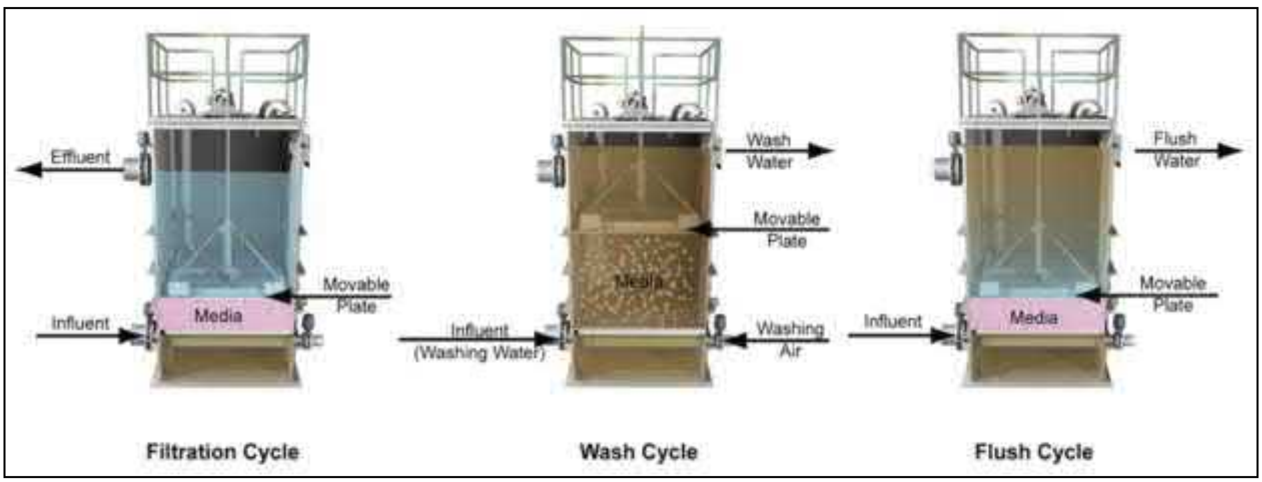
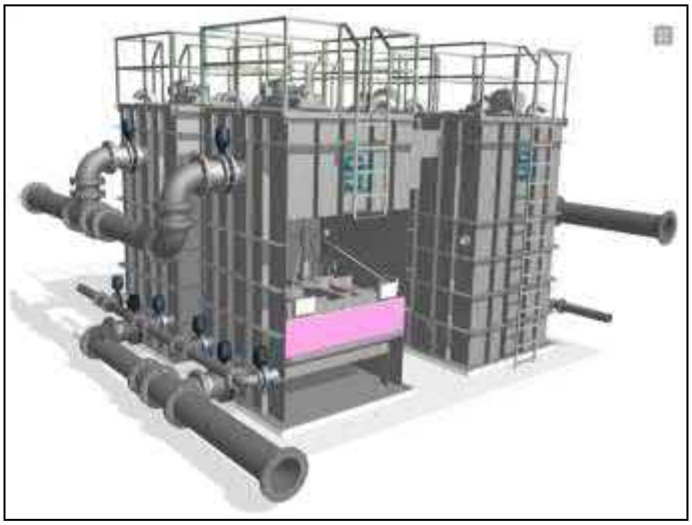


Figure 2-18 - Fuzzy Filter Unit

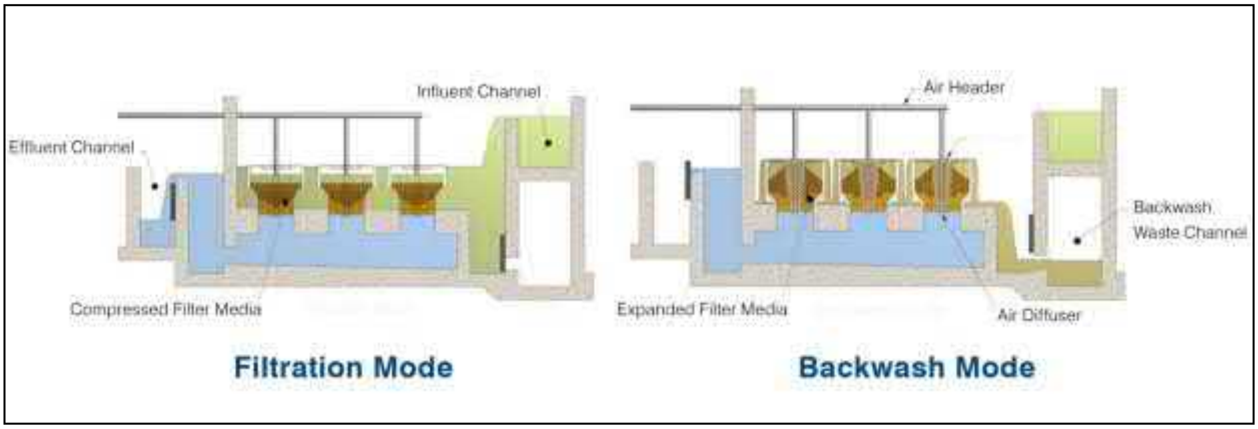


(Source: Schreiber, LLC.)

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.

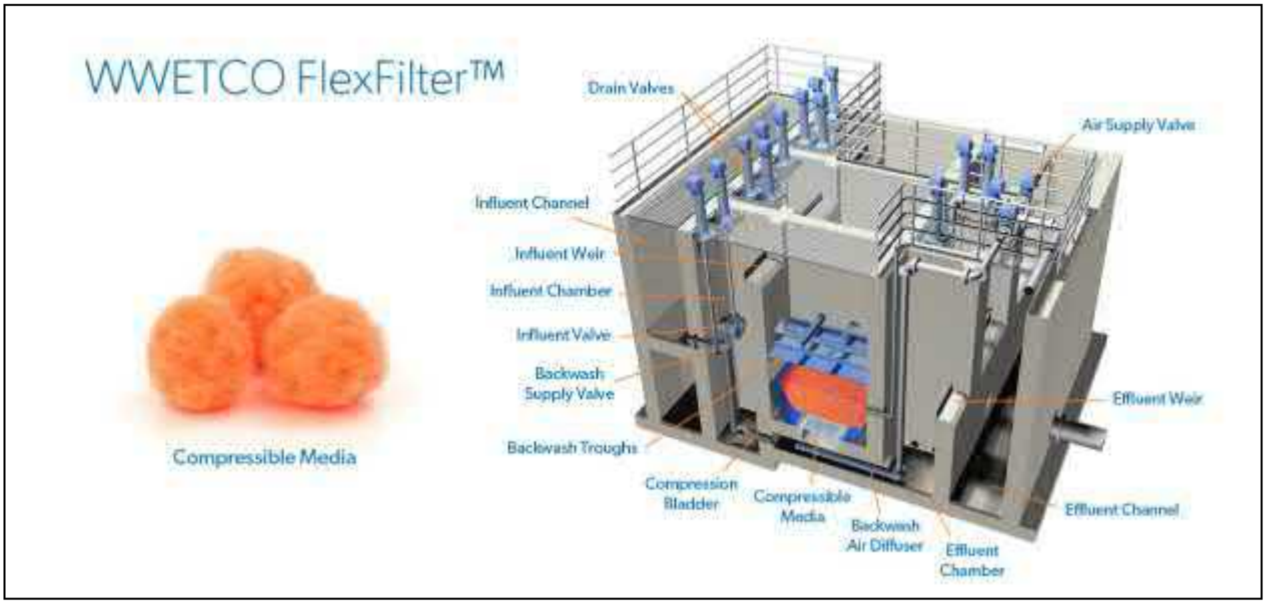
The filters use air scouring in the wash cycle to clean the media. During the wash cycle, the feed to the filter is stopped, allowing the media to uncompress. The air scour is initiated along with a small amount of backwash water. The length of the backwash cycle is adjustable. Once cleaned, the filter 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

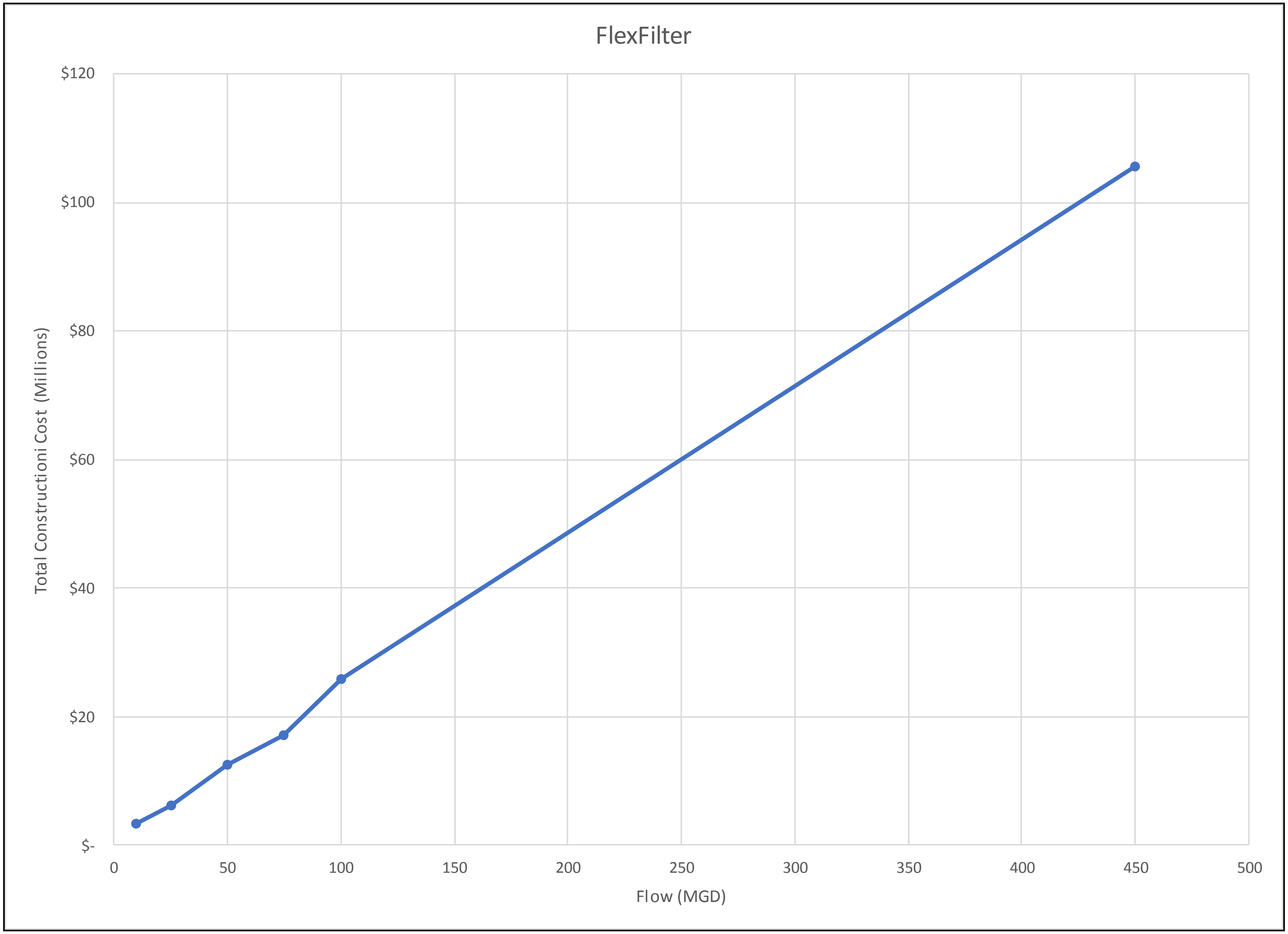


Table 2-25 - Annual Operation and Maintenance Cost of FlexFilter

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

	Vortex Separator		Modified Vortex	Ballasted Flocculation		Polishing Filter
Criteria	**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

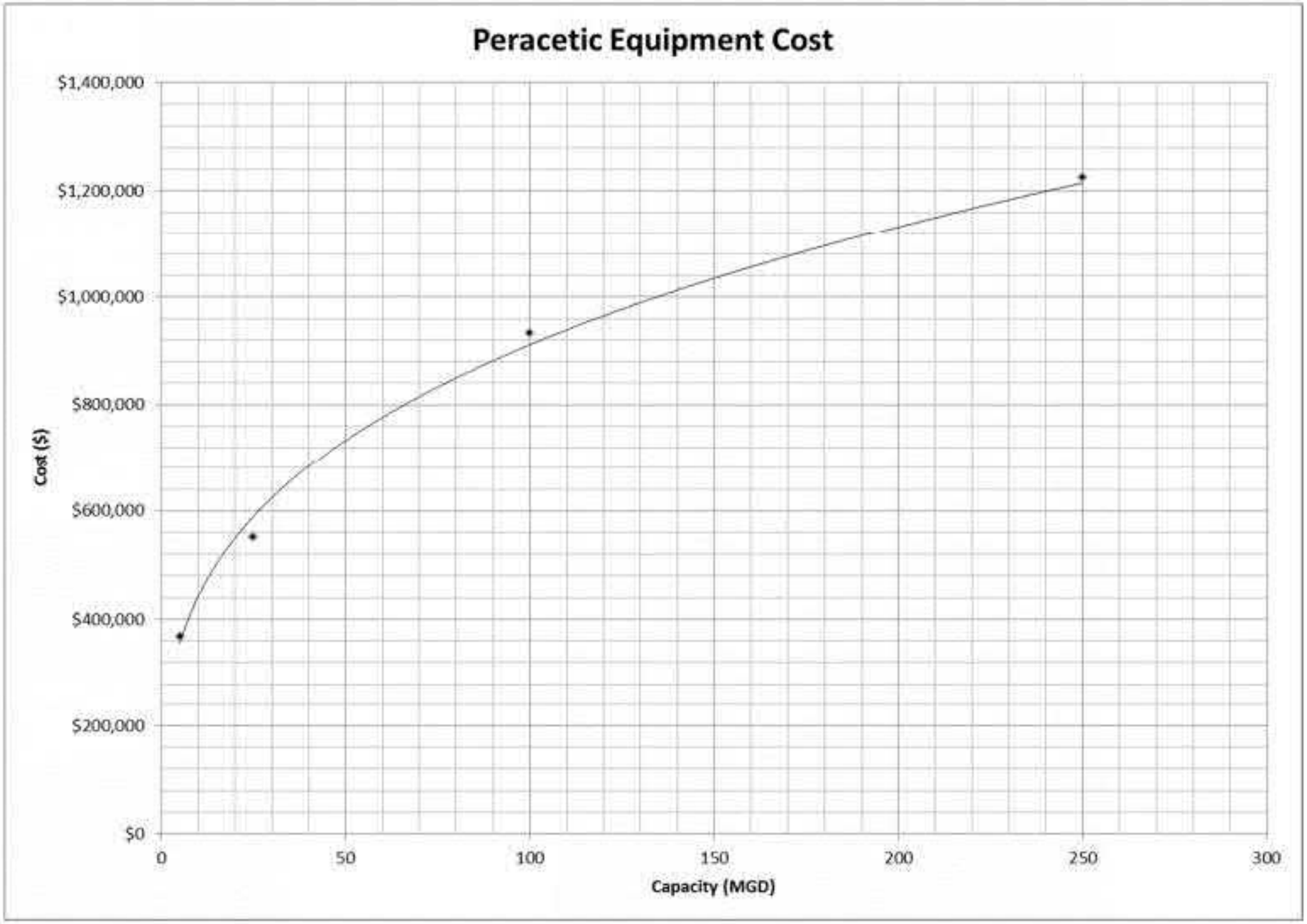
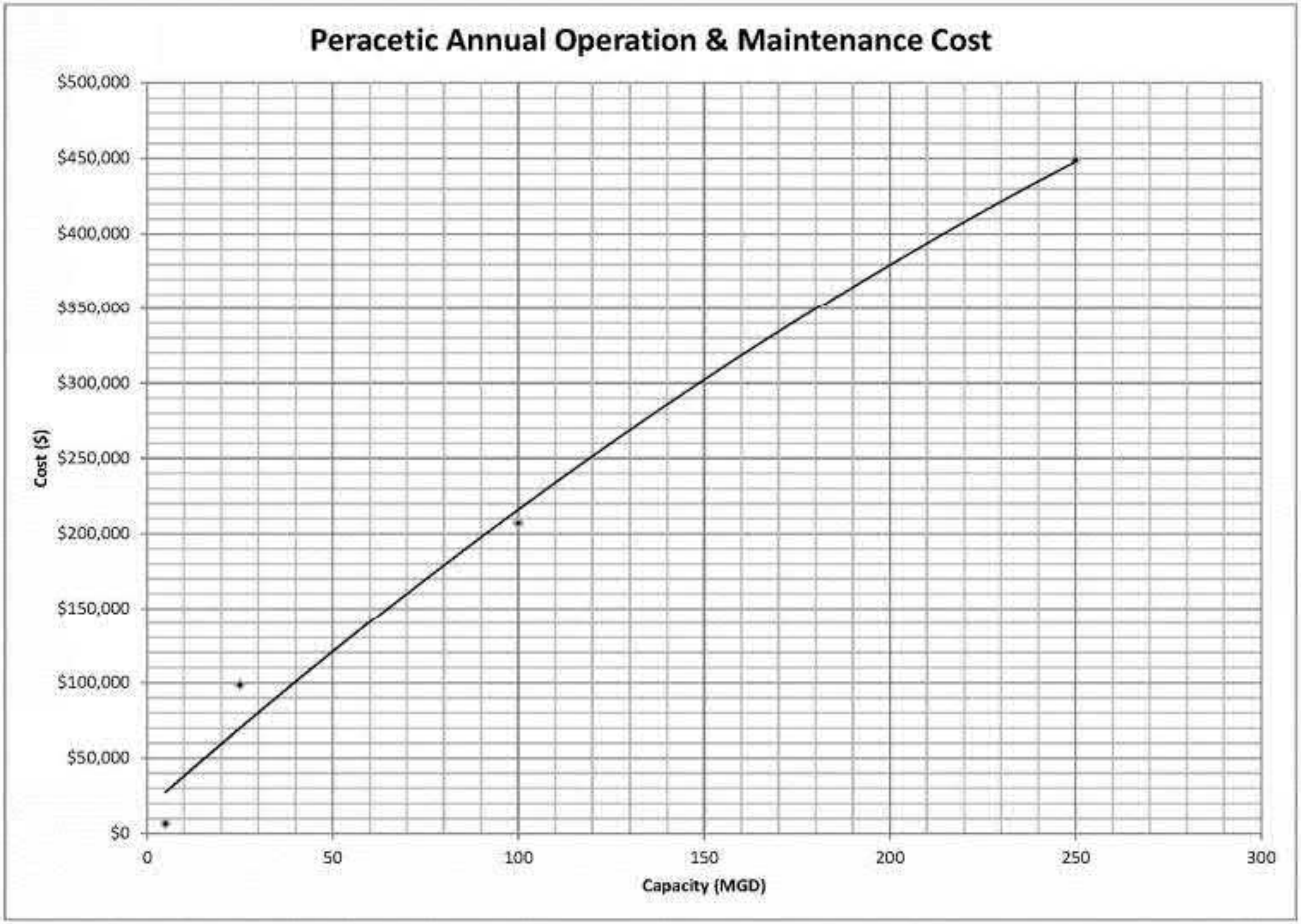


Figure 2-23 - Annual O&M Cost for Peracetic Acid System



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^{o}C. LP-HO lamps operate at temperature range of 100 -150^{o}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^{o}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.

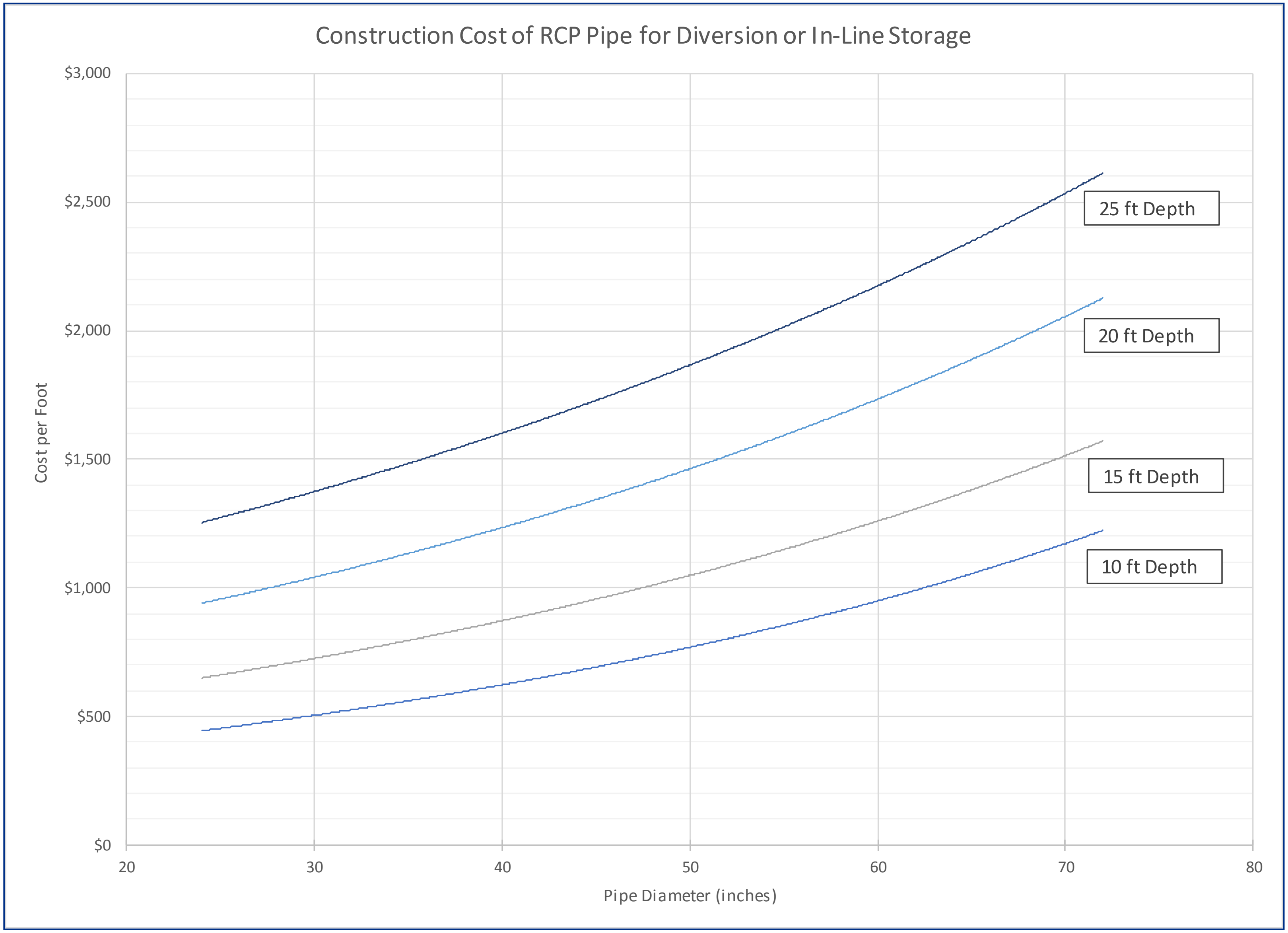
A CSO Facility Plan was completed by Killam Associates (now Mott MacDonald) in 1983 for the Passaic Valley Sewerage Commissioners on the combined sewer systems within the Cities of Newark and Paterson, and Towns of Harrison and Kearny, and the Borough of East Newark. The evaluation of in-line storage 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

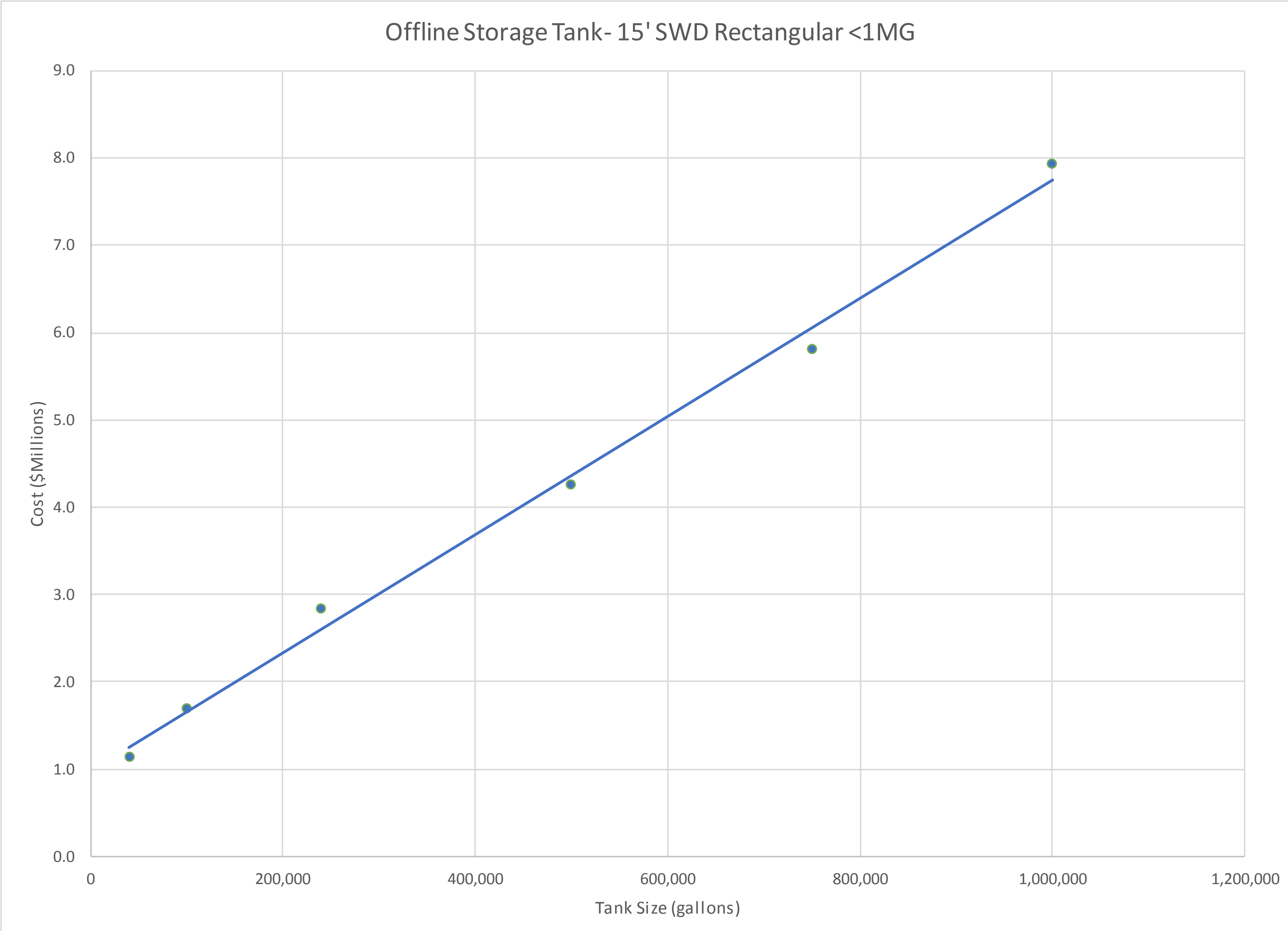


Figure 3-3 - Construction Cost Estimates for Off-Line Storage – 15’ SWD Rectangular > 1 MG

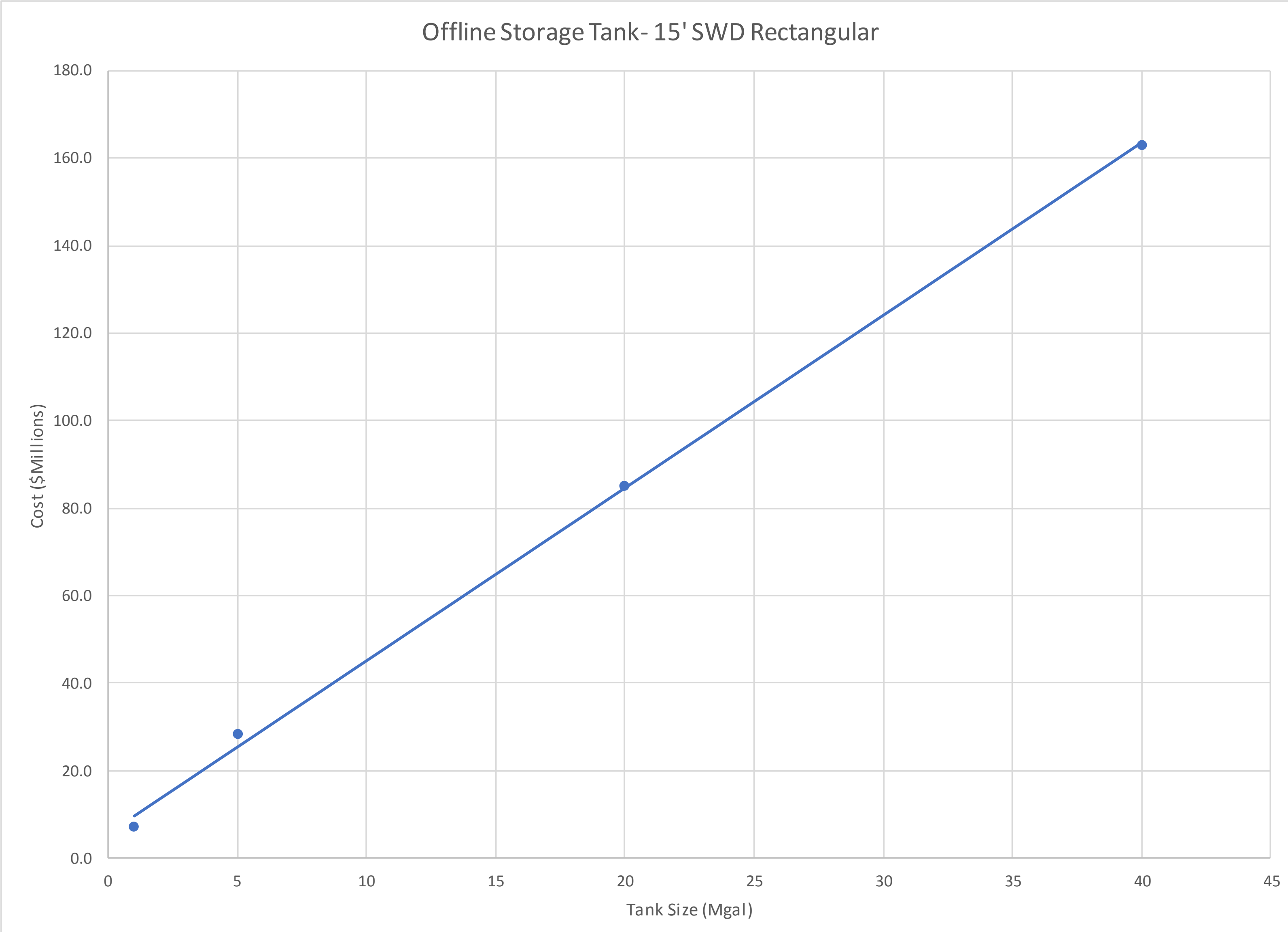


Figure 3-4 - Construction Cost Estimates for Off-Line Storage – 22’ SWD Rectangular

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

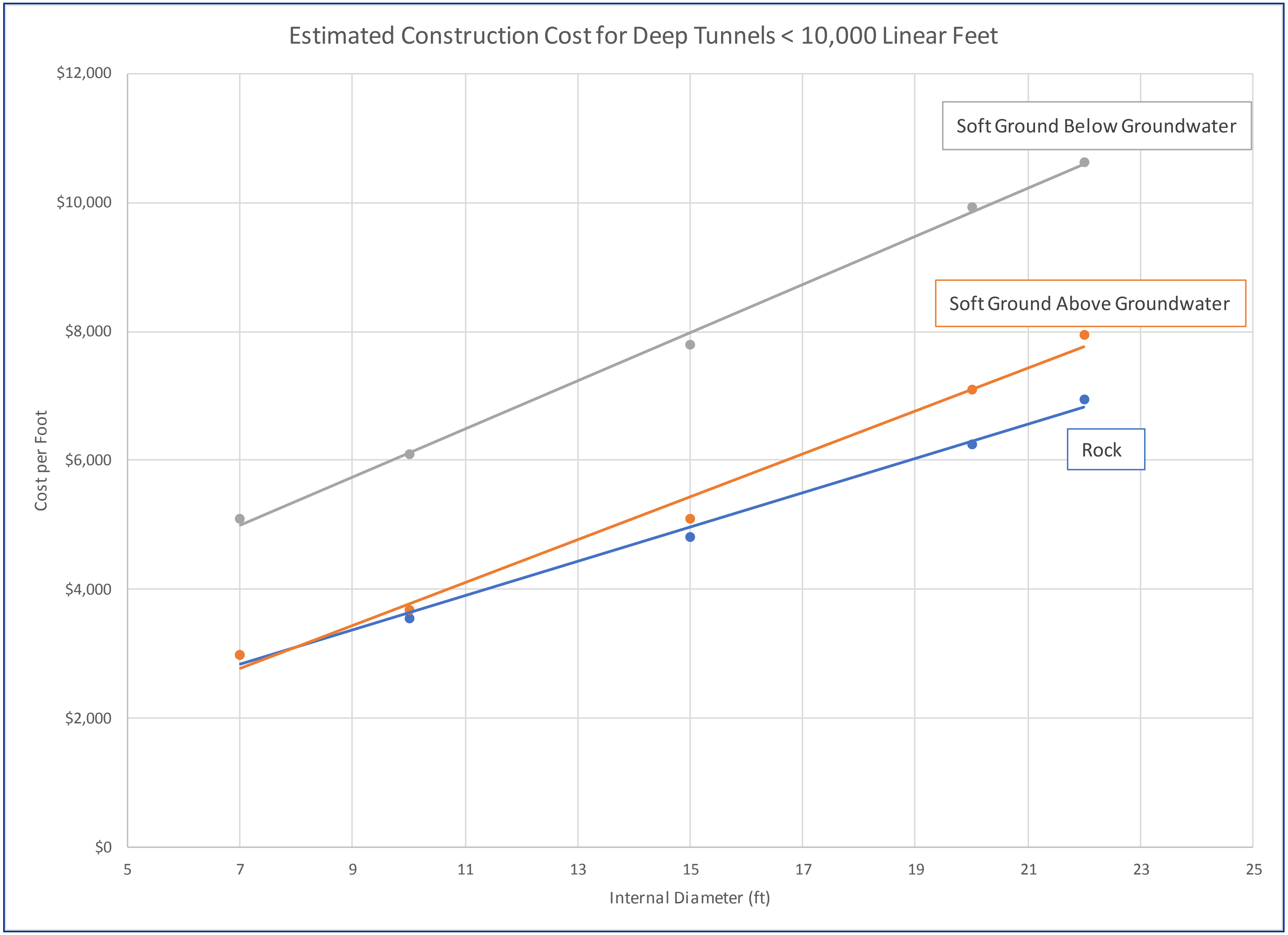


Figure 3-7 - Estimated Cost of Deep Tunnels Greater Than 10,000 Linear Feet

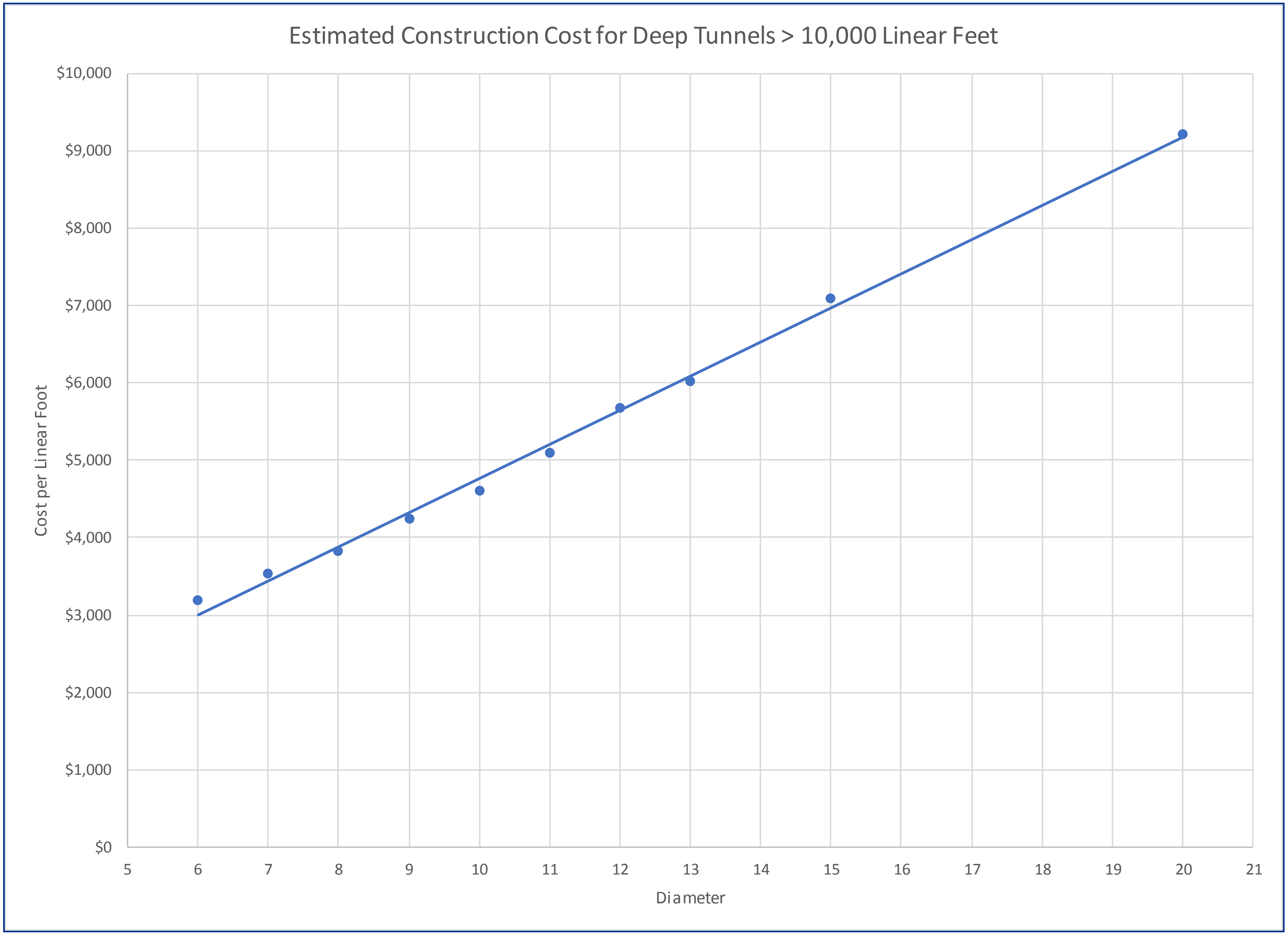
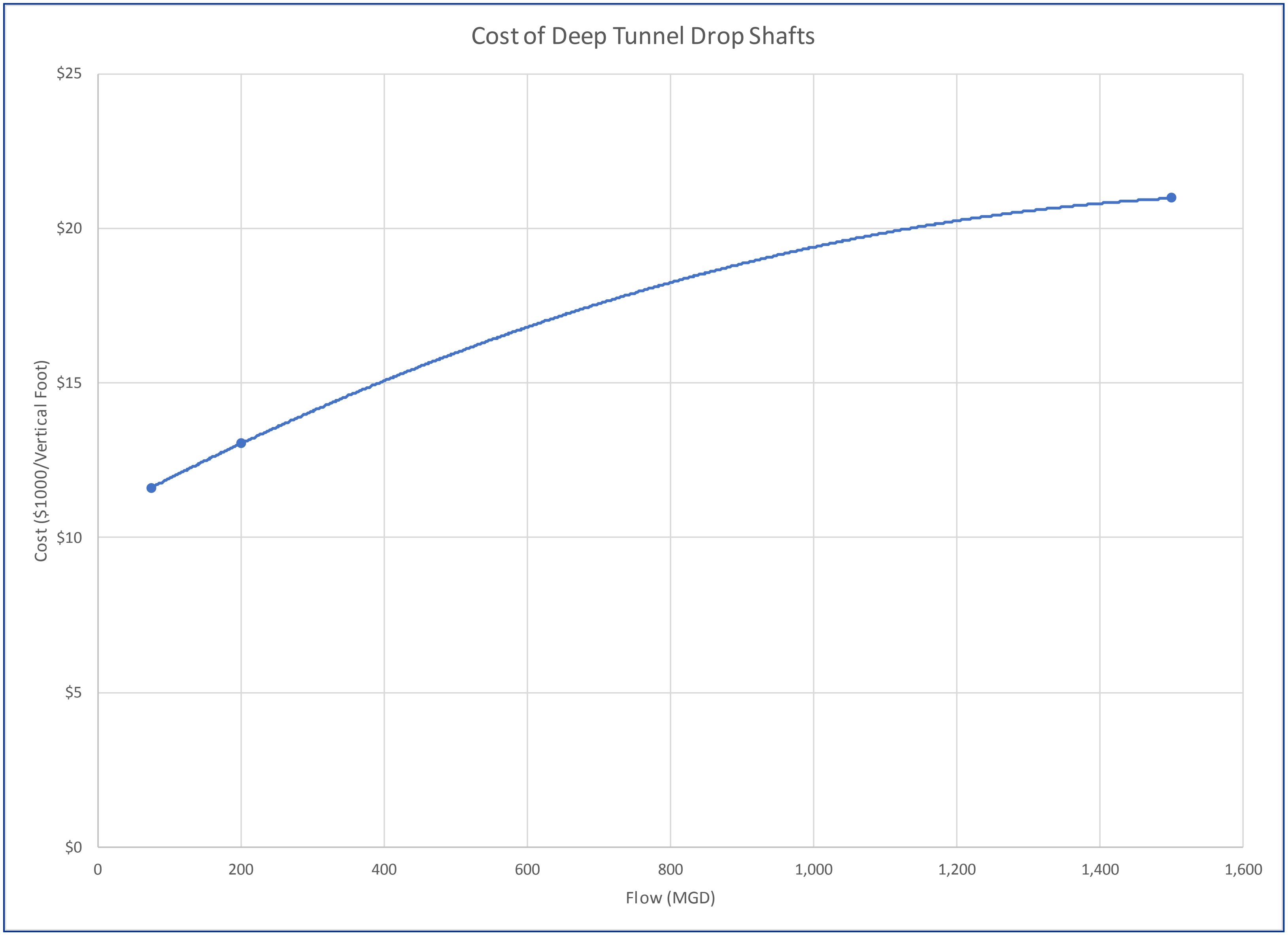


Figure 3-8 - Construction Cost Estimates for Tunnel Drop Shaft



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

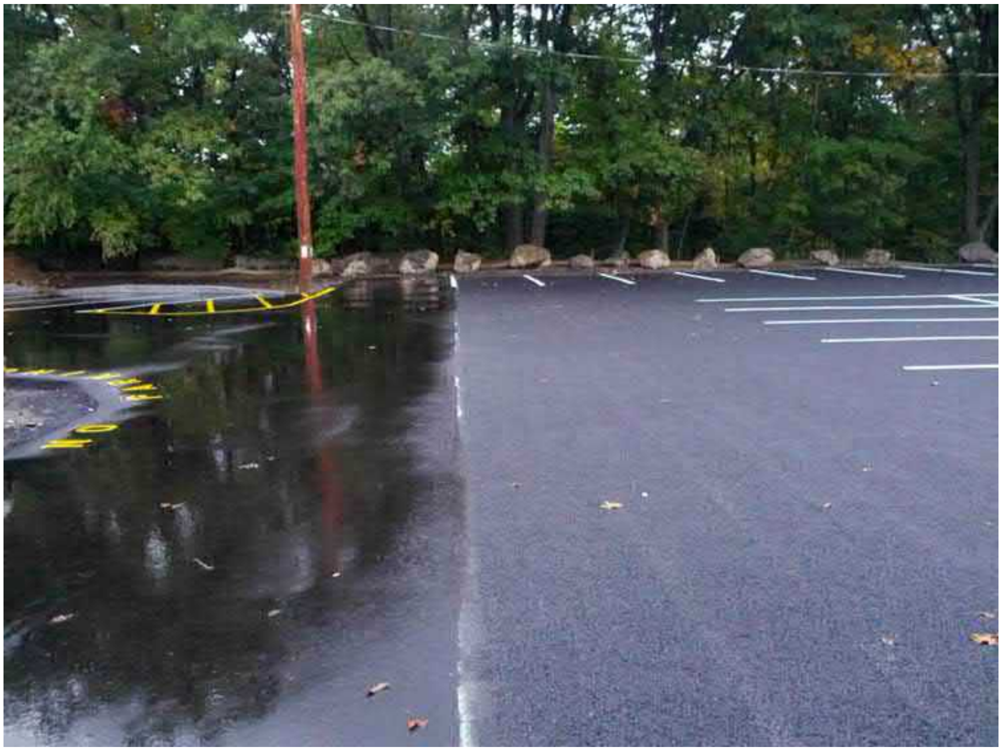
Description of Practice

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

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-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

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-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

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-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

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-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

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^{TM} Installation List

(Source: WesTech Engineering, Inc.)

WESTECH

Installation List
ROMAG CSO SCREENS

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)

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**					

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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™ 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

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

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