DEVELOPMENT & EVALUATION OF ALTERNATIVES REPORT

Camden County Municipal Utilities Authority
City of Camden
City of Gloucester

NJPDES Permit Nos.

NJ0026182

NJ0108812

NJ0108847

June 2019

cdmsmith.com

Intermunicipal Agreements

Andy Kricun

Resolution of

THE CAMDEN COUNTY MUNICIPAL UTILITIES AUTHORITY

Authorizing an Intermunicipal Agreement With the Cities of Camden and Gloucester for Preparation of a NJDEP-Required Combined Sewer Overflow System Management Plan

#R-13:7-103

Whereas, the New Jersey Department of Environmental Protection has promulgated new regulations for combined sewer overflow systems that require the CCMUA, Camden City and Gloucester City to develop a new Combined Sewer Overflow System Management Plan for the overall system that comprises the CCMUA's wastewater treatment plant, Camden City's combined sewer overflow system and Gloucester City's combined sewer overflow system; and

Whereas, the interconnectedness of these three systems dictates that one plan addresses all three systems; and

Whereas, accordingly, the CCMUA, Camden City and Gloucester City have negotiated an intermunicipal agreement which calls for the CCMUA to prepare the NJDEP-required plan for the three systems, while reaffirming the Cities' ongoing responsibility to own, operate and maintain their own systems.

Now, Therefore Be It Resolved by the CCMUA Board of Commissioners that it authorizes execution of an intermunicipal agreement with the Cities of Camden and Gloucester for preparation of a NJDEP-required Combined Sewer Overflow System Management Plan

ADOPTED: July 15, 2013

Kim Michelini, Authority Secretary

I hereby certify that the foregoing is a true copy of the Resolution adopted by the members of the Camden County Municipal Utilities Authority at a meeting held on July 15, 2013.

Intermunicipal Agreement Among Camden County Municipal Utilities Authority, Camden City and Gloucester City For Completion of NJDEP-Required Planning For Combined Sewer Systems

Whereas, the New Jersey Department of Environmental Protection has promulgated new requirements for combined sewer overflow systems which include the requirement that owners of combined sewer systems must develop new planning documents to demonstrate best management practices and minimization of combined sewer overflows, and

Whereas, accordingly the Camden County MUA, Camden City and Gloucester City agree as follows:

1) The Camden County MUA will complete the planning requirements for the Cities of Camden and Gloucester at its own cost. Camden City and Gloucester City will provide as much of the required background information as possible in order to assist the CCMUA in completing the required planning documents.

2) Although the CCMUA is undertaking this planning study on behalf of Camden City and Gloucester City at its own cost, all parties agree that this does not, in any way, alleviate Camden City and Gloucester City's ongoing responsibilities to own, operate and maintain their combined sewer systems, or to undertake any improvements that may be required as a result of the aforementioned planning study or any other NJDEP requirements.

IN WITNESS WHEREOF, the parties hereto have made and executed this Agreement and affixed their corporate seals as of the day and year first above written.

CAMDEN COUNTY MUNICIPAL UTILITIES AUTHORITY
OWNER

BY: Michael Brennan 8/19/13
Michael G Brennan, Chairman *Date*

ATTEST:
Kim [signature]

CAMDEN CITY

BY: Dana L. Redd 7/29/13
Name/Title: Dana L. Redd Mayor Date

ATTEST:
[signature]

GLOUCESTER CITY

BY: [signature]
Name/Title: Date

ATTEST:
Kathleen Jentsch

THE CAMDEN COUNTY MUNICIPAL UTILITIES AUTHORITY

RESOLUTION OF THE CITY OF GLOUCESTER CITY

#R 159-2013

RESOLUTION AUTHORIZING INTERLOCAL AGREEMENT BETWEEN CAMDEN COUNTY MUNICIPAL UTILITIES AUTHORITY, CAMDEN CITY AND GLOUCESTER CITY FOR COMPLETION OF NJDEP-REQUIRED PLANNING FOR COMBINED SEWER SYSTEMS

WHEREAS, the New Jersey Department of Environmental Protection has promulgated new requirements for combined sewer overflow systems which include the requirement that owners of combined sewer systems must develop new planning documents to demonstrate best management practices and minimization of combined sewer overflow; and

WHEREAS, accordingly the Camden County MUA, Camden City and Gloucester City agree as follows:

1) The Camden County MUA will complete the planning requirements for the Cities of Camden and Gloucester at its own cost. Camden City and Gloucester City will provide as much of the required background information as possible in order to assist the CCMUA in completing the required planning documents.
2) Although the CCMUA is undertaking this planning study on behalf of Camden City and Gloucester City at its own cost, all parties agree that this does not, in any way, alleviate Camden City and Gloucester City's ongoing responsibilities to own, operate and maintain their combined sewer systems, or to undertake any improvements that may be required as a result of the aforementioned planning study or any other NJDEP requirements.
3) Gloucester City will review, approve all plans and deliverables prior to any submittal.

William P. James, Mayor

Passed by the Mayor and Common Council of Gloucester City this 27th day of June, 2013.

It is hereby certified that the foregoing is a true and correct copy of a resolution duly adopted by the City of Gloucester City, in the County of Camden, at a meeting held on 6-27-13

Kathleen M. Jentsch, RMC

Kathleen M. Jentsch, City Clerk

RESOLUTION MC-13: 3188
On Motion Of: Dana M. Burley
APPROVED: June 11th, 2013

R-64

MAR:dh
06-11-13

RESOLUTION AUTHORIZING A SHARED SERVICES AGREEMENT BETWEEN THE CITY OF CAMDEN, THE CITY OF GLOUCESTER AND THE CAMDEN COUNTY MUNICIPAL UTILITIES AUTHORITY FOR THE PLANNING OF THE NEW PERMIT REQUIREMENTS FOR THE COMBINED SEWAGE OVERFLOW SYSTEM

WHEREAS, the New Jersey Department of Environmental Protection has promulgated new requirements for combined sewer overflow systems which include the requirement that owners of combined sewer systems must develop new planning documents to demonstrate best management practices and minimization of combined sewer overflows; and

WHEREAS, accordingly the Camden County MUA, Camden City and Gloucester City agree as follows:

SECTION 1. The Camden County MUA will complete the planning requirements for the Cities of Camden and Gloucester at its own cost. Camden City and Gloucester City will provide as much of the required background information as possible in order to assist the CCMUA in completing the required planning documents.

SECTION 2. Although the CCMUA is undertaking this planning study on behalf of Camden City and Gloucester City at its own cost, all parties agree that this does not, in any way, alleviate Camden City and Gloucester City's ongoing responsibilities to own, operate and maintain their combined sewer systems, or to undertake any improvements that may be required as a result of the aforementioned planning study or any other NJDEP requirements.

NOW THEREFORE, BE IT RESOLVED by the City Council of the City of Camden that a Shared Services Agreement is hereby authorized between the City of Camden and the City of Gloucester and the Camden County Municipal Utilities Authority for the Planning of the new permit requirements for the Combined Sewage Overflow System.

BE IT FURTHER RESOLVED, that pursuant to N.J.S.A. 52:27BBB-23, a true copy of this Resolution shall be forwarded to the State Commissioner of Community Affairs, who shall have ten (10) days from the receipt thereof to veto this Resolution. All notices of veto shall be filed in the Office of the Municipal Clerk.

Date of Introduction June 11, 2013

The above has been reviewed
and approved as to form.

Michelle Banks-Spearman
For MARC A. RIONDINO
City Attorney

FRANCISCO MORAN
President, City Council

ATTEST:

LUIS PASTORIZA
Municipal Clerk

I, LUIS PASTORIZA, MUNICIPAL CLERK OF THE CITY OF CAMDEN, DO HEREBY CERTIFY, that the foregoing is a true copy of a resolution entitled "**Resolution authorizing a shared services agreement between the City of Camden, The City of Gloucester and the Camden County Municipal Utilities Authority for the planning of the new permit requirements for the combined sewage overflow system** "**ADOPTED** by the Council of the City of Camden, New Jersey, the **11th** day of **June 2013** as taken from and compared with the orignal now on file in my office.

IN TESTIMONY WHEREOF, I have thereunto set my hand and affixed seal of the City of Camden, at this **31st** day of **July, 2013.**

Luis Pastoriza, Municipal Clerk

N.J.A.C. 7:14A-4.9 Certifications

Camden County Municipal Utilities Authority
NJPDES Permit NJ0026182 Submittal
N.J.A.C 7:14A-4.9 Certification Form

Pursuant to the requirements under NJPDES Permit NJ0026182, the Camden County Municipal Utilities Authority (CCMUA) is submitting the following document(s)

Development of Alternatives and Evaluation Report
(Title of Document)

As required under Part IV - Combined Sewer Management Paragraph D.1(c) (Submittals), the Authority is providing the following certification:

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

Andrew Kricun, P.E., BCEE
Executive Director/Chief Engineer
Camden County Municipal Utilities Authority

Signature

6/24/19

Date

Camden County Municipal Utilities Authority
NJPDES Permit NJ0026182 Submittal
N.J.A.C 7:14A-4.9 Certification Form

Pursuant to the requirements under NJPDES Permit NJ0108812, the City of Camden, New Jersey is submitting the following document(s):

Development of Alternatives and Evaluation Report
(Title of Document)

As required under Part IV - Combined Sewer Management Paragraph D.1(c) (Submittals), the Authority is providing the following certification:

> "I certify under penalty of law that this document and all attachments were prepared either: (a) under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted; or (b) as part of a cooperative effort by members of a hydraulically connected system, as is required under the NJPDES Permit, to provide the information requested. Based on my inquiry of the person or persons who manage the system, or those persons directly responsible for gathering the information, the information submitted is, to the best of my knowledge and belief, true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for purposely, knowingly, recklessly, or negligently submitting false information".

Name:____, Title:____________________________________
City of Camden, New Jersey

____
Signature

____
Date

Camden County Municipal Utilities Authority
NJPDES Permit NJ0026182 Submittal
N.J.A.C 7:14A-4.9 Certification Form

Pursuant to the requirements under NJPDES Permit NJ 0108847, Gloucester City, New Jersey is submitting the following document(s):

Development of Alternatives and Evaluation Report
(Title of Document)

As required under Part IV - Combined Sewer Management Paragraph D.1(c) (Submittals), the Authority is providing the following certification:

> "I certify under penalty of law that this document and all attachments were prepared either: (a) under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted; or (b) as part of a cooperative effort by members of a hydraulically connected system, as is required under the NJPDES Permit, to provide the information requested. Based on my inquiry of the person or persons who manage the system, or those persons directly responsible for gathering the information, the information submitted is, to the best of my knowledge and belief, true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for purposely, knowingly, recklessly, or negligently submitting false information".

Name: Jack Lipsott, Title: City Administator
Gloucester City, New Jersey

Signature

6/25/19
Date

DEVELOPMENT & EVALUATION OF ALTERNATIVES REPORT

Camden County Municipal Utilities Authority
City of Camden
City of Gloucester

NJPDES Permit Nos.

NJ0026182

NJ0108812

NJ0108847

June 2019

CDM Smith
cdmsmith.com

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Appendices

Executive Summary

E.1 Introduction

This document constitutes Camden County Municipal Utilities Authority's (CCMUA) *Development and Evaluation of Alternatives Report* (DEAR) developed by CCMUA on behalf of CCMUA, the City of Camden and Gloucester City (the Cities) for the required "Evaluation of Alternatives" under Part IV Section G.4 of their respective New Jersey Pollutant Discharge Elimination System permits.

Overview of Findings:

- Green stormwater infrastructure (GSI) for source reduction is foundational.
- The control performance target will be 85% capture of wet weather flows entering the combined sewer system on a system-wide annual basis.
- Expanding WPCF No.1 to 220 MGD coupled with 10% greening, completion of the cleaning of the Camden collection system, subsequent triennial cleaning and maintenance of equipment and regulators would bring the system-wide capture rate to 83% and will reduce street flooding.
- Depending on the rate of GSI installations, this level of control could be achieved during initial phases of the LTCP implementation.
- Additional controls will be required to achieve the 85% system-wide annual capture target.
- Street flooding volume will be reduced by an estimated 61%.
- Preliminary system-wide cost estimates range between $200 million to $390 million (present worth) including the costs of expanding WPCF No.1 treatment capacity to 220 MGD and the 10% greening.
- The control alternatives and cost estimates will be refined moving forward and subject to affordability, financial capability and other "reality checks".
- The Final Long Term Control Plan will feature phased implementation and adaptive management in coordination with NJPDES permit cycles. Initial phases will focus on ongoing collection system rehabilitation, the expansion of wet weather treatment capacity and implementation of green infrastructure.

E.2 Control Alternatives Baseline

The approved System Characterization Report (SCR) established that future CSO controls developed in this DEAR would be based upon what was termed as the Control Alternatives Baseline (CAB). The CAB is premised on two activities that are currently underway:

1. The expansion of CCMUA's WPCF No.1 to provide 185 million gallons per day (MGD) wet weather treatment capacity; and
2. The completion of the cleaning of Camden's collection system and CSO outfalls and ongoing recleaning at least every three years to restore their hydraulic capacities.

Subsequent to the establishment of the CAB in the July 2018 SCR, CCMUA progressed with its consideration of the subsequent expansion of wet weather treatment capacity to 220 MGD. The potential expansion to 220 MGD would be accomplished by bypassing primary effluent exceeding 185 MGD around secondary treatment where it would be disinfected and blended with the disinfected secondary treatment effluent prior to discharge into the Delaware River in accordance with a NJPDES permit.

In keeping with CCMUA's and the City of Camden's recognized long term commitment to sustainable green redevelopment, CCMUA and the Cities have established a target of controlling runoff from 10% of the directly connected impervious area (DCIA) as a foundational element of any long term CSO control plan. Therefore, the control alternatives developed in this DEAR are premised on:

- The Control Alternatives Baseline (wet weather treatment capacity of 185 MGD + the restoration of the design hydraulic capacity of the Camden combined sewer system; plus
- The further expansion of wet weather treatment capacity at CCMUA's Water Pollution Control Facility (WPCF) No. 1 to 220 MG; plus
- The phased implementation of green stormwater infrastructure projects to eventually control the runoff from 10% of the impervious area that is directly connected to the combined sewer system.

The system performance characteristics to be used as a starting point for control alternatives under the Control Alternatives Baseline are summarized on Table E-1. As may be noted, additional controls beyond those shown in the table will be required to achieve system-wide 85% capture of wet weather flows entering the combined sewer system.

Table E-1 – System Wide Performance Characteristics Used for the Development of Further Control Alternatives

Performance Metric		Existing Conditions	Control Alternatives Baseline	Add Additional Wet Weather Treatment Capacity*	Add 10% Control of Directly Connected Impervious Area Using GSI*
Treatment capacity (MGD)		150	185	220	220
Camden hydraulic capacity restored		No	Yes	Yes	Yes
Controlling Runoff from 10% of DCIA		<10%	<10%	<10%	10%
1	% Capture	68%	76%	79%	83%
2	Overflow Volume (MGY)	829	627	541	453
3	Overflow Frequencies	69-10	71-5	71-9	71-9
4	Surface Flooding (MGY)	90	44	43	35

*Also includes regulator C3 opened to utilized expanded wet weather treatment capacity.

E.3 Hydraulically Connected Sub-Systems

For purposes of developing control strategies, the 30 active outfalls within the combined sewer system have been divided into hydraulically connected and geographically proximate sub-systems as shown on Figure E-1. The approximate configurations of the CSO groupings are shown on Figure E-1 (following page).

E.4 Prior and Ongoing CSO Control Efforts

The development of CSO control alternatives documented in this report has been conducted in the context of prior and ongoing CSO control efforts that have been undertaken by CCMUA and the Cities. These efforts include:

- Implementation of the Nine Minimum Controls;
- Green Stormwater Infrastructure;
- Screening and Netting Facilities;

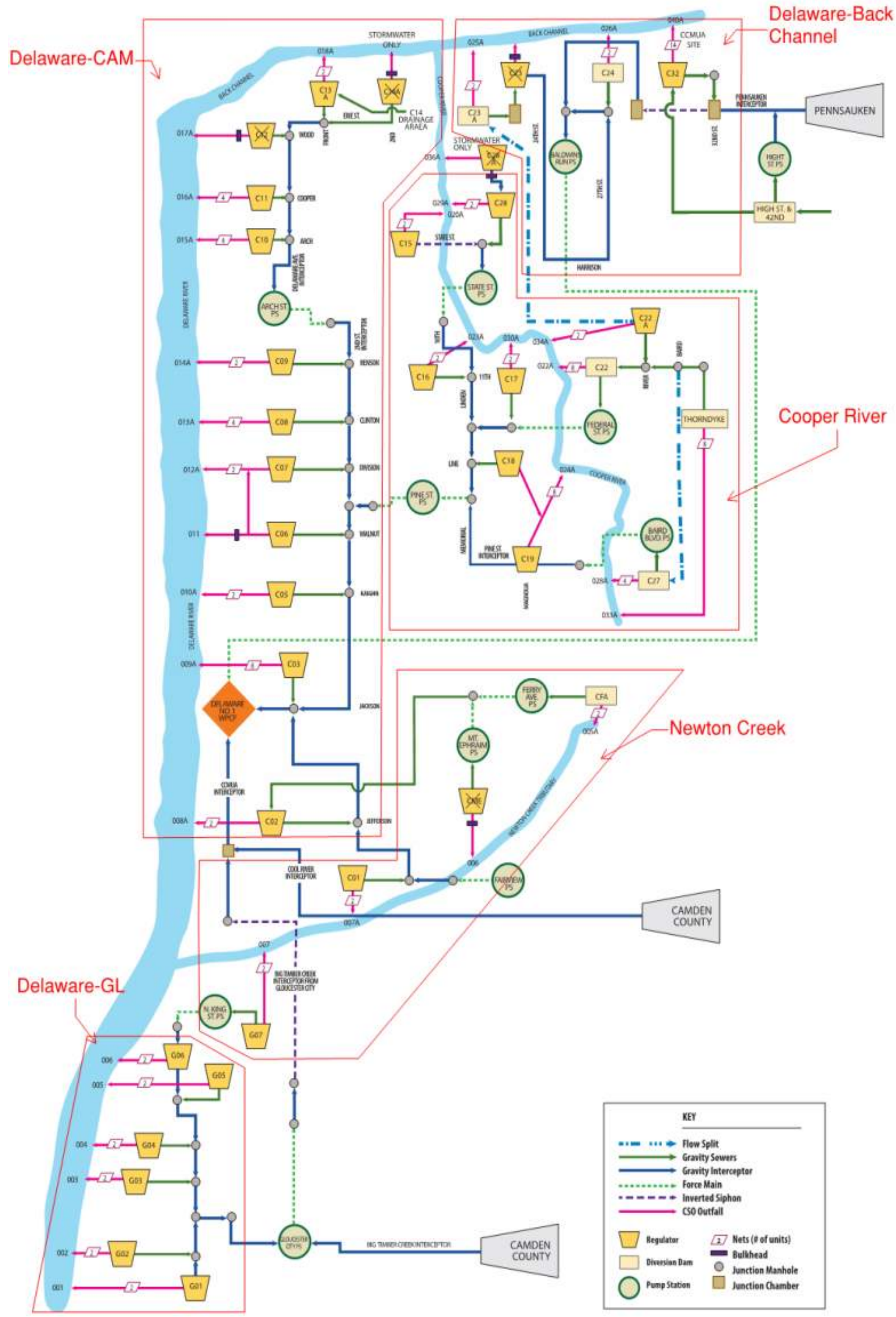

Figure E-1 – Logical Groupings of Outfalls

- Wet Weather Treatment Capacity Expansion at CCMUA's WPCF No. 1;
- Stormwater Management Improvement Projects;
- Arch Street Pump Station Improvements; and
- City of Camden Sewer & Outfall Cleaning Program

E.5 Selection of Control Objective

CCMUA and the Cities have committed to develop a final Long Term Control Plan (FLTCP) that will meet the requirements of the NJPDES permits through the elimination or the capturing for treatment of no less than 85% by volume of the combined sewage generated in the CSS during wet weather on a hydraulically connected system-wide annual average basis.

E.6 Control Technology Initial Screening

Potential control technologies are described and screened in terms of their applicability to CCMUA, Camden and Gloucester in Table E-2.

Table E-2 – Summary of Initial Screening

<table>
<tr><th rowspan="2">Alternative</th><th colspan="3">Applicability</th><th rowspan="2">Comments</th></tr>
<tr><th>System Wide</th><th>Local</th><th>Not Applicable</th></tr>
<tr><td rowspan="2">Green Stormwater Infrastructure</td><td colspan="4">Green stormwater infrastructure (GSI) encompasses a range of land-based stormwater management techniques and physical reconstruction of aquatic habitats where appropriate. GSI includes but is not limited to such technologies such as rain gardens, bioswales, permeable pavement and green roofs.</td></tr>
<tr><td>X</td><td></td><td></td><td>- Lead element in control strategy
- Assuming Controlling Runoff from 10% of DCIA</td></tr>
<tr><td>Increased System Storage Capacity</td><td colspan="4">Wet weather flows are stored until the conveyance and treatment capacities are available. Wet weather flows can be stored in:
• Within the current sewer system;
• Within storage and conveyance tunnels;
• Within satellite storage facilities.</td></tr>
<tr><td>Inline Storage (within collection system)</td><td></td><td>X</td><td></td><td>- Important intrinsic storage in clean pipes, ongoing maintenance will remain critical.
- System-wide applicability limited
- Potential for surcharging will remain</td></tr>
<tr><td>Offline Storage – Tunnels</td><td></td><td></td><td>X</td><td>- Cost prohibitive
- Construction can be disruptive</td></tr>
</table>

<table>
<tr><th rowspan="2">Alternative</th><th colspan="3">Applicability</th><th rowspan="2">Comments</th></tr>
<tr><th>System Wide</th><th>Local</th><th>Not Applicable</th></tr>
<tr><td>Offline Storage – Tanks</td><td>X</td><td></td><td></td><td>- Proven CSO control technology
- Site and consolidation considerations affect practical applications and cost effectiveness.
- Expanded O&M responsibilities</td></tr>
<tr><td rowspan="2">WPCF # 1 Wet Weather Capacity Expansion</td><td colspan="4">The treatment capacities at an existing wastewater treatment plant is expanded to allow additional wet weather flows. This can involve the expansion of the existing process train or the addition of dedicated wet weather flow treatment facilities. If space is available, flow equalization storage basins can be used to store peak flows for later treatment as plant capacity is available.</td></tr>
<tr><td>X</td><td></td><td></td><td>– Expansion of WPFC # 1 to 185 MGD is currently underway.
– CCMUA is evaluating the expansion of capacity to 220 MGD of wet weather capacity.</td></tr>
<tr><td rowspan="2">I/I Reduction in Hydraulically Connected Upstream Sanitary Sewered Systems</td><td colspan="4">Hydraulically connected sanitary collection sewer systems are rehabilitated to reduce the quantities and rates of ground water infiltration and wet weather related inflow.</td></tr>
<tr><td></td><td>X</td><td></td><td>- Limited impact on CSO reduction due to relative volumes of wet weather flows generated in combined areas.
- Notwithstanding, eliminating excessive I/I in sanitary sewered areas important to preserve local hydraulic capacity and to reduce risks of sanitary sewer overflows upstream of the combined sewer system.</td></tr>
<tr><td rowspan="2">Sewer Separation</td><td colspan="4">Sewer separation is defined as the reconstruction of an existing combined sewer system into non-interconnected sanitary and storm sewer systems. Sanitary sewer systems are tributary to the wastewater treatment facility, while storm sewer systems discharge directly or indirectly (through detention ponds or other stormwater control facilities) to local receiving waters</td></tr>
<tr><td></td><td>X</td><td></td><td>- System-Wide separation would be cost prohibitive.
- Local separation may be appropriate during redevelopment projects.</td></tr>
</table>

Alternative	Applicability			Comments
	System Wide	Local	Not Applicable	
				- Potential appropriate for CSO control in physically or hydraulically remote sewersheds.
Treatment of CSO Discharge	Wet weather flows which would otherwise be discharged through overflow points receive the equivalent of primary treatment and disinfection prior to discharge at remote facilities. Treatment is typically through physical (e.g. vortex treatment units) or physical / chemical (e.g. ballasted flocculation) equipment.			
	X			- Proven CSO control technology - Site and consolidation considerations affect practical applications and cost effectiveness. - Provides disinfection – key WQ issue - Provides equivalent of primary treatment. - Expanded O&M responsibilities.
CSO Related Bypassing	Encouraged under the CSO Policy, this is a type of plant expansion utilizing primary treatment capacities which may exceed the treatment capacities of a wastewater treatment plant's secondary treatment process equipment. During wet weather all flow receive preliminary and primary treatment. Flows exceeding the plant's secondary treatment capacity are shunted around the secondary treatment process equipment. These bypassed flows are disinfected and discharged.			
	X			- CCMUA anticipates that expanding the WPCF wet weather capacity to 220 MGD using CSO related bypassing will be implemented.

E.7 Alternatives Development & Evaluation

E.7.1 Benefits of Expanding WPCF No. 1 Wet Weather Capacity

The wet weather treatment capacity of CCMUA's Delaware WPCF No. 1 is being expanded to 220 MGD in a two phase process. Phase 1 will result in an increase of wet weather treatment capacity to 185 MGD and is currently under construction. Phase 2 is under planning, design and regulatory review. Phase 2 will expand wet weather capacity an additional 40 MGD. This will be accomplished by the bypassing of primary treatment effluent exceeding the capacity of the secondary treatment process in accordance with a NJDPES permit to authorize the bypass.

The wet weather capacity expansion at the treatment plant coupled with the ongoing restoration of the collection system hydraulic capacities and source reduction through green stormwater infrastructure are anticipated to provide the following levels of CSO control performance projected in the near term (three to five years):

- System-wide typical year wet weather capture of 79% to 83% depending upon the rate of source reduction through green stormwater infrastructure;
- Typical year capture rates for Newton Creek and along the main channel of the Delaware River in Camden of 87% and 93% respectively;
- A reduction in annual overflow volumes of approximately 300 million gallons; and
- A reduction in surface flooding in the City of Camden by approximately 50% (not counting additional reductions attributable to the control of runoff from DCIA through green infrastructure).

The influent channel re-routing and the increase in wet weather treatment capacity at CCMUA's WPCF No. 1 which are underway will eliminate the current need to throttle back the Arch Street pump station during wet weather. This exacerbates surface flooding in the C-10 and C-11 sewersheds due in part to obstructions in CSO outfalls in the Camden collection system.

The projected decrease in street flooding assumes that outfall pipes conveying combined sewer overflows from the regulator structures to the receiving streams are operating at their respective design capacities.

As noted and detailed within this DEAR, additional controls will be necessary to achieve the performance target of 85% capture of wet weather flows during the typical year.

E.7.2 Subsystem Alternatives

The expansion of wet weather treatment capacity at WPFC No. 1 along with the restoration of the Camden collection system hydraulic capacity and controlling runoff from 10% of the directly connected impervious area will not achieve the 85% system-wide wet weather capture goal. Moreover, system hydraulics effectively preclude achieving the 85% capture target through the expansion of the treatment plant and source reduction alone. Therefore, either remote (satellite) facilities and/or expanded conveyance capacities will be required in some subsystems to meet regulatory control targets.

Delaware River – Camden

Expansion of the WPCF # 1 and attainment of GSI with the systemwide control of runoff from 10% of the DCIA will allow for the typical year wet weather capture of at least 85% in the Delaware River – Camden subsystem.

Delaware River – Gloucester

- A conveyance only 85% capture control option would be feasible through the operation of the Gloucester City PS to at 45 MGD during wet weather along with regulator modifications and interceptor upsizing. Potential impacts on other areas served by the Gloucester City PS will need to be evaluated and managed.

- A satellite treatment or storage facility for Gloucester City would be hydraulically feasible. Potential sizes and sites of a Gloucester facility and the relationship of a facility to the Gloucester interceptor sewer capacity will be further evaluated in the Final LTCP.

Delaware River - Backchannel

- The Delaware River Backchannel subsystem is hydraulically isolated from the impacts of expanding the WPCF #1 by the current capacity limits of the Baldwins Run PS and forcemain.
- Upsizing the forcemain is considered impractical due to the length of the forcemain (3.4 miles), which runs from the Baldwin's Run PS to the Delaware WPCF #1.
- Therefore, a pure conveyance option for the Delaware Backchannel subsystem is not feasible. To achieve 85% capture, satellite treatment or storage for the C-32 outfall or the removal of wet weather flows from High St. in Pennsauken into the Camden collection system will be required.
- By removing the Pennsauken wet weather flow at High St. that is currently routed into the Camden collection system, the need for satellite treatment or storage at C-32 could be eliminated. The Baldwin Run's PS would need to be upgraded to 25 MGD capacity, which, subject to additional analysis appears to be possible without upsizing the forcemain to the WPCF.

Cooper River

- The Cooper River subsystem is hydraulically isolated from the impacts of expanding the WPCF #1 by the capacity limits of the Pine Street PS and forcemain.
- As a result, achieving 85% capture in the Cooper River subsystem will require satellite treatment or storage, and/or significant conveyance capacity upgrades starting at the C-27 and Thorndyke outfalls all the way to the WPCF#1, or a combination of satellite facilities and conveyance upgrades.
- In addition to conveyance capacity upgrades, a "pure conveyance" option for the Cooper River subsystem would require an additional 130 MGD wet weather treatment capacity at or in the vicinity of WPCF No. 1 through a dedicated process train using ballasted flocculation or other high rate treatment process.
- The conceptual level cost estimates indicate that a conveyance only approach to the Cooper River subsystem would be considerably more expensive than satellite treatment or storage.

Newton Creek

- The expansion of the WPCF # 1 to 185 MGD and the control of runoff from 10% of the DCIA using green stormwater infrastructure will allow for the typical year wet weather capture of at least 85% in the Newton Creek subsystem.

E.8 Conceptual Planning Level Cost Estimates

Conceptual planning level cost estimates have been developed for the sub-system 85% control alternatives described above. The lowest and highest estimated capital cost and net present worth values associated with each sub-system are shown in Table E-3 to illustrate the cost range of achieving 85% control in each sub-system.

Table E-3 – Range of Estimated Costs for 85% Capture Alternatives

Subsystem	Capital Costs		40 Year Present Worth	
	Low	High	Low	High
1 Newton Creek	$0.0	$0.0	$0.0	$0.0
2 Cooper River	$22.6	$76.1	$30.1	$91.3
3 Delaware River - Camden	$0.5	$0.5	$0.5	$0.5
4 Delaware River - Back Channel	$16.5	$34.4	$22.0	$37.7
5 Delaware River - Gloucester	$16.3	$37.3	$21.8	$49.4
Subtotal Subsystem		$148.2	$74.4	$178.9
System-Wide Costs				
Controlling Runoff from 10% of DCIA Using GSI	$56.1	$56.1	$76.8	$76.8
WPCF # 1 Expansions				
185 MGD (Under Construction)	$19.9	$19.9	$19.9	$19.9
220 MGD (Under Evaluation)	$20.0	$20.0	$29.1	$29.1
130 MGD Additional WW Treatment*	$0.0	$60.0	$0.0	$87.4
Subtotal System-Wide	$96.0	$156.0	$125.8	$213.3
Grand Total	**$151.9**	**$304.2**	**$200.2**	**$392.2**

***** The "pure conveyance" alternative for the Cooper River sub-system would require additional wet weather treatment capacity beyond 220 MGD at or in the vicinity of the WPCF No. 1.

These cost estimates will to be refined and revised moving forward during the development of the Final LTCP. At this conceptual planning level, they should be viewed primarily a s showing relative costs between alternatives and in helping to determine whether alternatives merit further development in the Final LTCP process.

E.9 Final Long Term Control Plan

The Selection and Implementation of the Final Long Term Control Plan (FLTCP) is due in July 2020. Its contents will build upon and incorporate the findings of this DEAR that:

- The control performance target will be 85% capture of wet weather combined sewer flow during the typical year;
- All control alternatives will incorporate a target controlling runoff from no less than 10% of the directly connected impervious area within the combined sewer system through green stormwater infrastructure;
- CCMUA's WPCF No. 1 wet weather treatment capacity be expanded further from the soon to be completed 185 MGD capacity to 220 MGD; and

- The hydraulic capacity of the Camden collection system will be restored through the ongoing cleaning of the pipes and the CSO outfalls and that regularly scheduled cleaning will occur to maintain the restored hydraulic capacity.

These findings will be expanded, refined and as necessary revised through the following activities to be conducted in preparation of the Final LTCP:

- Affordability and financial capability assessment;
- Technical refinement of control alternatives;
- Opportunities for community development, aesthetic, recreational, and environmental co-benefits;
- Siting, routing and community impacts of the control alternatives;
- Cost and Performance Considerations; and
- Scheduling and Adaptive Management.

Section 1

Introduction

1.1 Regulatory Context and Report Objectives

This document constitutes Camden County Municipal Utilities Authority's (CCMUA) *Development and Evaluation of Alternatives Report* (DEAR) developed by CCMUA on behalf of CCMUA, the City of Camden and Gloucester City (the Cities) for the required "Evaluation of Alternatives" under Part IV Section G.4 of CCMUA's New Jersey Pollutant Discharge Elimination System (NJPEDS) permit action (Permit number NJ0026182). The scope of this includes the Cities of Camden (Permit NJ0108812) and Gloucester (Permit NJ0108847).

The objective of the DEAR is to provide CCMUA and the Cities with a comprehensive and empirical understanding of the available and practicable combined sewer overflow (CSO) control alternatives. The DEAR was prepared pursuant to the permittees' approved *System Characterization Report, Baseline Consideration of Sensitive Areas* and *Baseline Compliance Monitoring Report*. This report documents that CCMUA and the Cities have prepared constitutes Step 2 - **Development and Evaluation of Alternatives for the LTCP** which is required under the NJPDES permits.[1-1] Pursuant to Paragraph G (Long Term Control Plan Requirements) of the permits, this report documents the evaluation of control alternatives that meet the water quality-based requirements of the Clean Water Act, to meet existing and designated uses in the Cooper and Delaware Rivers and Newtown Creek, to prioritize controlling CSOs to sensitive areas and to minimize impacts from significant industrial users.

This DEAR documents the selection between the "presumption" approach which is premised on the presumption that the achievement of certain performance standards, e.g. the capture of at least 85% of wet weather flows during a typical year would result in CWA compliance and the "demonstration" approach, under which permittees demonstrate that their proposed controls do not cause or contribute to a violation of receiving stream water quality standards.

A wide range of control alternatives have been evaluated, including:

- Green stormwater infrastructure;
- Collection system storage capacity;
- Expansion of CCMUA's Water Pollution Control Facilities # 1 including CSO related bypassing of secondary treatment to enable the maximization of flows to the plant during wet weather;
- The reduction of Inflow and Infiltration (I&I) to meet the definition of non-excessive inflow; and
- Sewer separation.

[1-1] Combined Sewer Management sub-section D.3.b.(v)

1.2 Overview of System Addressed in this DEAR

1.2.1 Combined Sewer System

The Combined Sewer System consists of the respective collection systems owned and operated by the Cities of Camden and Gloucester and the portion of the CCMUA's regional conveyance interceptor system that is located within the Cities of Camden and Gloucester. The general characteristics of the combined sewer system are summarized on Table 1-1.

Table 1-1 – Collection System Overview

Jurisdiction	# Sewer-sheds	Collection System Pipe in Miles[1-2]	Appurtenances				Contributing Area (square miles)
			Active Regulators	Active Outfalls	Pump Stations	Solids & Floatables Control Facilities	
Camden	27[1-3]	173	24	22	8	22	6.6
Gloucester	7	39	7	7	7	7	1.6
CCMUA			1	1	2	1	
Totals	34	212	32	30	17	30	8.2

The geographic and schematic layouts of the combined sewer system addressed in this DEAR are shown on Figures 1-1 and 1-2 respectively.

The City of Camden has a total area of approximately ten square miles and a population of about 77,344 (2010 Census). The average daily wastewater flow generated within the City is estimated to be approximately 20 MGD. The wastewater collection system consists primarily of combined sewers. There are approximately 170 miles of combined sewers capturing runoff from nearly 4,000 storm inlets, about 60 percent of which were constructed before 1920.

The Gloucester City collection system serves an area of approximately 1.6 square miles. The bulk is in combined sewered areas of the City, around 1.0 square miles however there are some areas within newer sanitary sewered neighborhoods that are east of the older riverfront portions of the City (0.6 square miles). As of the 2010 Census, Gloucester had a population of about 11,500. The average daily wastewater flow generated in Gloucester is about 2.0 MGD. There are about 40 miles of combined and sanitary sewers within Gloucester City. The seven sewersheds within Gloucester discharge into a City interceptor sewer roughly aligned along King Street. This interceptor flows southerly and discharges into CCMUA's Gloucester Pump Station for conveyance northerly across Newton Creek towards the CCMUA Water Pollution Control Facility No. 1.

[1-2] Source: Table 2-2 from the Sewer System Inventory and Assessment / Facilities Inventory and Assessment Analysis Final Report prepared by CH2MHill, November 1999

[1-3] Includes Camden sewersheds flowing to the C-32 regulator for which CCMUA is the permittee.

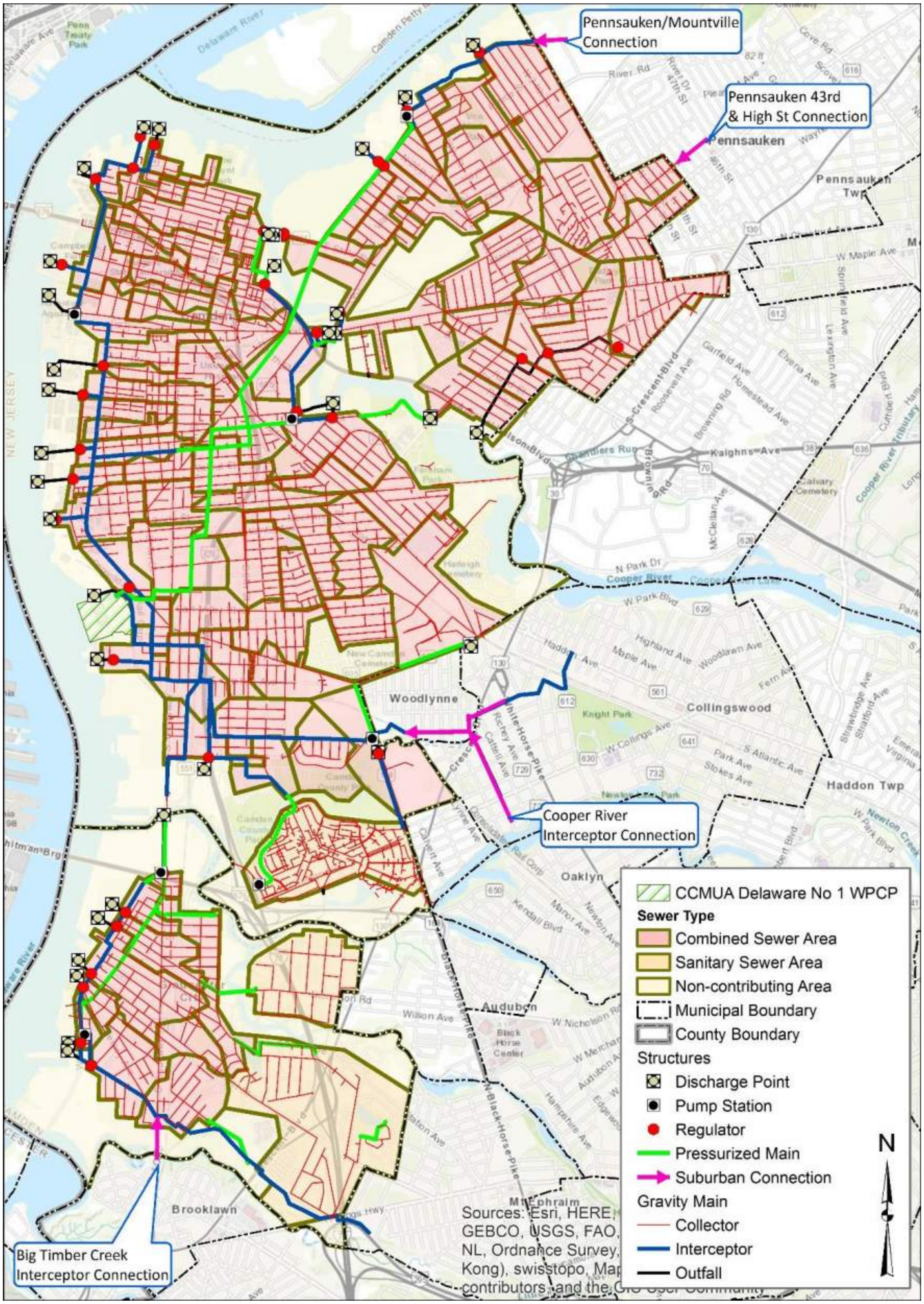

Figure 1-1 - Camden and Gloucester Combined Systems

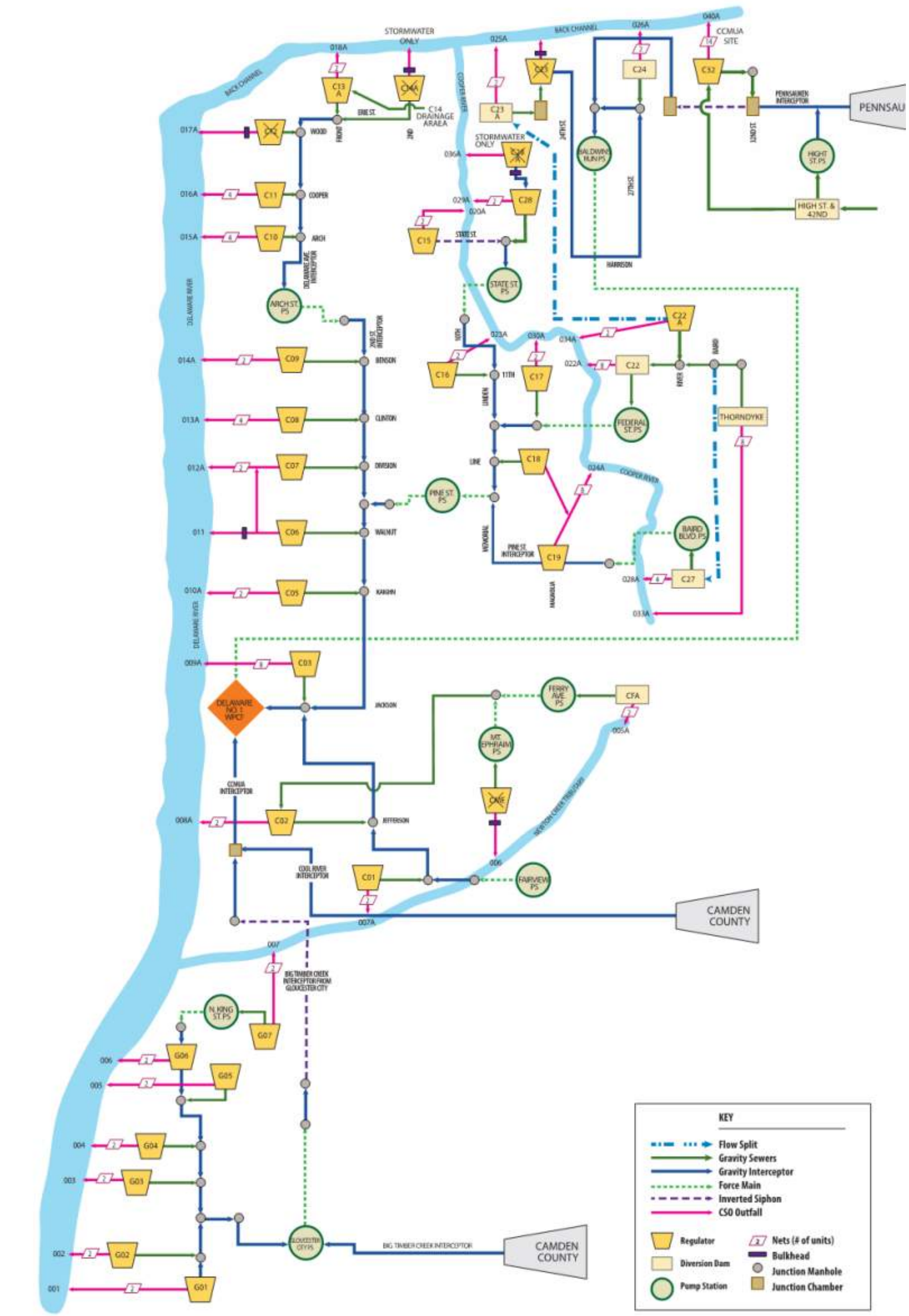

Figure 1-2 – System Schematic

1.2.2 Control Alternatives Baseline System Performance

In the 2018 System Characterization Report, CCMUA and the Cities identified the Control Alternatives Baseline (CAB) performance baseline to be used in the development and evaluation of CSO control alternatives. The CAB was premised on the expansion of CCMUA's Delaware No. 1 WPCF wet weather treatment capacity to 185 MGD and the restoration of the hydraulic capacities of the Camden sewer system, including stormwater inlets and CSO outfalls to current design capacities through comprehensive cleaning. CCMUA has subsequently determined that the expansion of wet weather treatment capacity of the WPCF No. 1 to 220 MGD by bypassing primary effluent that exceeds the secondary treatment capacity is feasible near- term (e.g. within the next three to five years). The bypassed primary effluent would be disinfected and blended with the disinfected secondary treatment effluent prior to discharge into the Delaware River.

The CAB (plant at 185 MGD) system performance characteristics to be used as a starting point for control alternatives are summarized on Table 1-2 below along with the system-wide performance with the plant at 220 MGD. The projected reduction in CSO volume, increased capture rates and reduction in surface flooding resulting from these early implementation steps may be noted.

Table 1- 2 – System Wide Performance Characteristics Used for Control Alternatives Development

System Wide Performance Metrics		Baseline Condition	Control Alternatives Baseline	Plant Further Expansion
		Plant @ 150 MGD + Camden Hydraulic Capacity not Restored	*Plant @ 185 MGD + Camden Hydraulic Capacity Restored**	*Plant @ 220 MGD + Camden Hydraulic Capacity Restored[1]*
1	% Capture	68%	76%	79%
2	Overflow Volume (million gallons)	829	627	541
3	Range of Overflow Frequencies (events)	69-10	70-5	70-9[2]
4	Modeled Surface Flooding (million gallons)	90	44	43

1.Includes C3 regulator opened to utilized increased available wet weather treatment capacity.

2. The increase in minimum overflow frequency is due to the increased capture flow at C3 preventing the flow at C5 from being captured. This led to an increase of overflow volume at C5 (1 MG) and 4 more overflows during the typical year. Meanwhile annual overflow at C3 reduced by 100 MG.

1.2.3 Receiving Waterbodies

Receiving Stream Baseline Water Quality

The 29 active CSO outfalls operated by the Cities of Camden and Gloucester and one by CCMUA discharge into the Cooper River and into the main stem and the back channel of the Delaware River and into Newton Creek. CCMUA conducted a baseline water quality assessment to ascertain the role of CSOs in violations of the applicable receiving stream water quality standards. This assessment focused on the water quality standards for pathogens shown on Table 1-3 and examined the differences in receiving stream pathogen levels upstream and downstream of the CSO outfalls.

Table 1-3 Water Quality Standards for Different Pathogen Species[1-4]

Pathogen/receiving water	E. Coli	Fecal Coliform	Enterococcus
Delaware River Zone 3	N/A	geometric mean < 770 cfu/100ml	geometric mean < 88 cfu/100ml
Cooper River	Single value < 235cfu/100ml geometric mean < 126 cfu/100ml	N/A	N/A
Newton Creek	Single value < 235cfu/100ml geometric mean < 126 cfu/100ml	N/A	N/A

1.2.4 Potential Sensitive Areas

In accordance with the National CSO Control Policy,[1-5] CCMUA and the Cities of Camden and Gloucester are required to give highest priority to controlling overflows to receiving waters considered to be sensitive areas. As a part of this system characterization, CCMUA and the Cities performed an analysis to identify any potential sensitive areas and the CSO outfalls that discharge to them. The results of this analysis are summarized on Table 1-4.

Table 1-4 – Potential Sensitive Areas Summary

Category		Applicable to Receiving Streams Impacted by CSOs			Notes
		Delaware River	Cooper River	Newton Creek	
1	Outstanding National Resource Waters	No	No	No	Source: NJ Antidegradation Standards, NJDEP
2	National Marine Sanctuaries	No	No	No	Source: National Oceanographic and Atmospheric Administration (NOAA)
3	Waters with Threatened or Endangered Species or Designated Critical Habitat	Potential Habitat for Fresh-water Mussel, Short-nose Sturgeon	Potential Habitat for Fresh-water Mussel	No	Delaware River – Tidewater Mucket Cooper River: – Yellow Lampmussel – Eastern Pondmussel – Tidewater Mucket
4	Waters used for Primary Contact Recreation	No	No	No	No authorized bathing beaches or other primary contact recreation areas were identified.

1-4 Source: Delaware River Basin Commission

1-5 59 FR 75-18692

Category		Applicable to Receiving Streams Impacted by CSOs			Notes
		Delaware River	Cooper River	Newton Creek	
5	Public Drinking Water Intakes	No	No	No	• Camden & Gloucester water supplies are from wells.[1-6] • Philadelphia Delaware intake is upstream of applicable CSOs.
6	Shellfish Beds	No	No	No	No shellfish beds as defined in the NJ Costal Management Program have been identified.

As shown on Table 1-4 there is the potential that parts of the Delaware and Cooper Rivers receiving CSO discharges could be suitable habitats for the Short-Nose Sturgeon and for several threatened or endangered species of mussels. If these areas are suitable habitats and these species of aquatic wildlife are present there, these areas would be considered as sensitive areas as defined in the U.S.EPA CSO Control Policy and will require prioritization in the implementation of the approved long term CSO control plan.

1.3 Previous Studies

This report builds upon the information provided in the previous studies required under the Cities' and the CCMUA's respective NJPDES permits as well as other studies and documents prepared for the Cities and for CCMUA. These are listed in Table 1-5.

Table 1-5 – Previous Studies

Title		NJDEP Approval Date
1	System Characterization Report	1/24/19
3	Baseline Compliance Monitoring Report	2/7/19
3	Baseline Consideration of Sensitive Areas	9/20/18

1.4 Organization of Report

Table 1-6 provides the locations of the DEAR elements referenced under the NJPDES permit within this Development and Evaluation of Alternatives Report.

[1-6] Sources: Camden 2016 Annual Water Quality Report – PWS NJ0408001, Gloucester City 2016 Consumer Confidence Report.

Table 1-6 Location of NJPDES Referenced Elements of the DEAR

Permit Section	Permit Requirement	DEAR Section	Comments
G.4.b	• Ensure CSO controls will meet water quality requirements of the CWA; • Protect existing and designated uses; • Prioritize sensitive areas	This DEAR report is intended to present a reasonable range of CSO control alternatives intended to meet the water quality requirements of the CWA upon full implementation of the eventual LTCP. Documentation of the prioritization of sensitive areas and the protection of designated uses must be deferred until the control strategies are finalized and documented in the Final LTCP.	
G.4.c	The permittee shall select either the Demonstration or Presumption Approach for each group of hydraulically connected CSOs and identify each CSO group and its individual discharge location.	3.0	
G.4.d	The Evaluation of Alternatives Report shall include a list of control alternative(s) evaluated for each CSO.	5.3	
G.4.e	The permittee shall evaluate a range of CSO control alternatives: i. Green infrastructure ii. Increased storage capacity iii. STP expansion and/or storage iv. I/I reduction v. Sewer separation vi. Treatment of the CSO discharge vii. CSO related bypass	4.4	
G.4.f	The Presumption Approach – documentation of conformance with one of the three criteria.	5.4	
G.4.g	The Demonstration Approach – Documentation of conformance with all of the four criteria.	Not Applicable	
G.5.a	Cost-Performance Considerations – Conduct “Knee of the Curve” analysis for a range of overflow event control levels.	Deferred until the refinement of control strategies under the Final LTCP.	

1.5 Prior and Ongoing CSO Control Efforts

The development of CSO control alternatives documented in this report has been conducted in the context of prior and ongoing CSO control efforts that have been undertaken by CCMUA and the Cities. These efforts include:

- ***Implementation of the Nine Minimum Controls*** – The Cities and CCMUA continue to implement the Nine Minimum Controls pursuant to the documentation submitted to NJDEP under previous NJPDES permit cycles.
- ***Green Stormwater Infrastructure*** – CCMUA, the Cities and a broad coalition of community development, environmental, and academic groups have gained a

national reputation for leadership in sustainable redevelopment and green stormwater management. Since 2011, fifty-seven projects have been completed removing 63 million gallons of stormwater (on an average annual basis) from the combined sewer system.

- ***Screening and Netting Facilities*** - Pursuant to their respective NJPDES permits, CCMUA and the Cities of Camden and Gloucester installed netting or bar screens at the point of discharge that prevent the passage of any item through an opening smaller than ½ inch. In Camden City, all 22 active outfalls have solids/floatables control facilities installed.[2-7] Gloucester City has solids/floatables controls currently in operation at all 7 of its CSOs[2-8]; and CCMUA has a facility at C-32, its one combined sewer overflow outfall.

- ***Wet Weather Treatment Capacity Expansion*** – As detailed in Section 6 of this DEAR, expanding CCMUA's Water Pollution Control Facility to maximize wet weather flows to the treatment plant will be a crucial component in any long term control plan. CCMUA is currently implementing hydraulic upgrades to the influent chamber and influent pump station and other hydraulic improvements that will increase wet weather capacity from 150 to 185 million gallons per day (MGD).

  CCMUA also anticipates further increasing the capacity to 220 MGD through the primary treatment and disinfection of flows exceeding the secondary capacity of WPCF # 1 and the blending of these wet weather flows with the wastewater receiving secondary treatment, in accordance with a NJPDES permit authorizing the partial bypass of flows around the secondary treatment process.

- ***Stormwater Management Improvement Projects*** - CCMUA and the City of Camden have undertaken several capital improvement projects intended to improve stormwater management and to reduce localized street flooding in the downtown riverfront redevelopment area. CCMUA is installing a wet weather lift station near the foot of Elm Street and the Delaware River. The lift station will have a firm peak capacity of 100 MGD and is intended to address street flooding along and in the vicinity of Delaware Avenue south of the Ben Franklin Bridge for the ten-year storm.

  In addition, CCMUA is undertaking a related project involving the rehabilitation and reconfiguration of the 84" Cooper Street sewer. Approximately 190 linear feet of new 84" diameter Class V reinforced concrete pipe with cured in place liner will be installed from a new relocated netting facility on the Delaware River along Cooper

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[2-7] Sources: Design data and calculations relating to the Camden City trash capture systems prepared by DRM Consulting Engineers, July 2010, including plan views and cut-away diagrams of the TrashTrap® in-line netting systems of Fresh Creek Technologies. Also, July 2010 Technical design memorandum prepared for CCMUA by D&B/Guarino Engineers, LLC reviewing the plans and specifications for the City of Camden CSO Systems B – Netting facilities. Additional information was from Hazen & Sawyer relative to Camden Phase A netting facilities design.

[2-8] Source: Design Calculations for Netting System and Screens at Gloucester City CSO Improvements Prepared for Seprotech Systems Incorporated by August C. Lozano, P.E. Inc. June 2008.

Street towards Riverside Drive. East of Riverside Drive the existing Cooper Street sewer will be lined to Delaware Avenue.[2-9]

- ***Arch Street Pump Station Improvements*** – CCMUA and Camden are upgrading the capacity of the Arch Street Lift Station by replacing the three existing 75 horsepower motors with new 100 horsepower motors and replacing the three existing 22.25" impellers with 24.25" impellers.[2-10]
- ***City of Camden Sewer & Outfall Cleaning Program -*** The City of Camden acknowledges that it is essential that the City's sewer and outfall pipes be cleaned to restore their hydraulic capacities. As the sewer cleaning efforts by the City have determined the presence of more materials than originally projected, the original cleaning cycle which commenced in February of 2016 is currently projected to be completed in February of 2021 or approximately 1.5 years from the date of this Development and Evaluation of Alternatives Report. Thereafter, the inlets, sewers, regulators and outfalls of the Camden CSO system shall be cleaned every three years, except in the instances of force majeure events.

1.6 Anticipated Results of Implementing the Initial Phases of the LTCP

CCMUA and the Cities are working proactively on the implementation of CSO controls and surface flooding projects that will be key components of the Final Long Term Control Plan (FLTCP). Through the expansion of the Delaware WPCF No. 1 to 220 MGD through wet weather blending, the ongoing restoration of the collection system hydraulic capacities and source reduction through green stormwater infrastructure; the following levels of CSO control performance are projected:

- System-wide typical year wet weather capture of 79% to 83% depending upon the rate of source reduction through green stormwater infrastructure;
- Typical year capture rates for Newton Creek and along the main channel of the Delaware River in Camden of 87% and 93% respectively;
- A reduction in annual overflow volumes of approximately 300 million gallons; and
- A reduction in surface flooding in the City of Camden by approximately 50%.

2-9 Source: Plan and profile drawings for the Cooper Street Combined Sewer Rehabilitation Project prepared by Remington & Varick Engineers, November 2017.

2-10 Source: Design drawings for the Arch Street Pump Station Upgrades Project prepared by Remington & Varick Engineers, November 2017.

Section 2

Subsystem and Sewershed CAB Performance

This section documents the basis for the aggregation of the thirty active combined sewer outfalls into five sub-systems and provides the control alternatives baseline performance of the subsystems by their component sewersheds.

2.1 Sewersheds

There are 34 sewersheds within the Camden and Gloucester combined sewer collection systems. These include twenty-seven within the City of Camden and seven in Gloucester City. Each of these sewersheds drain to a regulator structure controlling the amount of wet weather flow that enters into the CCMUA interceptors from the Camden and Gloucester trunk sewers. As of 2018, there are a total of 30 active CSO outfalls located within the two cities. Overflows from these CSO outfalls discharge into three receiving streams: the Delaware and Cooper Rivers and Newton Creek. Key characteristics of the sewersheds are summarized on Table 2-1.

Table 2-1 – Sewershed Location and General Characteristics

Sewershed / Regulator				Owner Municipality	Receiving Stream	Contributing Area (acres)
Count	Name		Status			
1	C1		Active	Camden	Newton Cr.	422
2	C2		Active	Camden	Delaware R.	193
3	C3		Active	Camden	Delaware R.	686
4	C5		Active	Camden	Delaware R.	104
5	C6		Active	Camden	Delaware R.	52
6	C7		Active	Camden	Delaware R.	66
7	C8		Active	Camden	Delaware R.	100
8	C9		Active	Camden	Delaware R.	103
9	C10		Active	Camden	Delaware R.	86
10	C11		Active	Camden	Delaware R.	175
11	C12		Inactive	Camden	Delaware R.	15
12	C13	C13	Inactive	Camden	Delaware R. back Channel	94
		C13A	Active			
13	C14		Inactive	Camden	Delaware R. back Channel	27
14	C15		Active	Camden	Cooper R.	25
15	C16		Active	Camden	Cooper R.	33
16	C17		Active	Camden	Cooper R.	129
17	C18		Active	Camden	Cooper R.	79
18	C19		Active	Camden	Cooper R.	179
19	C22		Active	Camden	Cooper R.	518
20	C22A		Active	Camden	Cooper R.	81

Sewershed / Regulator				Owner Municipality	Receiving Stream	Contributing Area (acres)
Count	Name		Status			
21	C23A	C23	Inactive	Camden	Delaware R. back Ch.	67
		C23A	Active	Camden	Delaware R. back Ch.	
22	C24		Active	Camden	Delaware R. back Ch.	66
23	C27		Active	Camden	Cooper R.	120
24	C28	C28	Active	Camden	Cooper R.	33
		C28A	Inactive	Camden	Cooper R.	
25	C32		Active	CCMUA	Delaware R. back Ch.	491
26	CFA		Active	Camden	Newton Cr.	170
27	CME		Inactive	Camden	Newton Cr.	122
28	G1		Active	Gloucester	Delaware R.	160
29	G2		Active	Gloucester	Delaware R.	16
30	G3		Active	Gloucester	Delaware R.	20
31	G4		Active	Gloucester	Delaware R.	144
32	G5		Active	Gloucester	Delaware R.	182
33	G6		Active	Gloucester	Delaware R.	468
34	G7		Active	Gloucester	Newton Cr.	10
Totals						5,235

2.2 CSO Outfall Groupings

For purposes of developing control strategies, the 30 active outfalls within the combined sewer system have been divided into hydraulically connected and geographically proximate groupings as shown on Table 2-2. Also shown on Table 2-2 are the list of active outfalls and the total numbers of active outfalls by group. The approximate configurations of the CSO groupings are shown spatially and schematically on Figures 2-1 and 2-2 respectively.

Table 2-2 – CSO Outfall Groupings

Outfall Grouping		Active Outfalls	No. of Active Outfalls
1	Delaware River – Camden	C2 – C13A	9
2	Delaware River – Gloucester	G1-G6	6
3	Delaware River – Back Channel	C23A, C24, C32	3
4	Cooper River	C15-C19, C22, C22A, C27, C28, Thorndyke	9
5	Newton Creek	C1, CFA, G7	3
	Total		30

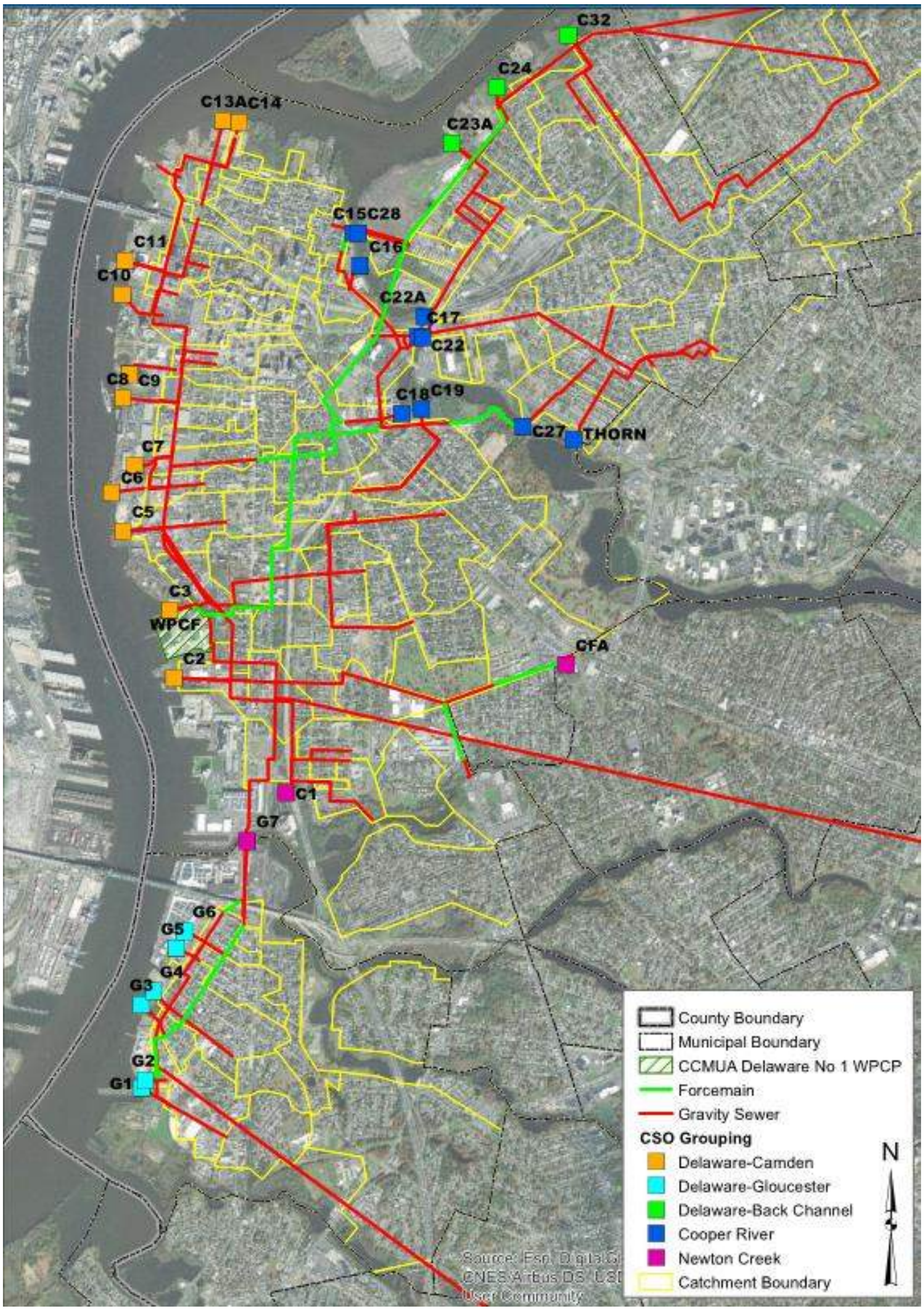

Figure 2-1 – Logical Groupings of Outfalls - Map View

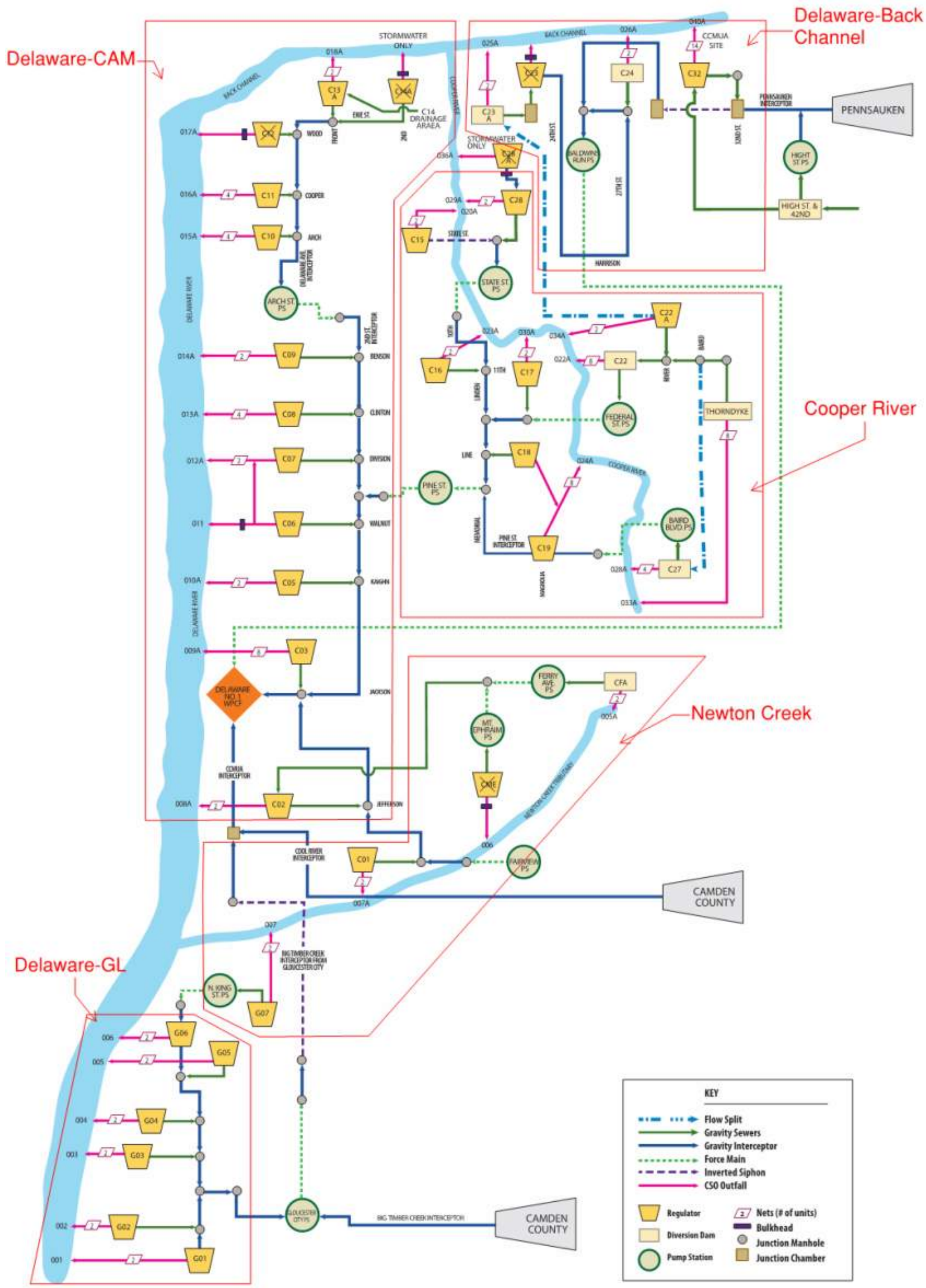

Figure 2-2 – Logical Groupings of Outfalls - Schematic View

The bases for these groupings are provided in Table 2-3.

Table 2-3 – Basis for Defined Sub-Systems

Subsystem		Basis for Grouping
1	Delaware River - Camden	Consists of the sewersheds whose flows are captured by Camden's Delaware River / Second Street interceptor and the nine associated regulator structures discharging into the Delaware River.
2	Delaware River - Gloucester	Consists of the six outfalls in Gloucester City that discharge to the Delaware River and their associated regulators from which the captured flows get conveyed to the Gloucester City pump station.
3	Delaware River - Back Channel	Consists of the three active outfalls discharging to the Delaware River back channel. The captured flows from these sewersheds are conveyed to the Baldwin's Run Pump Station.
4	Cooper River	Consists of the nine active Camden CSO outfalls that discharge to the Cooper River. The sewersheds and regulators associated with these nine outfalls all convey dry weather flow and captured wet weather flow to the Pine Creek Pump Station.
5	Newton Creek	Includes the two active Camden outfalls and the one (G-7) Gloucester City outfall that discharge to Newton Creek.

2.3 The CAB Performances of the Outfall Groups

The Control Alternatives Baseline system (CAB) performances of the outfall groups are summarized in Table 2-4 in terms of annual overflow volume, the range of overflow frequencies, and percent capture.

Table 2-4 – Control Alternatives Baseline Performance by Outfall Group

Outfall Group		Annual CSO Volume, MG	Overflow Frequency	Percent Capture
1	Delaware River – Camden	216.8	70-5	85%
2	Delaware River – Gloucester	75.1	64-43	69%
3	Delaware River – Back Channel	142.0	53-28	69%
4	Cooper River	170.5	63-25	70%
5	Newton Creek	23.4	50-23	85%
	System wide	627.8	70-5	76%

As described in the SCR, percent capture is a more complex metric than volume and frequency. It is the fraction (as a percentage) of wet weather flow in the combined sewer system that is captured for treatment. Conversely, the percentage of what is not being captured (overflow and flooding) during wet weather can be calculated. The percent captured is therefore calculated as shown by the formula below. The "WW" in the formula stands for wet weather.

$$Percentage\ Capture = 1 - \frac{(Total\ CSO\ Volume + Total\ Flooding\ Volume)}{(\text{Total System WW Inflow} - \text{Total WW Flow from Separate Sanitary Communities})}$$

The Delaware River-Camden can be used as an example here to elaborate how percent capture is calculated on an outfall group basis

First the wet weather period is flagged by comparing simulated total inflows entering this part of the sewer system in wet weather to that in dry weather for every time step. When the former was more than 10% of the latter, this time step would be flagged as a wet weather time step.

Wet weather time steps were flagged for the entire typical year. For the Delaware River-Camden outfall group, the total inflows consist of sanitary flow, groundwater base infiltration, and wet weather runoff coming from thirteen sewersheds (see Table 2-5 for details). There are also inflows from other outfall groups, i.e. flow from Pine Street Pump Station (captured flow from the Cooper River outfall group), captured flow from C1 and Fairview area, and captured flow from CFA and CME. These flows are combined sewer flows and should be considered as part of the system inflow.

Annual overflow and flooding volumes for the Delaware River-Camden outfall group can also be calculated from model output as listed in Table 2-6. Due to the limitation of the model on flooding volume prediction, simulated flooding may be somewhat understated, thus the percentage capture estimates calculated using the above formula may be slightly overstated. However, since overflow volume is ten times bigger, the effect is limited.

Flow contribution from separate sanitary communities during wet weather needs to be subtracted from the total inflow as percentage capture only applies to the combined system flow. In the case of Delaware River – Camden outfall group, no inflow is from any of the sanitary communities in Camden County.

The different flow components for the other four outfall groups are listed in Table 2-5. The values of the components used to calculate percent capture are detailed in Table 2-6 for all outfall groups under the CAB.

Table 2-5 Flow Components for Each Outfall Group

Outfall Group		Wet Weather (WW) Inflow		Captured Flow	Flow from Separate Sewered Area
		From Sewershed	Other Inflows		
1	Delaware River – Camden	C2, C3, C5, C6, C7, C8, C9, C10, C11, C12, C13, C14	Pine St PS, C1 & Fairview, CFA & CMT	Camden Mag Meter	None
2	Delaware River – Gloucester	G1, G2, G3, G4, G5, G6	King PS	City of Gloucester flow into Gloucester City PS	None
3	Delaware River – Back Channel	C23, C24, C32	Part of C22 inflow splits into C23; Pennsauken High St connection	Baldwin's Run PS	Pennsauken Int
4	Cooper River	C15, C16, C17, C18, C19, C22, C17	C22 flow splits into C23	Pine St PS	None
5	Newton Creek	C1, CFA, G7		C1, CFA, G7	None

Table 2-6 Values of Different Components for Percent Capture Calculation of Each Outfall Group under the CAB

Outfall Group		WW Inflow, MG	CSO, MG	Flooding, MG	Separate Community WW Flow, MG	Capture
1	Delaware River – Camden	1589.9	216.8	14.3		**85%**
2	Delaware River – Gloucester	264.9	75.1	6.1		**69%**
3	Delaware River – Back Channel	690.1	142.0	0.7	235.6	**69%**
4	Cooper River	586.6	170.5	7.0		**70%**
5	Newton Creek	196.1	23.4	5.1		**85%**

Under the CAB, outfall groups Delaware River- Camden and Newton Creek already reached 85% capture. The overall system wide percent capture is 76% as shown in Table 2-4.

Section 3
CSO Control Objectives

Pursuant to their respective NJPDES permits, CCMUA and the Cities are required to select either Demonstration or Presumption Approach for each group of hydraulically connected CSOs and identify each CSO group and its individual discharge locations.

The “presumption” approach is premised on the presumption that the achievement of certain system-wide performance standards would provide adequate levels of control to meet the water quality based requirements of the Clean Water Act. Under the presumption approach, CCMUA and the Cities must any of the following three criteria below:

- No more than an average of four overflow events per year from a hydraulically connected system as the result of a precipitation event that does not receive the minimum treatment specified below. The Department may allow up to two additional overflow events per year. (NJPDES paragraph G.4.f.i); or
- The elimination or the capture for treatment of no less than 85% by volume of the combined sewage collected in the CSS during precipitation events on a hydraulically connected system-wide annual average basis (NJPDES paragraph G.4.f.ii; or
- The elimination or removal of no less than the mass of the pollutants, identified as causing water quality impairment through the sewer system characterization, monitoring, and modeling effort, for the volumes that would be eliminated or captured for treatment under Section G.4.f.ii. (NJPDES paragraph G.4.f.ii)

Under the “demonstration” approach, permittees must demonstrate that their proposed controls do not cause or contribute to a violation of receiving stream water quality standards and to protect designated uses unless water quality standards or designated uses cannot be met as a result of natural background conditions or pollution sources other than CSOs. The Demonstration approach was intended to provide a means of demonstrating compliance even if the performance based metrics under the Presumption approach could not be accomplished.[3-1]

CCMUA and the Cities have determined to develop a final Long Term Control Plan that will meet the requirements of the NJPDES permits through the elimination or the capture for treatment of no less than 85% by volume of the combined sewage collected in the CSS during precipitation events on a hydraulically connected system-wide annual average basis.

[3-1] USEPA CSO Policy: “A permittee may demonstrate that a selected control program, though not meeting the criteria specified in II.C.4.a. (*Presumption Approach*) above is adequate to meet the water quality-based requirements of the CWA.” 59 FR 18693

Section 4

Identification and Screening of Control Alternatives

4.1 Control Strategies

CSO controls fit into four broadly defined and overlapping strategies:

1. ***Source Reduction*** – removing wet weather flow before it enters the combined sewer system through green stormwater infrastructure, sewer separation, etc.
2. ***Storage*** – local or centralized storage of peak wet weather flows to be bled back into the interceptor sewers for conveyance to the treatment plant after the storm;
3. ***Increase Conveyance Capacities*** – expanding the ability to convey wet weather flows through the interceptor sewers and related appurtenances such as pump stations to the wastewater treatment plant or satellite wet weather treatment facilities; and
4. ***Wet Weather Treatment*** – through the expansion of wet weather treatment capacity at the wastewater treatment plant or through one or more satellite wet weather treatment facilities.

Phased combinations of these strategies are typically used to develop a comprehensive long term CSO control plan. Various physical and institutional technologies are used to implement these control strategies.

4.2 Control Technology Overview

The respective NJPDES permits require that the following seven control technologies be evaluated in the development and evaluation of CSO controls:

1. Green infrastructure.
2. Increased storage capacity in the collection system.
3. 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).
4. I/I reduction to meet the definition of non-excessive infiltration and non-excessive inflow as defined in N.J.A.C. 7:14A-1.2 in the entire collection system that conveys flows to the treatment works to free up storage capacity or conveyance in the sewer system and/or treatment capacity at the STP, and feasibility of implementing in the entire system or portions thereof.
5. Sewer separation.
6. Treatment of the CSO discharge.
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.

An overview of CSO control technologies considered for CCMUA and the Cities is provided as Table 4-1. Additional technical and cost information about these control technologies is provided in the Passaic Valley Sewerage Commissioners ' CSO Long Term Control Plan - Updated Technical Guidance Manual which is used by permission of the PVSC and is provided in Appendix A and is incorporated by reference of this DEAR.

Table 4-1 – Summary of Control Technologies

	Control Technology	Description
1	Green Infrastructure	Green stormwater infrastructure (GSI) encompasses a range of land-based stormwater management techniques and physical reconstruction of aquatic habitats where appropriate. GSI includes but is not limited to such technologies such as rain gardens, bioswales, permeable pavement and green roofs
2	Increased Storage Capacity	
	Inline – within existing collection system	Available space within the existing combined sewers is utilized to retain wet weather flow until downstream conveyance and treatment capacity is available. Flows can be controlled through the regulator structures or through the installation of gates, weirs or controllable dams (e.g. inflatable dams).
	Offline – Storage Tanks	Purpose-built storage tanks which can be either underground or above grade are used to retain wet weather flow. The wet weather flow is then bled back into the system once downstream conveyance and treatment capacities are available.
	Offline – Storage Tunnels	Tunnels with large diameters relative to the collection or interceptor sewers are used to store large quantities of wet weather flow. These are typically centralized to parallel an existing interceptor sewer and can also provide conveyance capacities.
3	Sewage Treatment Plant Expansion and/or Storage at the treatment plant	The treatment capacities at an existing wastewater treatment plant is expanded to allow additional wet weather flows. This can involve the expansion of the existing process train or the addition of dedicated wet weather flow treatment facilities. If space is available, flow equalization storage basins can be used to store peak flows for later treatment as plant capacity is available.
4	Inflow / Infiltration Reduction	Hydraulically connected sanitary collection sewer systems are rehabilitated to reduce the quantities and rates of ground water infiltration and wet weather related inflow.

	Control Technology	Description
5	Sewer Separation	Sewer separation is defined as the reconstruction of an existing combined sewer system into non-interconnected sanitary and storm sewer systems. Sanitary sewer systems are tributary to the wastewater treatment facility, while storm sewer systems discharge directly or indirectly (through detention ponds or other stormwater control facilities) to local receiving waters.
6	Treatment of CSO Discharges	Wet weather flows which would otherwise be discharged through overflow points receive the equivalent of primary treatment and disinfection prior to discharge at remote facilities. Treatment is typically through physical (e.g. vortex treatment units) or physical / chemical (e.g. ballasted flocculation) equipment.
7	CSO Related Bypass of Secondary Treatment	Encouraged under the CSO Policy, this is a type of plant expansion utilizing primary treatment capacities which may exceed the treatment capacities of a wastewater treatment plant's secondary treatment process equipment. During wet weather all flow receive preliminary and primary treatment. Flows exceeding the plant's secondary treatment capacity are shunted around the secondary treatment process equipment. These bypassed flows are disinfected and discharged.

4.3 Economic Screening Criteria

An Alternatives Costing Tool (ACT) was used to estimate capital and operation & maintenance (O&M) costs for the development and evaluation of control alternatives. The ACT utilizes capital cost curve equations based on concurrent CSO control planning within New Jersey as well as data bases of prior CSO and related projects nationally. Construction cost data have been updated using the first quarter 2019 Construction Cost Index for Philadelphia from the Engineering News Record. Representative examples of construction costs for the various treatment technologies are shown on Table 4-2.

It should be noted that the construction cost estimates used are Class 5 (Conceptual Screening) as defined by the Association for the Advancement of Cost Engineering and therefore have an expected accuracy range of -50% through +100%.[4-1]

[4-1] Cost Estimate Classification System – as Applied in Engineering, Procurement, and Construction for the Process Industries. [AACE International Recommended Practice No. 18R-97]

Table 4-2 – Control Technology Construction Costs (Representative Examples)

Control Technology	Units	Unit Cost	Estimate Range	
			-50%	+100%
Green Stormwater Infrastructure	Controlled Acre (5:1 contributing to GSI ratio)			
Rain Gardens		$200,500	$100,250	$401,000
Bioswales		$225,000	$112,500	$450,000
Porous Asphalt		$402,500	$201,250	$805,000
Porous Concrete		$457,500	$228,750	$915,000
Permeable Pavers		$250,000	$125,000	$500,000
Average		$307,100	$153,550	$614,200
Medium		$250,000	$125,000	$500,000
Storage*				
Inline		Site specific		
Offline - Storage Tanks	Million Gallons (Representive Sizes)			
10 million		$36,000,000	$18,000,000	$72,000,000
20 million		$66,000,000	$33,000,000	$132,000,000
30 million		$96,000,000	$48,000,000	$192,000,000
Offline - Tunnels	Tunnel Length per Linear Feet (representative sizes)			
8' Diameter		$2,300	$1,150	$4,600
12' Diameter		$3,400	$1,700	$6,800
16' Diameter		$4,500	$2,250	$9,000
Wastewater T.P. Expansion	MGD	Not Applicable - CCMUA WPCF # 1 Currently being expanded		
Inflow & Infiltration Reduction				
Collection Sewer Lining	Linear Foot	$35.00	$17.50	$70.00
Average Building Lateral Replacement	Lump Sum	$7,000	$3,500	$14,000
Sewer Separation	Acre			
Dense Residential (5-8 Units / Acre)		$150,000	$75,000	$300,000
Commercial / Industrial		$450,000	$225,000	$900,000
Downtown Urban		$950,000	$475,000	$1,900,000
High Rate Treatment (Ballasted Floc.)*	MGD		$0	$0
CSO Related Bypassing of Secondary Treatment	MGD	Site specific - CCMUA is currently evaluating		

*Excludes consolidation pipes, drop shafts, and pumping (as applicable).

Non-construction costs must be added to the raw construction costs to derive total capital cost estimates. Non construction costs used in the cost estimations in this DEAR are shown on Table 4-3. Also shown are the assumptions used for estimating operation and maintenance (O&M) costs and in the calculation of present worth.

Table 4-3 – Cost Estimation Assumptions

Variable	Value
Present Worth Factors	
Discount Rate	2.75%
Planning Period (years)	20
Present Worth Factor	15.2
Non-Construction Costs as % Construction Costs	
Overhead and Profit	15%
Bonds and Insurance	3%
Mobilization/Demobilization	5%
Engineering	25%
Permitting	3%
Total Non-Construction	51%
Contingency Factors - % of Capital Costs	15% to 65% depending on technology and site considerations
Operations & Maintenance	
Operations (Full Time Equivalents)	
Green Stormwater Infrastructure	Included in Maintenance
High Rate Treatment	2.0
Pump Stations	Varies with PS Size
Storage Facility	0.5
Tunnels	1.0
Conveyance / Sewer Separation	Included in Maintenance
Maintenance (% construction costs except where noted)	
Green Stormwater Infrastructure	$/Sq. Ft - varies by type
High Rate Treatment	3.0%
Pump Stations	2.0%

Variable	Value
Storage Facility	3.0%
Tunnels	2.0%
Conveyance / Sewer Separation	2.0%

4.4 Results of Initial Screening

The seven control technologies have been evaluated based on the criteria described in sub-sections 4.3 and 4.4. The results of the initial screening are summarized on Table 4-5; with additional details following below. In Table 4-4 the control technologies are categorized as being applicable system-wide, meaning that they could have system wide benefit and are not limited in application to specific locations (e.g. GSI and plant expansion), or as being potentially applicable in certain local situations (e.g. sewer separation) or as not applicable due to environmental, community or cost impacts (e.g. deep tunnels).

Table 4-4 – Summary of Initial Screening

Alternative	Applicability			Comments
	System Wide	Local	Not Applicable	
Green Stormwater Infrastructure	X			- Lead element in control strategy - Assuming Control of Runoff from 10% of DCIA using GSI
Inline Storage (within collection system)		X		- Important intrinsic storage in clean pipes, ongoing maintenance will remain critical. - System-wide applicability limited - Potential for surcharging will remain
Offline Storage – Tunnels			X	- Cost prohibitive - Construction can be disruptive
Offline Storage – Tanks	X			- Proven CSO control technology - Site and consolidation considerations affect practical applications and cost effectiveness. - Expanded O&M responsibilities

Alternative	Applicability			Comments
	System Wide	Local	Not Applicable	
WPCF # 1 Wet Weather Capacity Expansion	X			– Expansion of WPFC # 1 to 185 MGD is currently underway. – CCMUA is evaluating the expansion of capacity to 220 MGD.
I/I Reduction in Hydraulically Connected Upstream Sanitary Sewered Systems		X		- Limited impact on CSO reduction due to relative volumes of wet weather flows generated in combined areas compared to high I/I sanitary sewered areas. - Notwithstanding, eliminating excessive I/I in sanitary sewered areas important to preserve local hydraulic capacity and to reduce risks of sanitary sewer overflows upstream of the combined sewer system.
Sewer Separation		X		- System-Wide separation would be cost prohibitive. - Local separation may be appropriate during redevelopment projects. - Potential appropriate for CSO control in physically or hydraulically remote sewersheds.
Treatment of CSO Discharge	X			- Proven CSO control technology - Site and consolidation considerations affect practical applications and cost effectiveness. - Provides disinfection – key WQ issue - Provides equivalent of primary treatment. - Expanded O&M responsibilities.
CSO Related Bypassing	X			- CCMUA anticipates that expanding the WPCF wet weather capacity to 220 MGD using CSO related bypassing will be implemented.

4.4.1 Green Stormwater Infrastructure

Green stormwater infrastructure will be a foundational component of the control strategy due to the many environmental, community, aesthetic, economic and community health benefits intrinsic in GSI. However, green stormwater infrastructure alone will be insufficient to provide the level stormwater capture necessary to reduce CSOs to levels complying with regulatory requirements.

CCMUA and the Cities of Camden and Gloucester are targeting a 10% reduction in impervious area and all control strategies evaluated in this DEAR include this impervious area reduction. The 10% target equates to approximately 145 acres as shown in Table 4-5.

Table 4-5 – Calculation of Target Control of Runoff from DCIA

Combined Sewer Area	Acreage
Total	4,499
Directly Connected Impervious Area	1,446
Less 10% of DCIA	-145
Remaining DCIA	1,302

The 10% impervious area reduction target reflects the upper limit of feasible GSI implementation during a twenty – forty-year implementation timeframe typical of CSO control programs. Over a longer timeframe, redevelopment and the renewal and replacement of the impervious areas represented by current buildings, roads, etc. will occur and the impervious area would be expected to decline as building and zoning codes and practices integrate GSI. CCMUA and the Cities of Camden and Gloucester intend to incorporate phased implementation and adaptive management in the Final Long Term Control Plan. This will provide opportunities to re-evaluate the role of GSI in long term CSO control.

The results of the hydrologic and hydraulic analyses performed for this evaluation indicate that a ten percent decrease in impervious area throughout the combined sewer area would result in a system-wide wet weather capture rate during the typical year of 80%. This compares to 76% for the Control Alternatives Baseline conditions.

4.4.2 Increased Storage

In-Line Storage

The Camden combined sewer system is currently undergoing a comprehensive cleaning which will restore its design conveyance capacities and intrinsic storage capacity. As documented in section 9 of the System Characterization Report, Camden is committed to cleaning the collection system every three years to maintain hydraulic capacities.

The intrinsic storage capacities in the Camden and Gloucester collection systems are limited however and available capacities are required for drainage and the prevention of surface street flooding. Opportunities for active CSO control through such devices as inflatable dams have not been identified in the Camden or Gloucester collection systems.

Off-Line Storage: Tunnels

Tunnels are not feasible for CSO control in Camden and Gloucester due to cost. Base on the screening level analysis performed, tunnel segments of 10' to 26' diameter and with an aggregate length of around 6.6 miles would be necessary to achieve target control levels. A conceptual screening level cost estimate exceeds $300 million, which does not include the costs for necessary consolidation sewers, drop shafts and dewatering pump station(s).

Off-Line Storage: Tanks

Off line storage tanks control CSOs by retaining peak wet weather flows until the conveyance interceptor sewers and treatment plant can accept these flows after a precipitation event. Tanks are a proven CSO control technology; however, they can pose siting issues and impose ongoing operation and maintenance requirements. The potential roles of tank storage will be further evaluated.

4.4.3 Increased Treatment or Storage Capacity at the WWTP

As detailed in Section 2.5 of the System Characterization Report, CCMUA is proactively expanding the capacity of its Delaware WPCF #1 from 150 MGD to 185 MGD through improvements to the influent flow control, influent pumping and primary treatment equipment. CCMUA is also evaluating the further expansion of wet weather treatment capacity to 220 MGD. This capacity expansion along with the restoration of the hydraulic capacity of the Camden collection sewer system is the basis of the Control Alternatives Baseline used in this report.

4.4.4 Inflow and Infiltration Reduction

Inflow and infiltration reduction will not play a major role in long term CSO control due to the high volumes of wet weather flow generated in the combined sewered areas relative to the volume of I/I contributed from the hydraulically connected sanitary sewered areas. There are approximately 101 square miles of sanitary sewered areas contributing flow to CCMUA's WPCF #1. If a 50% reduction in I/I from the sanitary sewered area is assumed, the total annual CSO discharge volume would be reduced by approximately 12% from 628 million gallons / year to 550 MGY.

There is an estimated length of collection sewer within the sanitary sewered area contributing to WPCF No. 1 of 1,245 miles. The screening level cost for sewer lining used in this report is $35 per linear foot. Assuming that 30% to 50% of the collection systems would require lining, the capital costs would range from around $69 million to 115 million. Nationally, upwards of 50% of I/I is estimated to originate through building laterals and the screening level cost estimate for a lateral replacement for a typical suburban home is $7,000. Excluding the City of Camden and Gloucester City there are around 184,000 dwelling units in Camden County. Simply for example, if 15% of the laterals on Camden County dwellings were to be replaced, the capital costs would approach $193 million.

The above is not to suggest that the reduction in I/I is not important for purposes of preserving base conveyance and treatment capacities and to control sanitary sewer overflows upstream of the combined sewered areas. Moreover, CCMUA and the Cities will continue to evaluate I/I reduction opportunities in conjunction with road openings, sewer renewal and replacement and redevelopment projects.

4.4.5 Sewer Separation

Sewer separation as a system-wide CSO control strategy would be cost prohibitive. Using a screening level cost of $212,500 per acre for sewer separation, the complete separation of Camden and Gloucester would cost around $960 million as shown on Table 4-6.

Table 4-6 – Screening Level Sewer Separation Cost Estimate

City	Acres	Estimated Costs for Total Sewer Separation ($ millions) @ $212,500 per acre		
		-50%	Middle	+100%
Camden Combined Area	4073	$433	$865	$1,731
Gloucester Combined Area	426	$45	$91	$181
Total	4499	$478	$956	$1,912

In addition to cost, the separation of sewers in an urban area poses potential water quality and regulatory concerns. Separated storm sewers would be subject to stormwater discharge permit requirements such as the MS-4 requirements under the USEPA Stormwater Rules 40-CFR 122. Using the National Stormwater Quality Data Base,[4-2] the pollutant loading from urban runoff is significant, particularly for total suspended solids (TSS) and oil & grease as shown on Table 4-7.

Table 4-7 – Statistical Summary of NSQD Stormwater Runoff Quality Data

Statistic	Concentrations (mg/l)			
	BOD	TSS	Total Phosphorous	Oil & Grease
# of EMCs	5,152	7,637	7,943	2,256
Average	14.1	142	0.39	15.4
Median	8.50	63.0	0.25	4.00
Minimum	0.11	0.10	0.005	0.06
Maximum	433	10,700	21.2	2,980
Standard Deviation	20.4	312	0.68	80.7

Therefore, the water quality benefits of reducing combined sewer overflows during wet weather events large enough to trigger overflows must be weighed against the benefits of capturing and treating urban runoff during the more frequent smaller storms that do not trigger overflows.

4.4.6 Treatment of CSO Discharge

Under this technology, high flow rate treatment (HRT) facilities are installed on the overflow of regulator structures and upstream of the outfall structures. HRT facilities provide the equivalent of primary treatment and disinfection pursuant to the CSO Control Policy. There

4-2 *The National Stormwater Quality Database (NSQD), Version 4.02*, R. Pitt, A. Maestre, and J. Clary. February 17, 2018.

are a variety of HRT processes available which are described in Appendix B of this report. For purposes of bounding this preliminary alternatives analysis, ballasted flocculation is being assumed. High rate treatment is a proven CSO control technology; however, it can pose siting issues and impose ongoing operation and maintenance requirements. The potential roles of high rate treatment will be further evaluated.

4.4.7 CSO Related Bypassing

CCMUA is currently evaluating the expansion of wet weather treatment capacity at the Delaware WPCF #1 through the CSO related bypassing of the secondary process train during wet weather in accordance with a NJPDES permit. Wet weather flows exceeding the 185 MGD capacity of the secondary treatment processes would receive preliminary and primary treatment prior to disinfection. Earlier analysis has indicated that a peak flow rate of 220 MGD could be delivered to the treatment plant through the three interceptors that discharge to the plant. As documented in Section 5, preliminary CSO control alternatives have been developed based on a treatment plant capacity of 220 MGD.

Section 5

Alternatives Development & Evaluation

5.1 Introduction

In this section the control technologies that were screened in Section 4 are arrayed into control alternatives intended to meet the performance standards of the Presumption approach.

The control technologies used to develop the control alternatives include:

1. Green stormwater infrastructure (assuming 10% system-wide reduction in directly connected impervious areas);
2. Expansion of the Delaware WPCF #1 plant beyond 185 MGD;
3. Satellite storage and high rate treatment facilities;
4. Upgrades to existing pump stations and related force mains;
5. Modification of regulator structures controlling the flow from the Camden and Gloucester collection systems into the interceptor sewers including connector pipes and related appurtenances;
6. Modification of flow restrictor settings; and
7. The removal and redirection of wet weather flow from the Pennsauken High Street connection from the Camden collection sewer system.

5.2 Capture Through Source Reduction and Treatment Plant Expansion

CCMUA and the Cities evaluated the ability to meet the Presumption Approach based performance standard of 85% typical year capture through the expansion of the Delaware WPCF #1 and source reduction without the use of significant conveyance capacity upgrades or satellite storage or treatment. The treatment plant capacity is currently being upgraded to 185 MGD. The potential to expand the capacity to 220 MGD is currently under evaluation. Projected typical year capture rates with the WPCF #1 capacity at 185 MGD and 220 MGD and the control of runoff from 10% of the DCIA are shown on Table 5-1. Additional information concerning the plant capacity expansion is provided in the alternatives report *Wet Weather Upgrades at Delaware No. 1 WPCF, Study of Alternatives*; a copy of which is included as Appendix B to this DEAR. Please note that the C-3 regulator and connection conduit would need to be upgraded in order to fully utilize 220 MGD WPCF #1 capacity,.

As shown in Table 5-1, the system wide control target of 85% cannot be met through the expansion of the treatment plant capacity and source reduction alone. Thus, sub-system level controls will be necessary. Control alternatives for the five hydraulically connected sub-systems that were identified in Section 2 of this report have been developed and are described below.

Table 5-1 – Typical Year Capture Impacts of Controlling Runoff from DCIA by 10%

System / Sub-System	Control Alternative Baseline	With 10% Control of Runoff in DCIA and WPCF @	
		185 MGD	220 MGD
System-Wide	76%	80%	83%
Sub-System			
Delaware R. – Camden	85%	88%	93%
Delaware R. – Gloucester	69%	74%	74%
Delaware R. - Back Channel	69%	72%	72%
Cooper River	70%	75%	75%
Newton Creek	85%	88%	87%

The CSO Supplemental Committee has reinforced the desire among Camden stakeholders that the Final LTCP include as much green stormwater infrastructure as quickly as possible. This desire is reflected in the inclusion of a 10% DCIA control target throughout the combined sewer system. The 10% target is highly aggressive and ambitious based on the experiences of urban CSO programs nationally.

The Final LTCP will be based on what is determined by CCMUA and the Cities to include the optimal array of green and grey CSO controls that are necessary to meet the 85% capture of wet weather control strategy based on then current (in 2020) neighborhood characteristics such as land use, street layouts and the existing combined sewer collection system. However, the Final LTCP will include a phased implementation schedule with the initial phases focusing on the expansion of wet weather treatment capacity at CCMUA's WPCF No. 1 and the expansion and acceleration of green stormwater management and sustainable development within Camden. This will provide opportunities to evaluate the continued need, size and location of satellite facilities or other grey controls outlined below prior to their implementation.

5.3 85% Capture Alternatives by Sub-System

5.3.1 Delaware River - Camden

The expansion of the WPCF #1 to 185 MGD will eliminate the need for the Arch Street pump station to be throttled back during wet weather. This, along with controlling runoff from 10% of the directly connected impervious area would enable a projected capture rate of more than 85% for the Delaware River – Camden sub-system. Table 5-2 summarizes the resulted performance of CAB as well as the additional 10% GSI. GSI resulted in an eighteen percent reduction in annual overflow volume. The increased overflow frequency from 70 to 77 was due to the tidal impact at outfall C6/C7 where the overflow stored in the outfall pipe can discharge a small amount during low tidal cycles.

Table 5-2 Summary of Performances for Sub-System Delaware River - Camden

Performance Parameters	Control Alternative Baseline	185 MGD WPCF With Control of Runoff from 10% of DCIA
Subsystem Annual CSO, MG	216.8	177.4
CSO frequency	70-5	77-5
Subsystem percent capture	85%	88%

5.3.2 Delaware River – Gloucester

Three 85% capture alternatives have been developed for the Delaware River - Gloucester sub-system:

Delaware River – Gloucester 1 – Satellite Treatment or Storage Only – This alternative includes:

- Control of runoff from 10% of the directly connected impervious area reduction using GSI
- Expansion of WPCF # 1 to 185 MGD
- 32 MGD high rate treatment or 2.4 MG storage facility serving G-4 and G-5

Delaware River - Gloucester 2 – Satellite Treatment / Storage + Increase Conveyance - This alternative would consist of the following:

- Control of runoff from 10% of the directly connected impervious area reduction using GSI
- Expansion of WPCF # 1 to 185 MGD or 220 MGD
- Operate CCMUA's Gloucester City PS at 35 MGD
- 11.7 MGD high rate treatment or 0.9 MG storage serving G-4 and G-5

The increased conveyance capacity at the Gloucester City PS reduced the sizes needed for the satellite facility. However, the potential for this alternative to impact the separate sewer service areas that contribute flows to the Big Timber Interceptor and Cooper River Interceptor would need to be evaluated and addressed as necessary.

Delaware River - Gloucester 3 – Conveyance Only: This alternative would consist of the following:

- Control of runoff from 10% of the directly connected impervious area reduction using GSI

- Potentially additional wet weather treatment capacity beyond 220 MGD at the WPCF # 1. System-wide, an additional 130 MGD wet weather capacity would be required for aggregate conveyance only controls.
- Upgrades to Gloucester City regulators G-3, G-4, and G-5.
- Increased capacity of Gloucester City interceptor between regulators G-3 through G-5 and to CCMUA's Gloucester City PS.
- Increased CCMUA's Gloucester City PS at 45 MGD

Same as Alternative 2, the increased conveyance of City of Gloucester flow through CCMUA's Gloucester City pump station may have a potential negative impact on the separate sewer service area which would need to be further evaluated.

Table 5-3 summarized the performances of the above described alternatives. Any of the three options can achieve 85% capture for this subsystem and some level of CSO frequency reduction. The overflow frequency range shows the highest and lowest numbers of overflow among all three alternatives at any one outfall.

Table 5-3 Summary of Performances for Sub-System Delaware River - Gloucester

Performance Parameters	Control Alternative Baseline	185 MGD WPCF With Control of Runoff from 10% of DCIA	Range of Results from above Alternatives
Subsystem Annual CSO, MG	75.1	63.8	34.1-31.6
CSO frequency	64-43	64-42	62-4
Subsystem percent capture	69%	74%	86% - 85%

While alternatives 1 and 2 include satellite treatment or storage for overflows at G4 and G5; site constraints may preclude placing a facility in the vicinity of these regulators. The CSO Supplemental Committee noted that a satellite facility in the vicinity of G4 and G5 would impinge on the municipal park.

Alternatively, the wet weather flows could be conveyed through a consolidation pipe southward to the vicinity of regulator structures G1 and G2 where brownfield sites may be available for a facility. Given the need for conveyance towards G1 / G2 under this scenario, the potential to pick up wet weather flows from G3 as well as G1 and G2 could make sense and will be evaluated further during the development of the Final LTCP. The technical feasibility and cost implications of transferring wet weather flows southerly towards the vicinity of G2 will be evaluated in the Final LTCP.

5.3.3 Delaware River – Back Channel

Three alternatives have been developed to obtain 85% capture from the Delaware Backchannel sub-system:

Delaware Backchannel 1 – Satellite Treatment or Storage Only:

- Control of runoff from 10% of the directly connected impervious area reduction using GSI
- Expansion of WPCF # 1 to 185 MGD
- Satellite treatment (16.5 MGD) or storage (2.5 MG) at C-32.

Delaware Backchannel 2 – Satellite Treatment / Storage + Conveyance Upgrades – This alternative would consist of the following:

- Control of runoff from 10% of the directly connected impervious area reduction using GSI
- Delaware WPCF # 1 capacity expanded to 185 or 220 MGD
- Remove flow control restriction from C-23A and C-24 to allow for increased capture
- Reduce flow restriction at C-32
- Increase Baldwin Run PS capacity to 25 MGD
- Satellite treatment (12.9 MGD) or storage (0.9 MG) at C-32

Delaware Backchannel 3 – Source Reduction + Conveyance Upgrades – This alternative would eliminate the need for satellite treatment or storage at C-32 through the removal of the Pennsauken wet weather flows that are currently diverted by a regulator into the Camden collection system upstream of the Pennsauken High Street pump station. The alternative consists of the following:

- Control of runoff from 10% of the directly connected impervious area reduction using GSI
- Delaware WPCF # 1 capacity expanded to 185 or 220 MGD
- Reduce the flow restrictions upstream of the Baldwin's Run PS from C-23A and C-24
- Reduce flow restriction at C-32
- Increase Baldwin Run PS capacity to 25 MGD
- Rerouting current Pennsauken wet weather flow from the Camden collection system.

Limiting the Baldwins Run Pump Station to only 25 MGD can help maintain the velocity in the downstream force main to less than 5.5 fps. This force main from Baldwins Run Pump Station to the WPCF #1 is about 3.4 miles and was repaired within the past fifteen years. Maintaining a velocity of 5.5 fps or less can avoid the high cost of upgrading this long force main.

Table 5-4 summarized the performances of the above described alternatives. Any of the three options can achieve 85% capture for this subsystem and some level of CSO frequency reduction.

Table 5-4 Summary of Performances for Sub-System Delaware River – Back Channel

Performance Parameters	Control Alternative Baseline	185 MGD WPCF With Control of Runoff From 10% of DCIA	Range of Results from above Alternatives
Subsystem Annual CSO, MG	142.0	125.3	65.8-57.3
CSO frequency	53-28	53-25	51-12
Subsystem percent capture	69%	72%	87%-85%

5.3.4 Cooper River (C15-C19, C22 &22A, C27, C28, Thorndyke)

Three 85% capture alternatives have been developed for the Cooper River sub-system:

Cooper River 1 – Satellite Treatment or Storage Only:

- Control of runoff from 10% of the directly connected impervious area reduction using GSI
- WPCF # 1 capacity increased to 185 MGD
- High rate treatment (20 MGD) or storage (1.2 MG) facility serving C-22 and C-22A
- High rate treatment (20.5 MGD) or storage (3 MG) facility serving C-27 and Thorndyke outfalls.

Cooper River 2 – Satellite Treatment / Storage + Conveyance Upgrades – This alternative would consist of the following:

- Control of runoff from 10% of the directly connected impervious area reduction using GSI
- Delaware WPCF # 1 capacity expanded to 220 MGD
- Modify regulator structures at C17, C18 and C19 to allow for increased capture
- Upgrade Pine Street pump station (PS) and forcemain to 35 MGD (~ 3,700 feet)
- High rate treatment (62 MGD) or storage (3 MG) serving the C-27 and Thorndyke outfalls

The increased conveyance illuminated the need for a satellite facility at C-22 and C-22A.

Cooper River 3 – Conveyance Only – In lieu of satellite treatment or storage for C-27 and Thorndyke, additional wet weather flows would be conveyed to the Delaware WPCF #1:

- Control of runoff from 10% of the directly connected impervious area reduction using GSI
- Additional wet weather treatment capacity beyond 220 MGD at the WPCF # 1. System-wide, an additional 130 MGD wet weather capacity at or in the vicinity of the WPCF No. 1 would be required for aggregate conveyance only controls.
- Modify regulator structures at C17, C18 and C19 to allow for increased capture
- New 48 MGD pump station, siphons, force main and parallel interceptor to convey wet weather flow from C-27 and Thorndyke under the Cooper River to the Pine St. PS
- Upgrade Pine Street pump station (PS) and forcemain to 75 MGD (~ 3,700 feet).
- Upsize all the pipes along Second Street Interceptor from Pine Street PS connection to WPCF #1.

Table 5-5 summarized the performances of the above described alternatives. Any of the three options can achieve 85% capture for this subsystem and some level of CSO frequency reduction.

Table 5-5 Summary of Performances for Sub-System Cooper River

Performance Parameters	Control Alternative Baseline	185 MGD WPCF With Control of Runoff from 10% of DCIA	Range of Results from above Alternatives
Subsystem Annual CSO, MG	170.5	142.0	82.7-76.7
CSO frequency	63-25	63-24	62-4
Subsystem percent capture	70%	75%	86% - 85%

5.3.5 Newton Creek (C1, CFA, G7)

The expansion of the WPCF # 1 capacity to 185 MGD coupled with the control of runoff from 10% of the directly connected impervious area would enable the 85% capture performance target to be met within the Newton Creek sub-system without satellite storage or treatment or the expansion of wet weather conveyance capacity to the treatment plant.

Table 5-6 summarizes the resulted performance of CAB as well as the additional 10% GSI. GSI resulted in an eighteen percent reduction in annual overflow volume although did not affect frequency significantly.

Table 5-6 Summary of Performances for Sub-System Newton Creek

Performance Parameters	Control Alternative Baseline	185 MGD WPCF With Control of Runoff From 10% of DCIA
Subsystem Annual CSO, MG	23.4	19.2
CSO frequency	51-23	51-22
Subsystem percent capture	85%	88%

5.4 System Wide Capture Performance

Each of the subsystem control strategies outlined above were sized to capture for treatment no less than 85% by volume of the combined sewage flow generated in the subsystem during wet weather on an annual average basis. Fully implementing the 85% capture strategy across the various subsystems is projected to produce system-wide capture rates of around 88% to 89% since some of the subsystems are projected to have capture rates greater than 85%. This represents a projected system wide annual overflow volumes of 270 to 300 MG,

5.5 85% Capture Sub-System Facility Cost Estimates

Conceptual planning level cost estimates have been developed for the sub-system controls beyond the expansion of the treatment plant and the control of runoff from 10% of the DCIA using green stormwater infrastructure. The estimated costs are summarized in Table 5-7 by sub-system.

Table 5-7 – 85% Capture Conceptual Planning Level Cost Estimates by Sub-System Alternatives

Control Alternative (*Note: Columns do not sum to a grand total)*	Control Facilities Beyond Treatment Plant Expansion and GSI		Present Worth
	Total Capital	Annual O&M	20 Years
Newton Creek Sub-System	$0	$0	$0
Cooper River Sub-System			
Cooper River 1 - Satellite Treatment / Storage + Conveyance Upgrades			
Storage @ C-27 / Thorndyke (3.0 MG)	$47,100,000	$594,000	$56,145,000
Treatment @ C-27 / Thorndyke (62.3 MGD)	$52,000,000	$702,000	$62,690,000
Cooper River 2 - Conveyance Only	$76,051,000	$1,004,000	$91,339,000
Cooper River 3 - Satellite Treatment / Storage Only			
Storage (C-22/22A @ 1.2 MG) + (C-27 / Thorndyke @ 3.6 MG)	$38,200,000	$836,000	$50,930,000
Treatment (C-22/22A @ 20 MGD) + (C-27 / Thorndyke @ 20.5 MGD)	$22,599,000	$492,000	$30,091,000
Delaware River - Camden	$500,000	$0	$500,000
Delaware River - Back Channel			
Delaware BC1 - Satellite Treatment / Storage + Conveyance Upgrades @ C-32			
Storage @ C-32 (0.9 MG)	$31,850,000	$384,000	$37,697,000
Treatment @ C-32 (12.9 MG)	$31,650,000	$379,000	$37,421,000
Delaware BC2 - Source Reduction + Conveyance Upgrades	$34,350,000	$108,000	$35,995,000
Delaware BC3 - Satellite Treatment / Storage Only			
Storage @ C-32 (2.4 MG)	$24,000,000	$525,000	$31,994,000
Treatment @ C-32 (32 MGD)	$16,500,000	$360,000	$21,982,000
Delaware River - Gloucester City			
Delaware River GC1 - Satellite Treatment / Storage			
Storage Serving G-4 & G-5 (2.4 MG)	$23,800,000	$525,000	$31,794,000
Treatment Serving G4 & G-5 (32 MGD)	$16,300,000	$360,000	$21,782,000
Delaware River GC2 - Conveyance Only	$37,299,000	$793,000	$49,374,000

The lowest and highest estimated capital cost and net present worth values associated with each sub-system are shown in Table 5-8 to illustrate the cost range of achieving 85% control in each sub-system. A detailed breakout of the estimated costs by sub-system alternative is provided as Appendix C to this report.

Table 5-8 – Range of Estimated Costs for 85% Capture Alternatives ($ millions)

Subsystem		Capital Costs		20 Year Present Worth	
		Low	High	Low	High
1	Delaware River - Camden	$0.5	$0.5	$0.5	$0.5
2	Delaware River - Gloucester	$16.3	$37.3	$21.8	$49.4
3	Delaware River - Back Channel	$16.5	$34.4	$22.0	$37.7
4	Cooper River	$22.6	$76.1	$30.1	$91.3
5	Newton Creek	$0.0	$0.0	$0.0	$0.0
	Subtotal Subsystem	$55.9	$148.2	$74.4	$178.9
System-Wide Costs					
	DCIA Runoff Control Using GSI	$56.1	$56.1	$76.8	$76.8
	WPCF # 1 Expansions				
	185 MGD (Under Construction)	$19.9	$19.9	$19.9	$19.9
	220 MGD (Under Evaluation)	$20.0	$20.0	$29.1	$29.1
	130 MGD Additional WW*	$0.0	$60.0	$0.0	$87.4
	Subtotal System-Wide	$96.0	$156.0	$125.8	$213.3
	Grand Total	**$151.9**	**$304.2**	**$200.2**	**$392.2**

***** The "pure conveyance" alternative for the Cooper River sub-system would require additional wet weather treatment capacity beyond 220 MGD at or in the vicinity of the WPCF No. 1.

These cost estimates are likely to be refined and revised moving forward during the development of the Final LTCP. At this conceptual planning level, they should be viewed primarily as showing relative costs between alternatives and in helping to determine whether alternatives merit further development in the Final LTCP process.

5.6 Preliminary Siting Considerations

Most of the subsystem control alternatives described in sub-section 5.3 above would require the construction of satellite treatment or storage at or in the vicinity of the locations shown on Table 5-9.

Table 5-9– Potential Satellite Facilities Vicinity Information

Subsystem		Alternatives	Vicinity of Regulators	Approximate Area Required	Vicinity Notes
1	Delaware River – Camden	*No satellite facilities required for 85% capture*			
2	Delaware River – Gloucester	1 & 2	G4 – G5	~1.0	Municipal park, private industry parking area
3	Delaware River – Back Channel	1 & 2	C32	~1.6	CCMUA CSO Screening facility and wildlife refuge area
4	Cooper River	1 & 2	C22 – C22A	~1.4	Brownfield (status unknown) private bus yard, Federal Street pump station.
			C27 - Thorndyke	~1.1	Grassed area of Gateway Park
5	Newton Creek	*No satellite facilities required for 85% capture*			

Approximate site vicinity and current land use maps for these potential satellite facilities are shown on Figures 5-1 through 5-4.

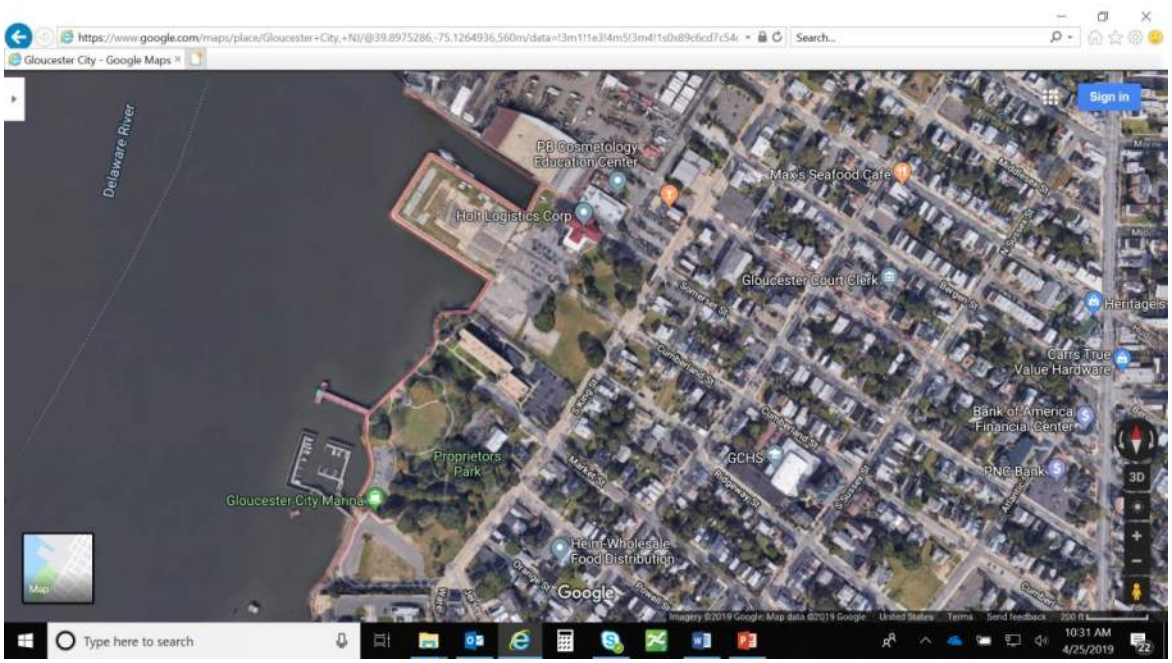

Figure 5-1a– Gloucester City - Site Vicinity for Satellite Facilities for G4 and G5

Figure 5-1b – Land Use for Satellite Facilities for G4 and G5

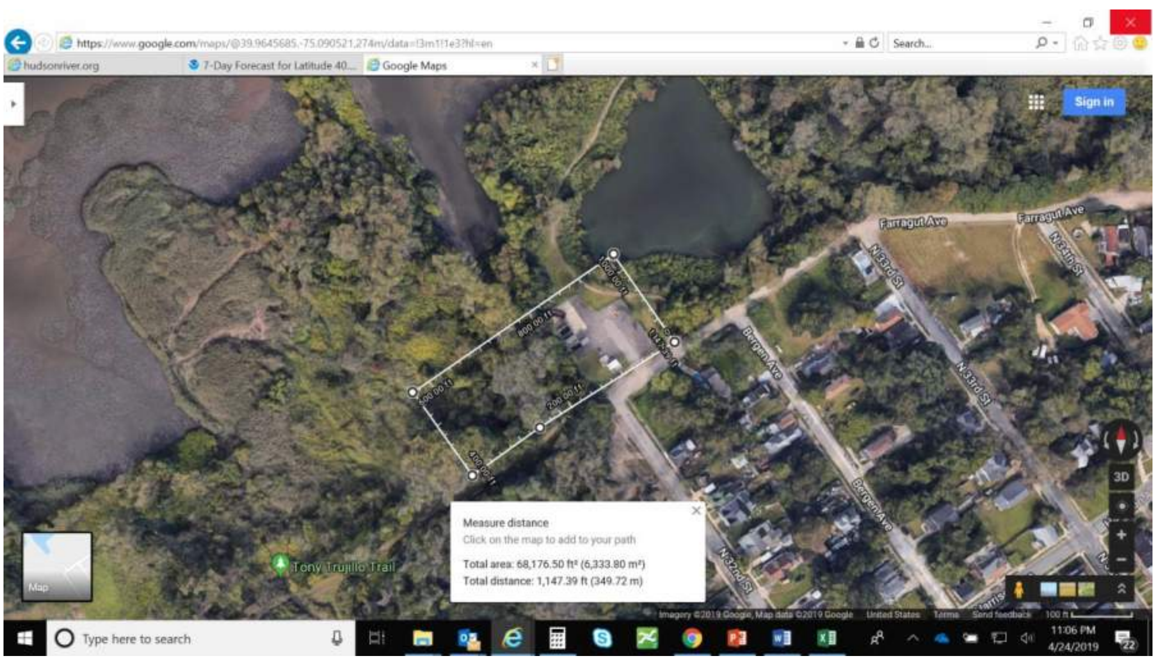

Figure 5-2a – Site Vicinity for Satellite Facility - C32

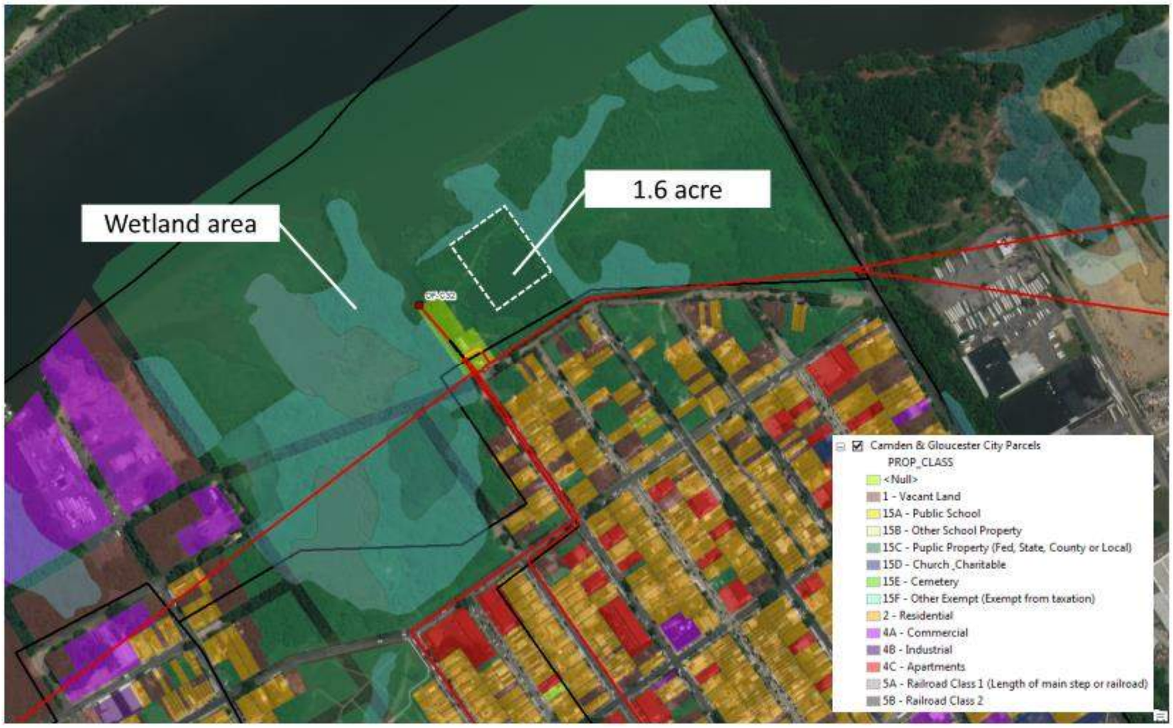

Figure 5-2b – Site Vicinity for Satellite Facility - C32

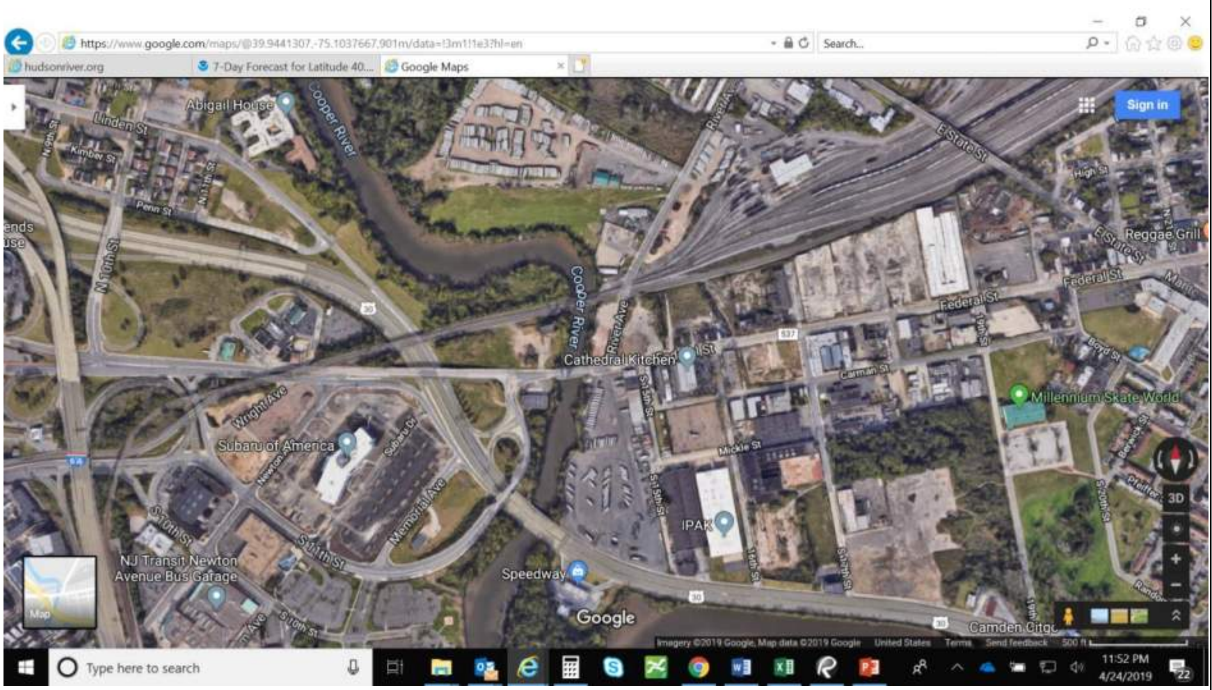

Figure 5-3a – Site Vicinity for Satellite Facility for C22 and C22A

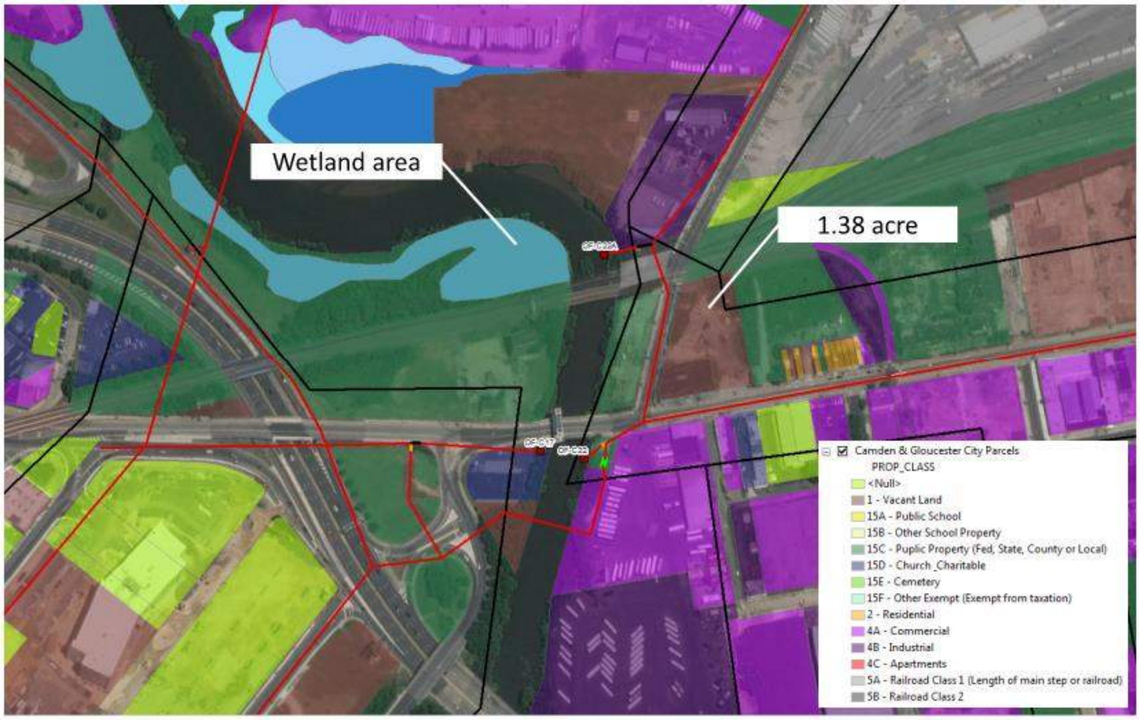

Figure 5-3b – Land Use for Satellite Facility for C22 and C22A

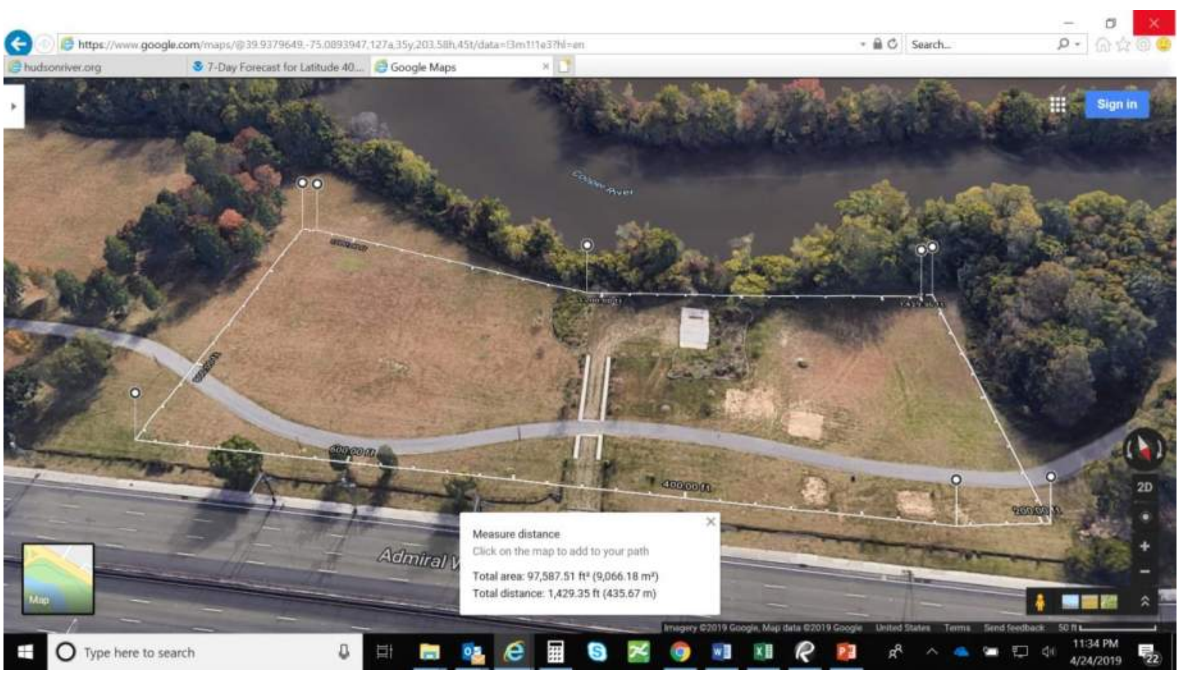

Figure 5-4a – Site Vicinity for Satellite Facility for C27 and Thorndyke

Figure 5-4b – Land Use for Satellite Facility for C28 and Thorndyke

Each vicinity for potential satellite treatment or storage facilities shown above are preliminary. Each is likely problematic, being located within public parkland (e.g. Gloucester and C27 – Thorndyke), on private property or on environmentally sensitive areas such as the wildlife refuge adjacent to the C32 screening facility. They should not be viewed as recommendations but as a preliminary determination that space might be available for a satellite facility. That said, it is possible that the sites for satellite facilities could serve multiple community benefit purposes such as the installation of the bulk of the facilities below grade with the surface being used for such purposes as recreational fields or community gardens. The feasibility of siting satellite facilities will be evaluated further in close cooperation with the neighborhood stakeholders and the CSO Supplemental Committee during the development of the Final LTCP.

5.7 Control Alternative Conclusions

The findings of the alternatives development and evaluation work described above can be summarized as follows. These will serve as the basis for the final development of control alternatives and an overall recommended control program in the Final LTCP.

5.7.1 Source Reduction & Delaware WPCF # 1 Expansion

- The 85% capture control target cannot be achieved solely through (1) Green Stormwater Infrastructure (which assumes the control of runoff from from 10% of the directly connected impervious area within the combined sewer system and (2) the expansion of the treatment plant capacity to 185 MGD or 220 MGD.
- However, the current expansion of WPCF # 1 to 185 MGD and the potential expansion to 220 MGD will significantly improve system-wide wet weather capture and help to reduce surface flooding, e.g. through the ability to maintain wet weather pumped flows from the Arch Street PS.
- To eliminate the need for satellite facilities in the Cooper River and Delaware River – Gloucester subsystems, additional wet weather treatment capacity beyond 220 MGD would be needed at or adjacent to the WPCF #1. The feasibility of adding a wet weather process train using ballasted flocculation or other high rate treatment technology will be evaluated further during the development of the Final LTCP.

5.7.2 Delaware River - Camden

- Expansion of the WPCF # 1 to 185 MGD and the control of runoff from 10% of the directly connected impervious area using GSI will allow for the typical year wet weather capture of at least 85% in the Delaware River subsystem;
- Increasing the level of control for the Delaware River – Camden subsystem would require satellite facilities or to a further expanded WPCF #1. Therefore, it is anticipated that the Final LTCP will focus on the 85% capture target for the Delaware River-Camden subsystem. (This sub-system can get to 93% capture with increased capture at C3 and plant 185 or 220.)

5.7.3 Delaware River – Gloucester

- A conveyance only 85% capture control option would be feasible. This would require the Gloucester City PS to be operated at 45 MGD during wet weather along with regulator modifications and interceptor upsizing.
- Doing this would elevate the hydraulic grade line downstream of the Gloucester City PS; evaluation of the potential impacts to the separate sewer areas contributing to Big Timber Interceptor and Cooper River Interceptor will be required.
- A satellite treatment or storage facility for Gloucester City's G-4 and G-5 would be hydraulically feasible, however siting constraints likely make this infeasible. The potential to convey wet weather flows from G-4 and G-5 to the vicinity of a facility near G-2 while also picking up flows from G-1 through G-3 will be further evaluated in the Final LTCP.

5.7.4 Delaware River - Backchannel

- The Delaware River Backchannel subsystem is hydraulically isolated from the impacts of expanding the WPCF #1 by the capacity limits of the Baldwins Run PS and forcemain. To keep the flow velocity acceptable within the forcemain (~5.4 feet per second), pumping from the Baldwins Run PS cannot exceed 25 MGD.

- Upsizing the forcemain is considered impractical due to the length of the forcemain (3.4 miles), which runs from the Baldwin's Run PS to the Delaware WPCF #1. The use of capacity within the Camden collection sewers adjacent to the forcemain was considered and found to be infeasible.
- Therefore, a pure conveyance option for the Delaware Backchannel subsystem is not feasible. To achieve 85% capture, satellite treatment or storage for C-32 or the removal of wet weather flows from High St. in Pennsauken into the Camden collection system will be required.
- By removing the Pennsauken wet weather flow at High St. that is currently routed into the Camden collection system, the need for satellite treatment or storage at C-32 could be eliminated. The Baldwin Run's PS would need to be upgraded to 25 MGD capacity.

5.7.5 Cooper River

- The Cooper River subsystem is hydraulically isolated from the impacts of expanding the WPCF #1 by the capacity limits of the Pine Street PS and forcemain.
- As a result, achieving 85% capture in the Cooper River subsystem will require satellite treatment or storage, significant conveyance capacity upgrades starting at the C-27 and Thorndyke outfalls all the way to the WPCF#1, or a combination of satellite facilities and conveyance upgrades.
- The conceptual level cost estimates indicate that the conveyance only alternative would be considerably more expensive than alternatives using satellite treatment or storage.
- Both approaches will be further evaluated for the Final LTCP.

5.7.6 Newton Creek

Expansion of the WPCF # 1 to 185 MGD and attainment of GSI with 10% Greened Acres systemwide will allow for the typical year wet weather capture of at least 85% in the Newton Creek subsystem.

Section 6

Development of the Final Long Term Control Plan

This section summarizes the content of the Final Long Term Control Plan (FLTCP) which is due in July of 2020. Its contents will build upon and incorporate the findings of this DEAR that:

- The control performance target will be 85% capture of wet weather during the typical year;
- All control alternatives will incorporate a target of no less than the control of runoff from 10% of the directly connected impervious areas using green stormwater infrastructure throughout the combined sewered area;
- All control alternatives are premised on the restoration of the hydraulic capacity of the Camden collection system through system cleaning on a three-year cycle and the maintenance of regulators and equipment;
- CCMUA's WPCF No. 1 wet weather treatment capacity be expanded further from the soon to be completed 185 MGD capacity to 220 MGD; and
- The hydraulic capacity of the Camden collection system will be restored through the ongoing cleaning of the pipes and the CSO outfalls and that regularly scheduled cleaning will occur to maintain the restored hydraulic capacity.

6.1 Affordability and Financial Capability Assessments

The evaluation, selection and implementation scheduling of the CSO control strategy for the FLTCP will be directed by the results of independent affordability and financial capability assessments for the three permittees. Key aspects of the affordability and financial capability assessments include:

- Time dynamic financial and scheduling models will be developed for CCMUA, Camden and Gloucester. These models will integrate the current general cash flows of the wastewater utilities, non-CSO related capital improvements, financing options and economic variables. The intent of the models is to allow for CCMUA and the Cities to evaluate the affordability and financial capability impacts of the scope and scheduling of CSO controls within the context of ongoing and evolving operations and community needs.
- The models will be used to project annual costs per household and household incomes through the planning period. These projected costs per household will include wastewater services, as applicable stormwater services and will also include the projected costs per typical household for potable water services to enable an understanding of total water service costs on households in various economic groups.
- A comprehensive financial capability assessment using the USEPA metrics listed in the NJPDES permit.

- Economic and demographic conditions and trends. The current USEPA affordability / financial capability assessment guidelines are limited to the assessment of impacts at the median household income. This will be expanded to include income groupings (e.g. income quintiles, poverty rates, housing costs, etc.

6.2 Refinement of Control Alternatives

Informed by the affordability and financial capability assessments, the CSO control alternatives that have been evaluated in this DEAR will be refined and finalized in the FLTCP. This will include:

- Routing and siting considerations;
- Community and environmental impacts;
- Opportunities for leveraging the CSO controls for community redevelopment and other needs;
- Operational considerations; and
- Sustainability, climate change and carbon footprint consideration.

The evolving alternatives will receive conceptual design sufficient to provide useful planning level cost estimates, e.g. level 4 estimates.

6.3 Cost & Performance Considerations

The control alternatives will be evaluated in terms of cost and performance (e.g. "knee of the curve" assessment) to identify opportunities to cost-effectively and affordably optimize control levels beyond the targeted 85% capture.

6.4 Opportunities for Community Benefits

The control alternatives will be evaluated through a triple bottom line perspective in terms of:

1. ***Regulatory Compliance Potential*** – the ability of the control technology to result in compliance with current and foreseeable regulatory requirements;
2. ***Water Quality Benefit*** – The potential for each control technology to produce tangible improvements to the receiving streams through the reduction in the volume and frequency of combined sewer overflows;
3. ***Public Health Benefit*** – The potential for each control technology to contribute to the public health of the community. Benefits can be directly related to CSO control, e.g. reducing the risk of pathogen exposure through the reduction in street and basement flooding and reduction in CSO discharges or indirect e.g. improved air quality and the reduction of the urban heat island effects of paved areas;
4. ***Reduction in Street Flooding*** – while no CSO control program will fully address street flooding after rain that is experienced by Camden, the ability of CSO technologies to contribute to the reduction of street flooding is an important criterion.

5. ***Opportunities to Support Community Redevelopment*** – The ability of the control technologies individually and as components of an integrated long term control plan to support community redevelopment and enhanced quality of life;

6. ***Maximize Use of Existing Resources and the Minimization of Community Disruption***– the ability to optimize the use current combined sewer system and the Delaware WPCF #1 as well as the relative levels of human, environmental and economic disruption posed by the construction of the CSO controls and of their ongoing operation & maintenance;

7. ***Sustainability*** – in terms of resistance and adaptability to climate change, and the efficient use of natural and human resources; and

8. ***Aesthetic and Recreational Opportunities -*** the opportunity for CSO control projects to provide neighborhood aesthetic improvements (e.g. street trees and gardens) and recreational opportunities (e.g. neighborhood playgrounds or small parks associated with underground CSO control facilities).

6.5 Further Evaluation of DCIA Runoff Control Opportunities

The control strategy defined in this DEAR includes a target of a 10% control of the runoff within directly connected impervious areas through green stormwater infrastructure, which is approximately 145 acres of controlled area. During the development of the Final LTCP the availability of this amount of DCIA suitable for green stormwater infrastructure will be verified. This assessment will be based on the perviousness information presented in the System Characterization Report along with land use, land ownership, neighborhood interest and impacts, etc. Based upon the results of this assessment, a determination as to the feasibility of achieving the 10% runoff control from DCIA will be made in the final LTCP and the relative role of "grey" control structures revised accordingly.

6.6 Scheduling and Adaptive Management

A detailed implementation schedule will be included in the FLTCP. It is anticipated that the implementation will be phased in coordination with NJPDES permit cycles, with the initial phase focusing on the ongoing collection system restoration efforts, the expansion of the WPCF #1 capacity and the reduction in wet weather flows through green stormwater infrastructure. Adaptive management considerations will include:

- Re-evaluations of implementation and efficacy of controls during an at the conclusion of phases to assess the need and opportunities for corrections and improvements;
- Evolving and emerging community needs;
- Advancing technologies;
- Re-evaluation of evolving economic and institutional issues beyond the controls of CCMUA and the Cities; and
- Re-evaluation as warranted by climatic changes and other emergent changes in baseline conditions.

Appendix A

PVSC CSO LTCP 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

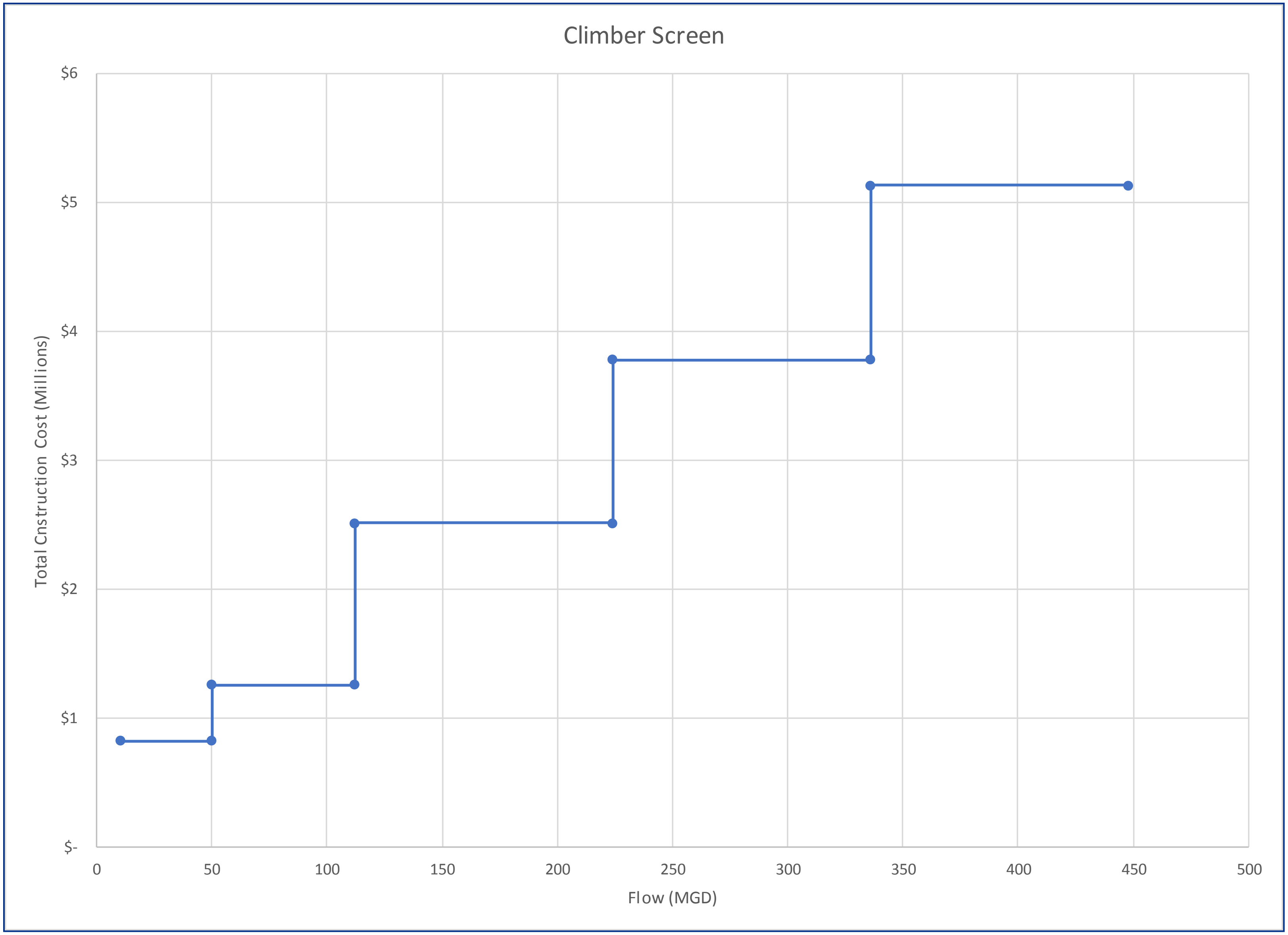


Table 2-2 - Annual Operation Costs of Climber Screens

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

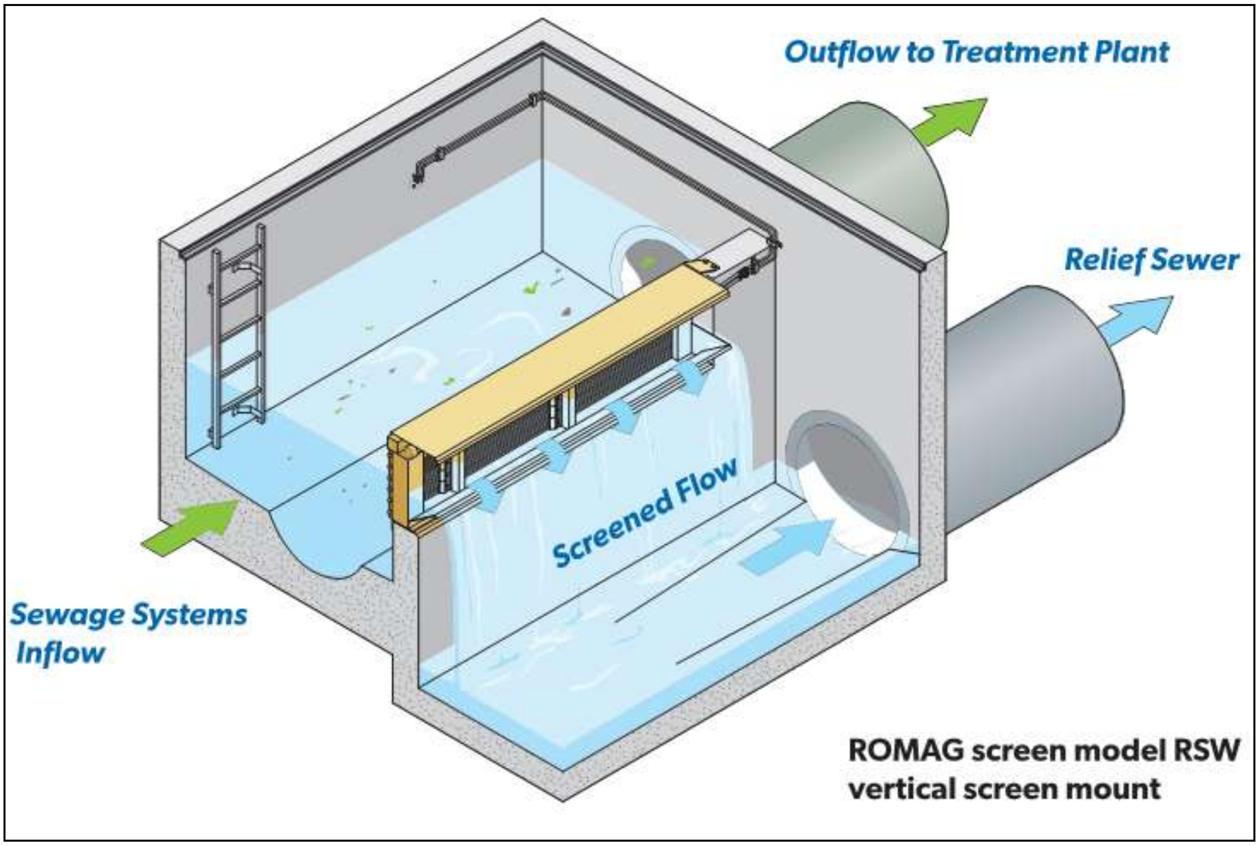
2.2.2 Fine Screens

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

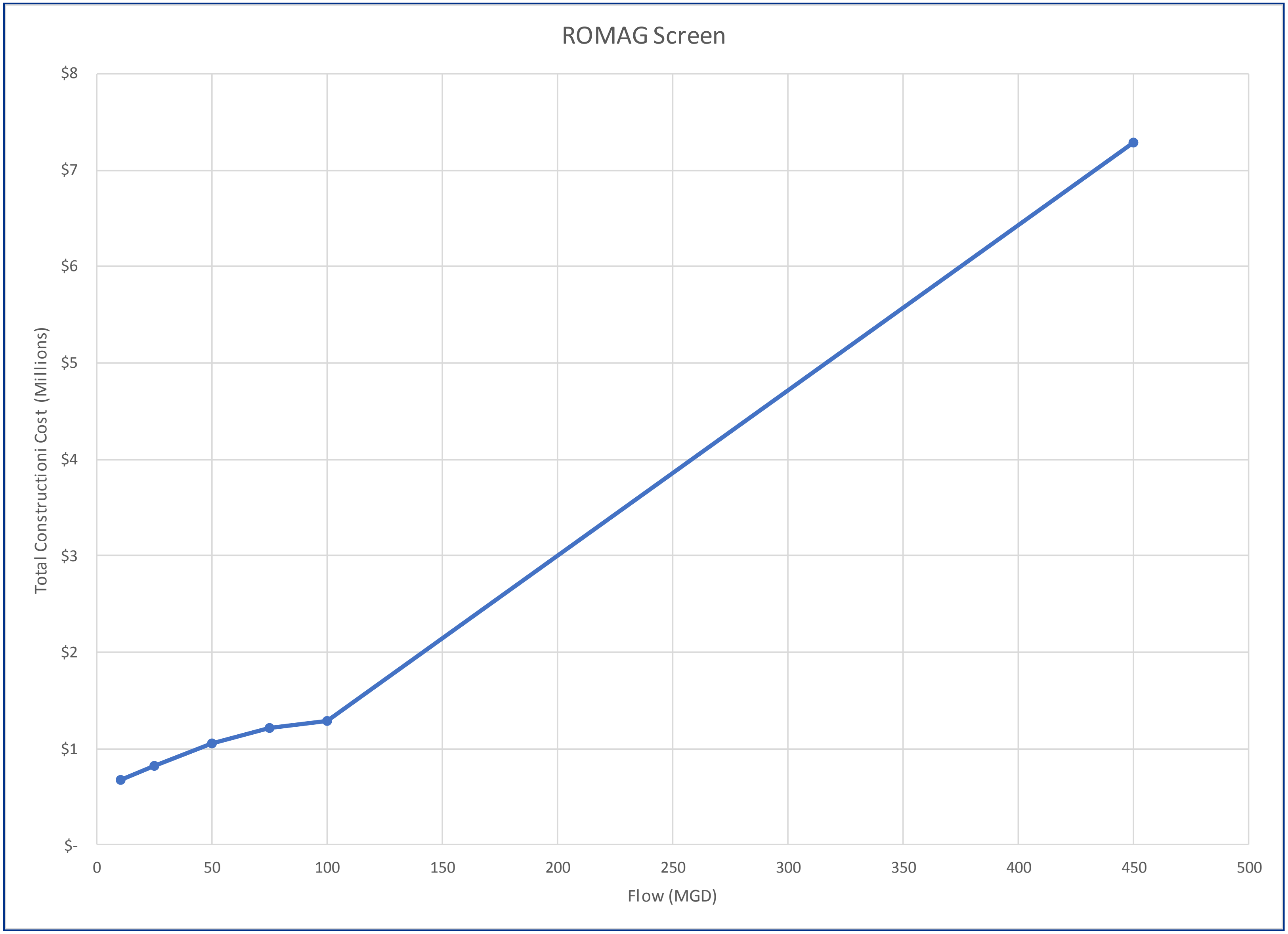


Table 2-5 - Annual Operation Costs of ROMAG Screens

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.

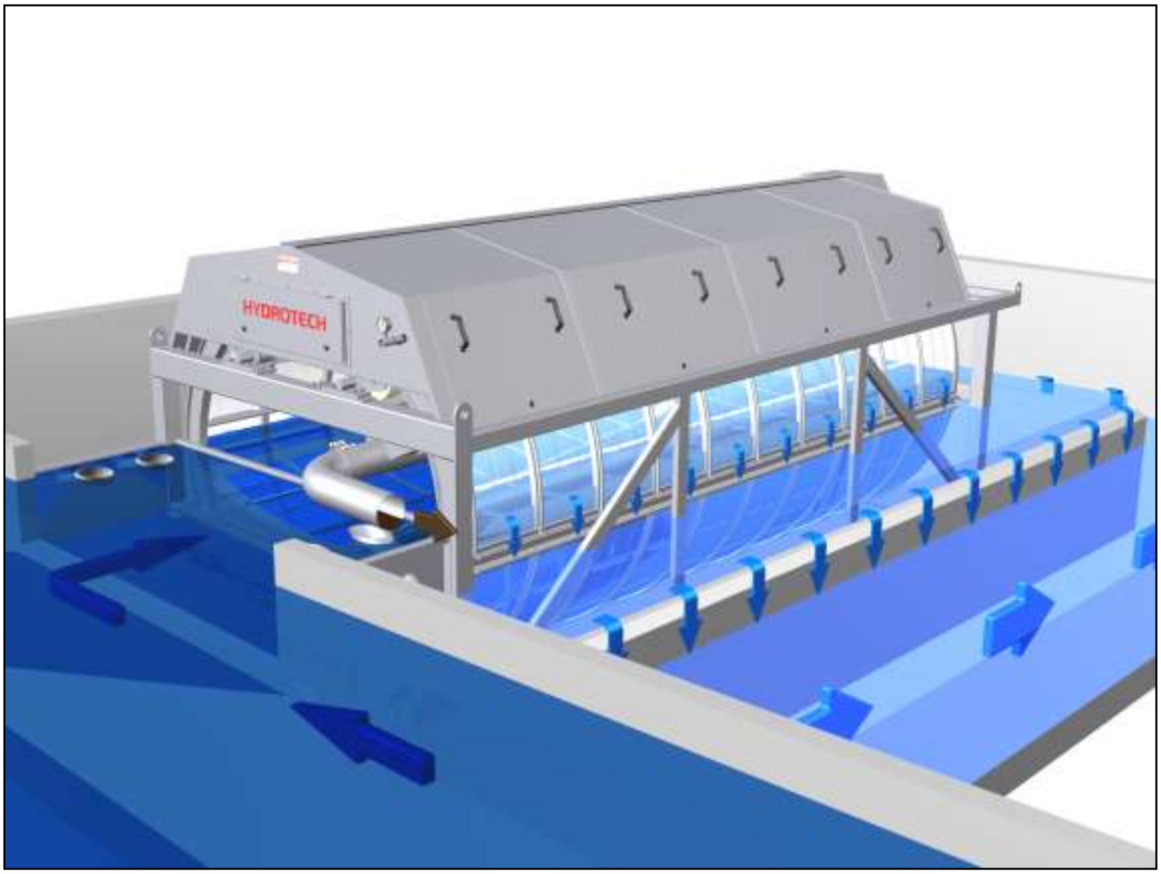
These screens are not considered applicable for CSO treatment and not further evaluated.

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.

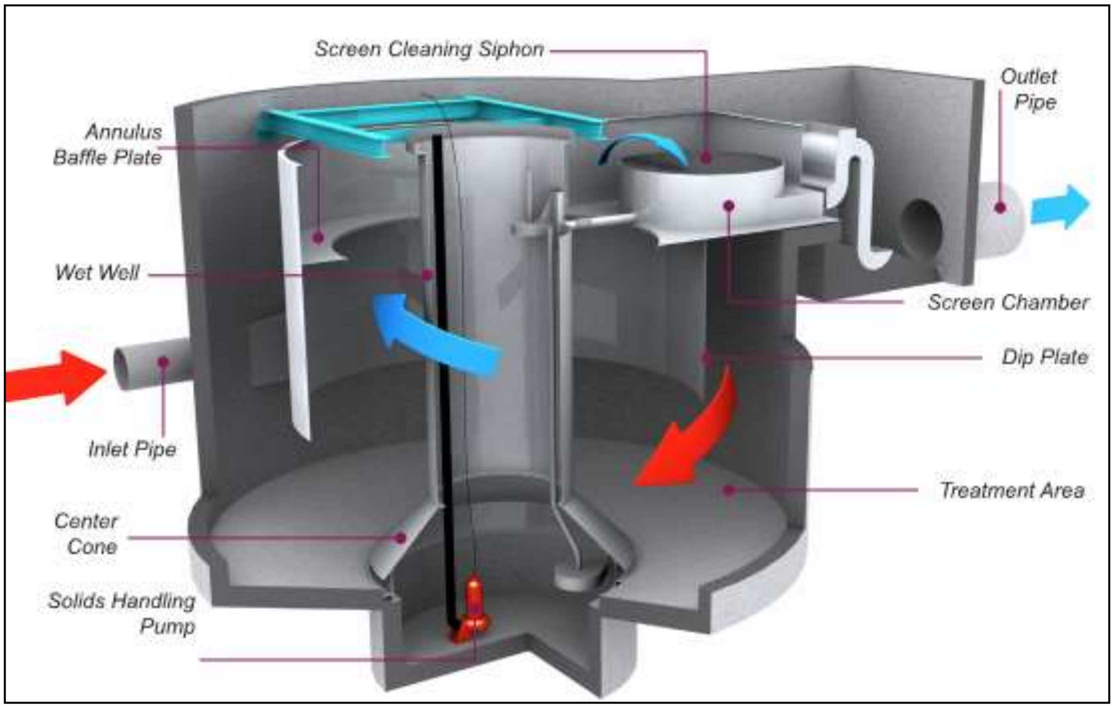
2.3.1.1 Storm King® Vortex Separator

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

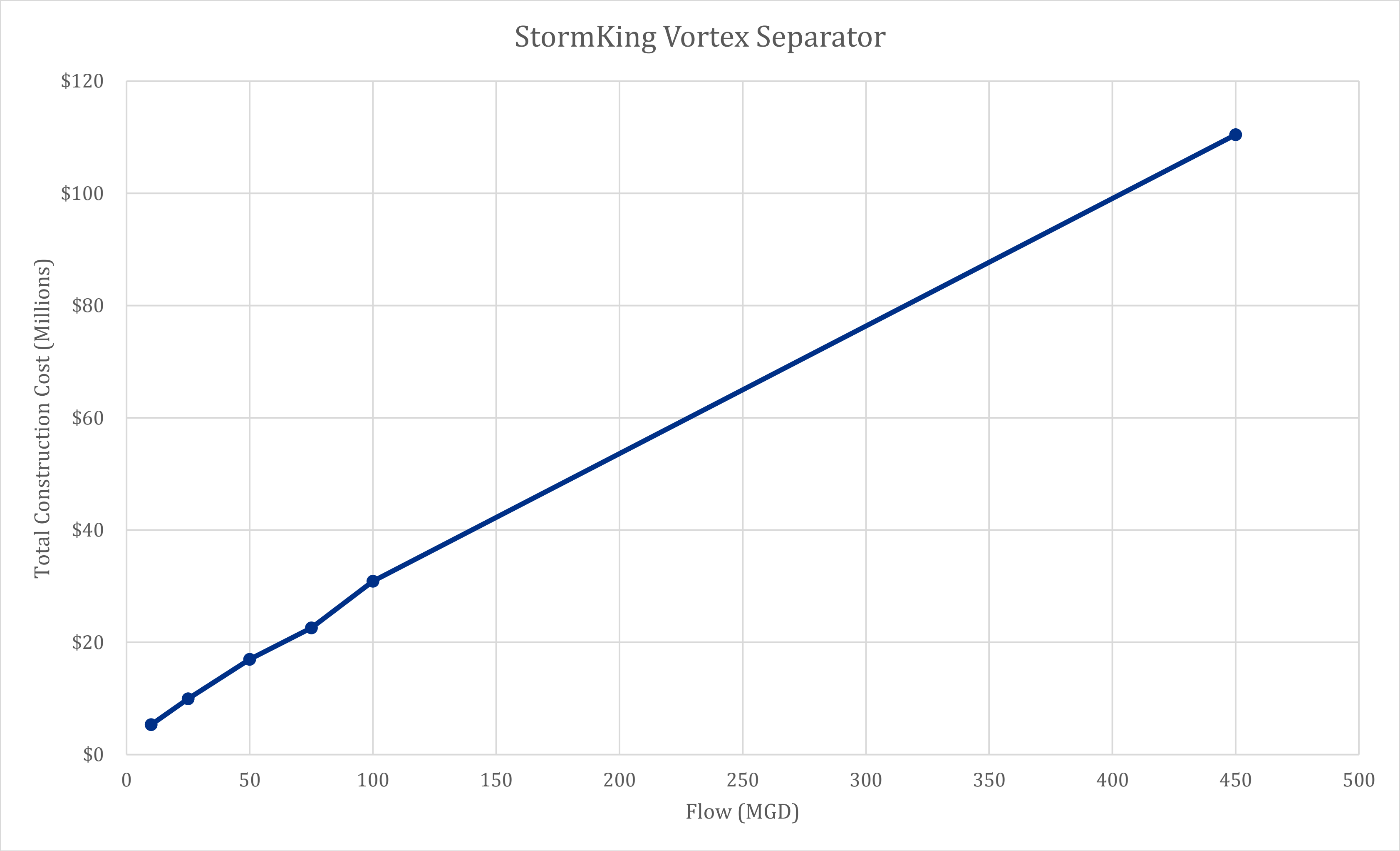


Table 2-9 - Annual Operation Costs of Storm King Vortex Separator

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

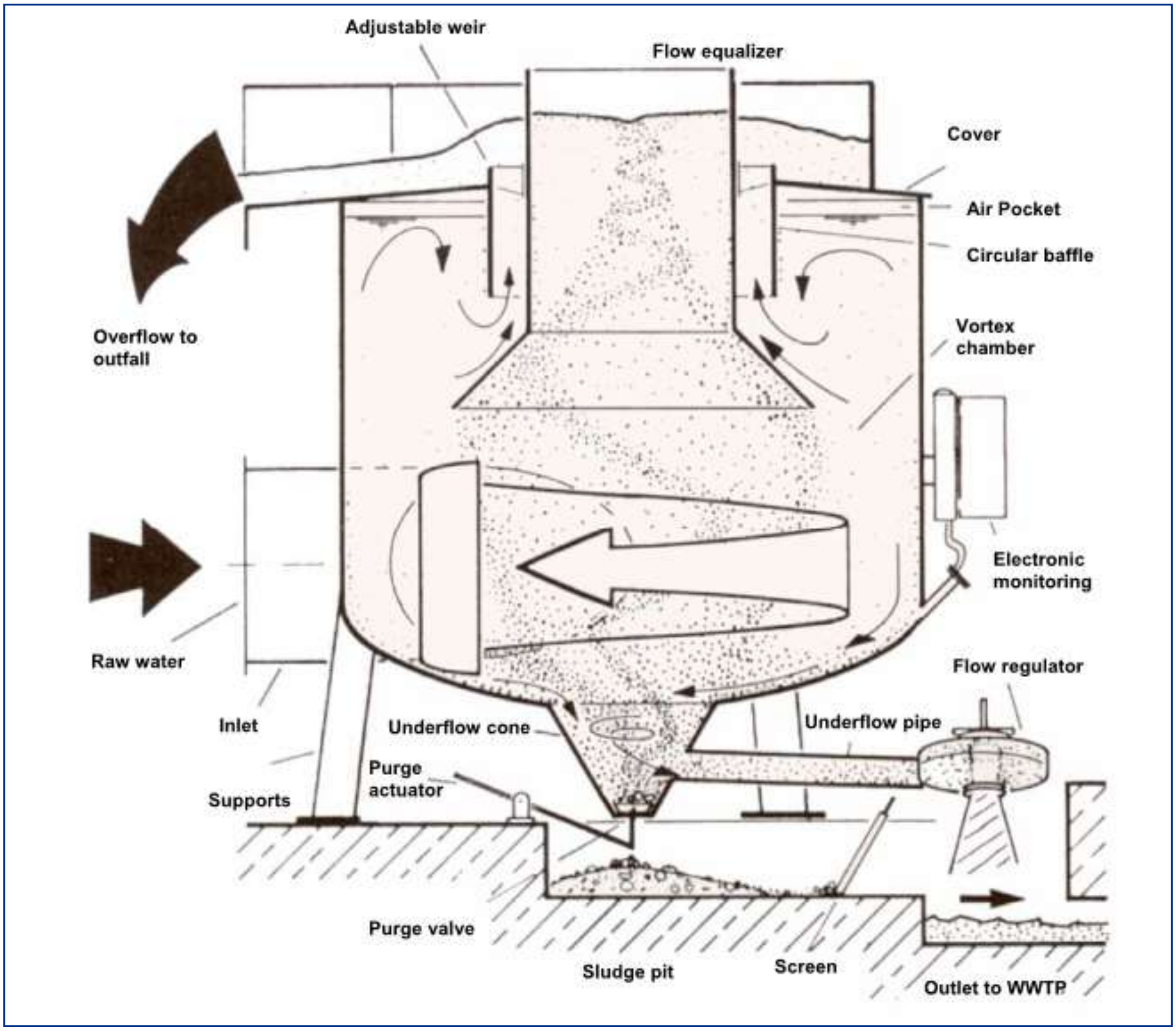
2.3.1.2 HYDROVEX® FluidSep Vortex Separator

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

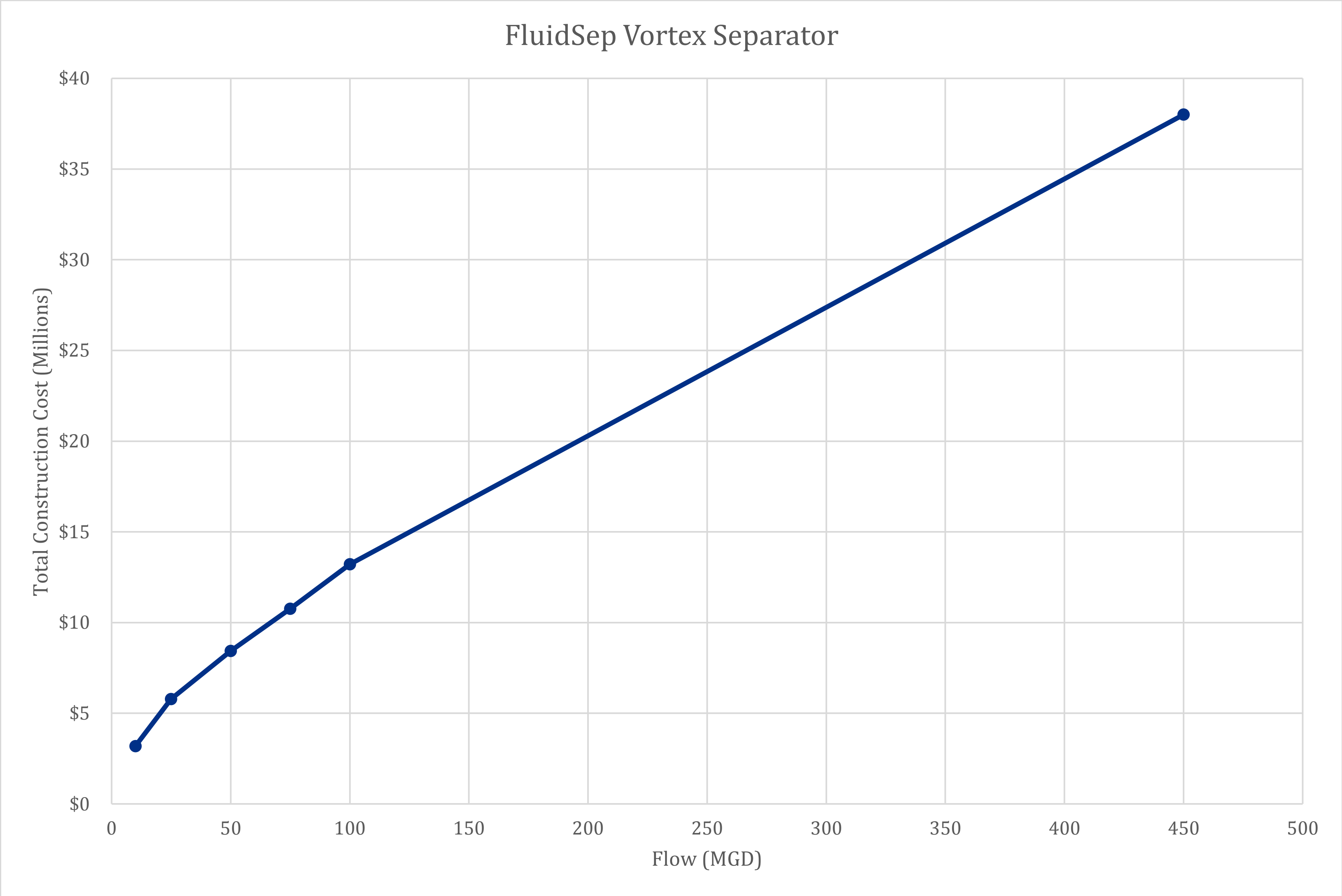


Table 2-12 - Annual Operation Cost of HYDROVEX Fluidsep Vortex Separator

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

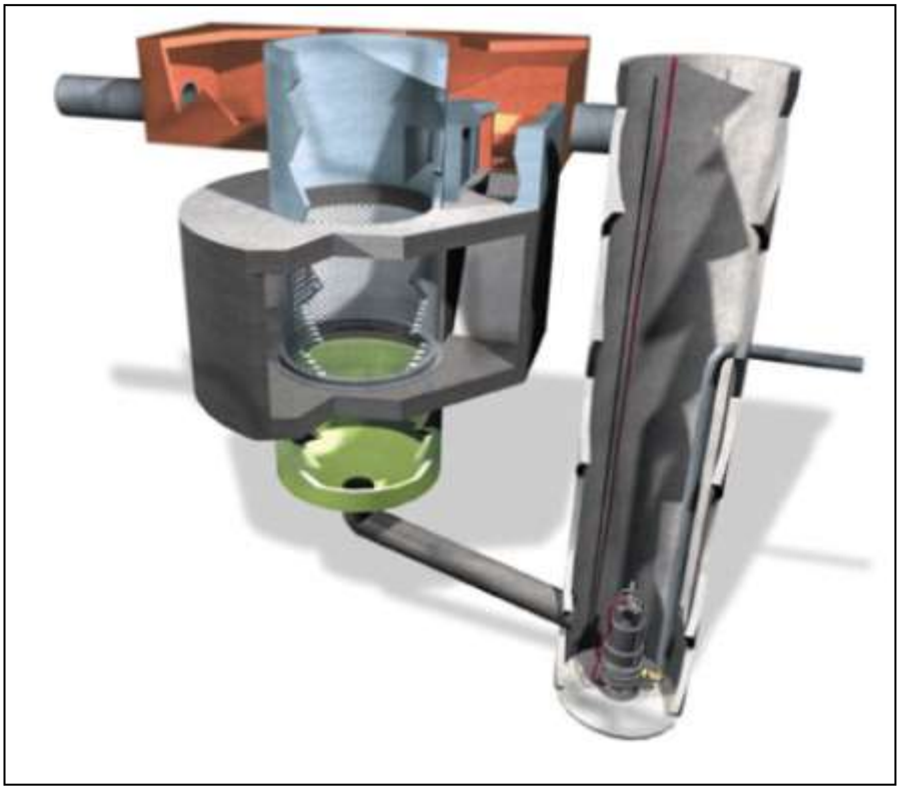
Notes:
(1) Assumes labor rate of $150/hour

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

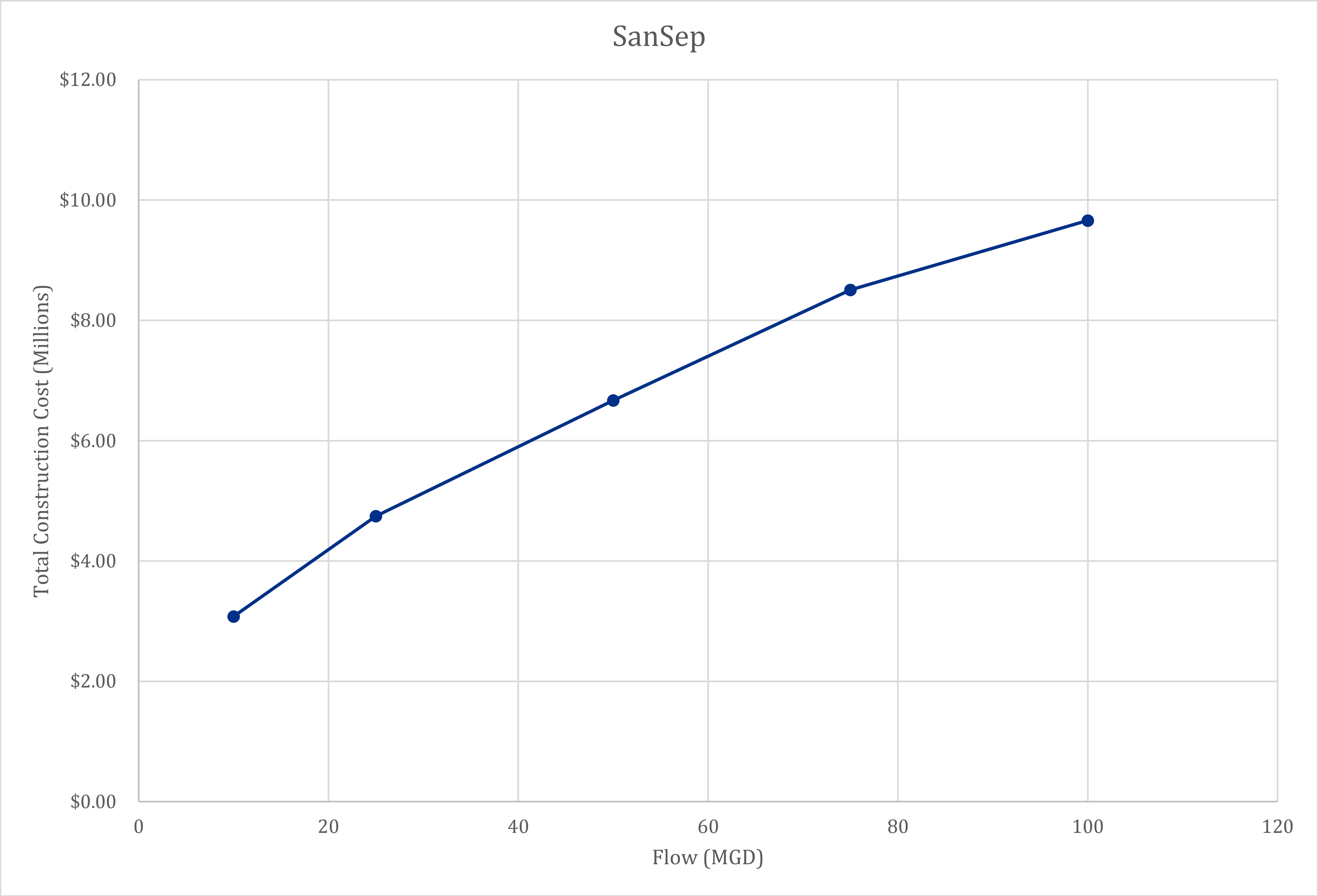


Table 2-15 - Annual Operation Cost of SanSep

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.

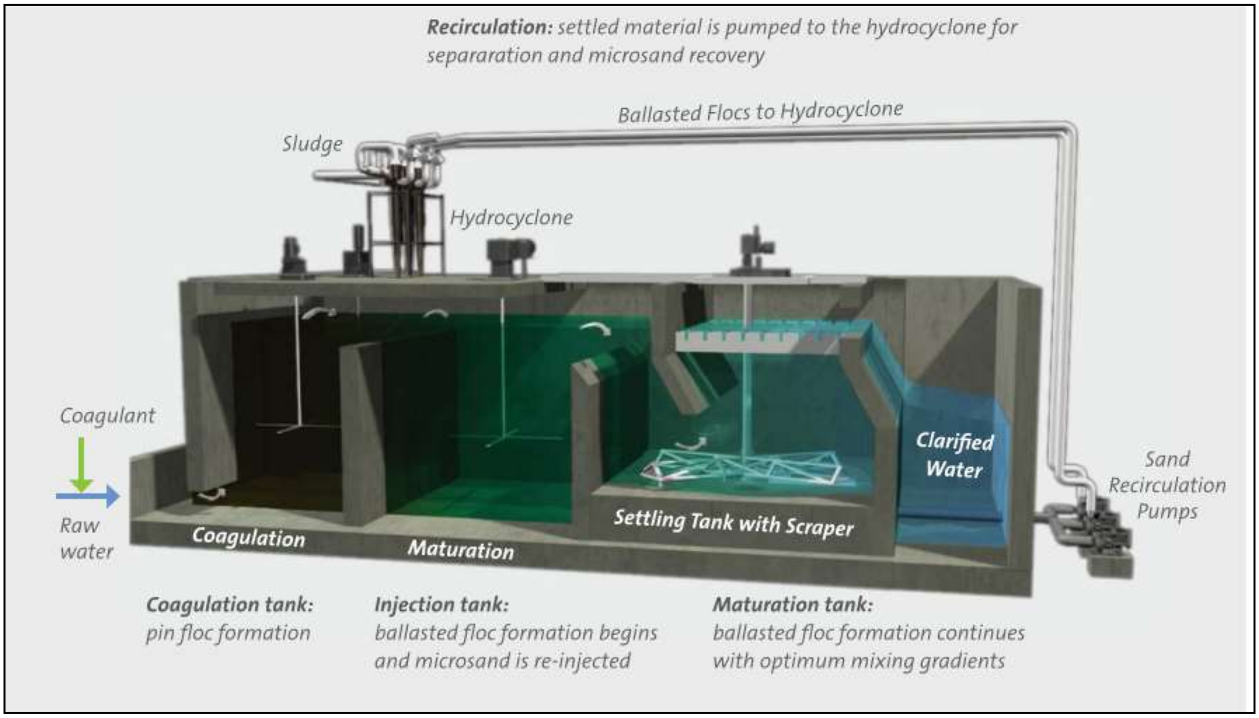
The ballasted flocculation processes, using chemical addition as a critical part of their operation, have higher removal percentages than vortex/swirl separation processes for virtually all the pollutants with the exception of total nitrogen and NH_3. The compact size of ballasted flocculation units 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

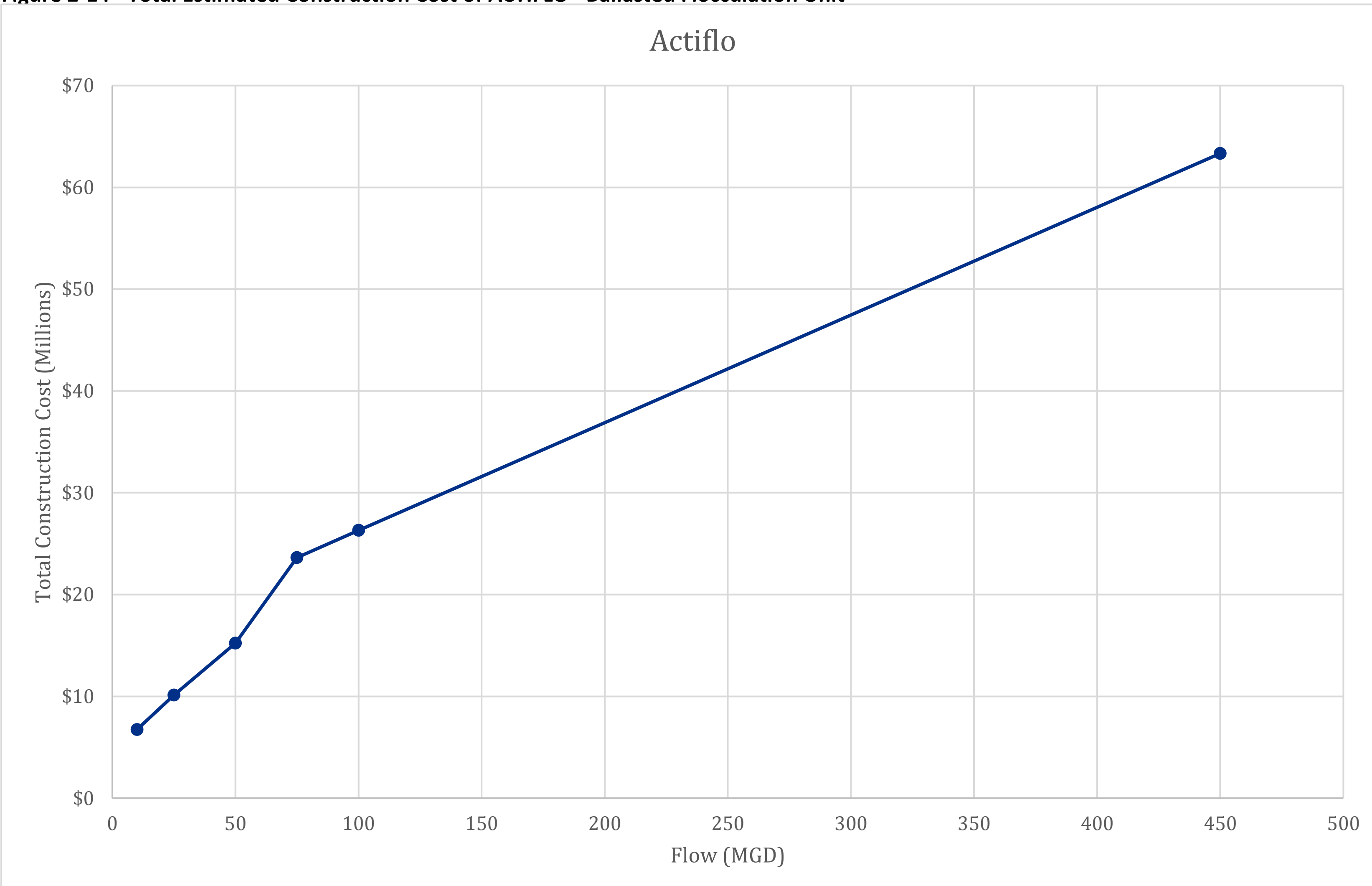


Table 2-19 - Annual Operation Cost of ACTIFLO® Ballasted Flocculation

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.

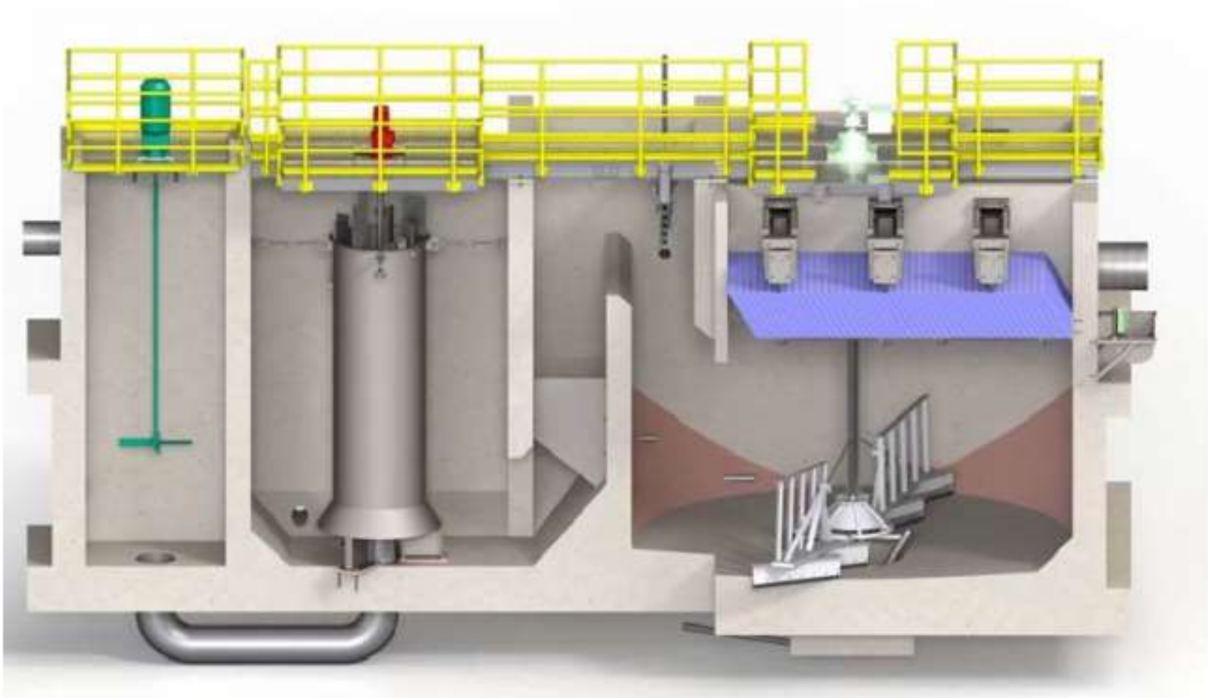
4. Settling & clarification zone: Flocculated solids enter the settling zone, over a submerged weir wall, where dense, suspended matter settles to the bottom of the clarifier. Clarified water is displaced upward from the downward moving slurry, through inclined plate settlers. The plate modules act as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

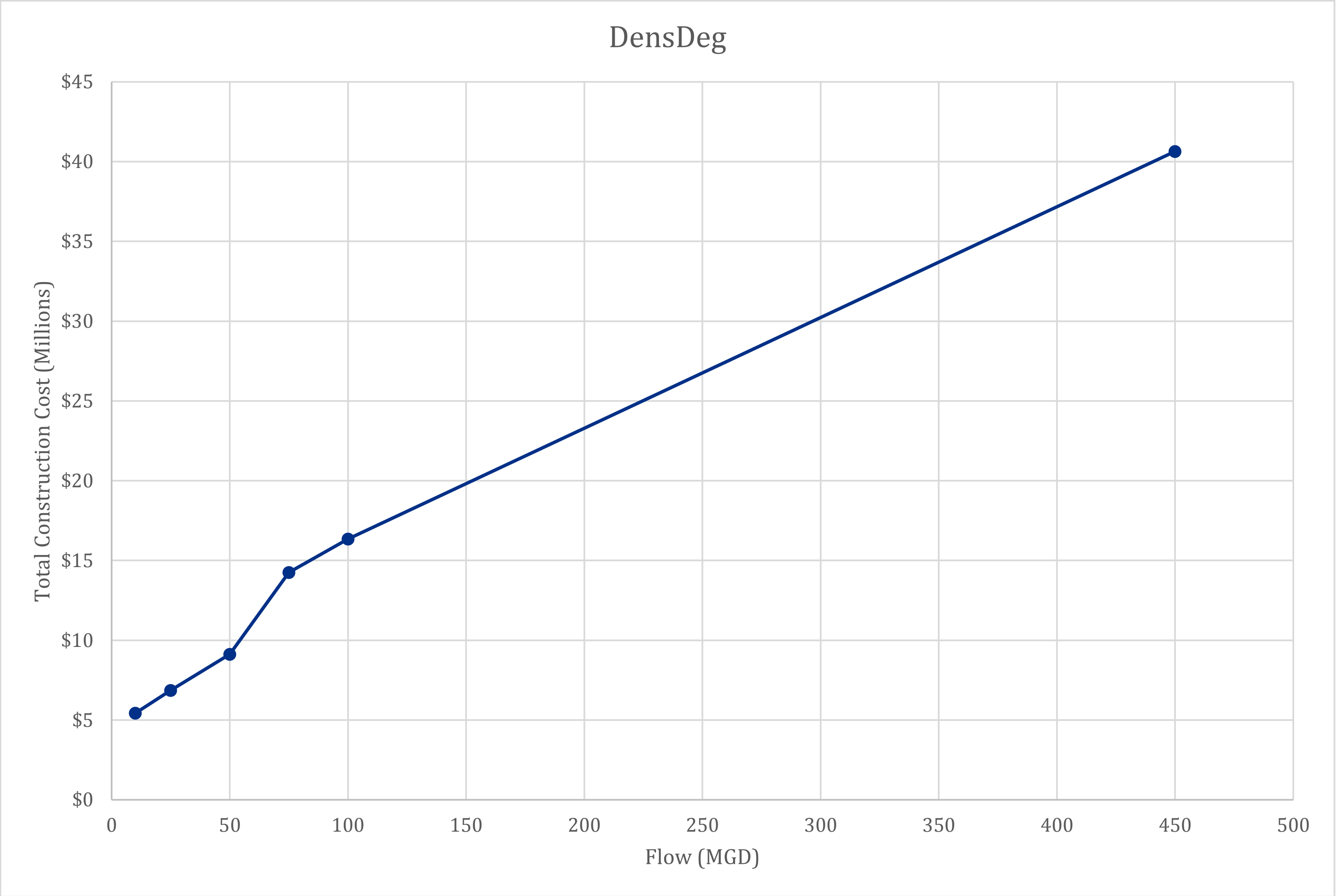


Table 2-22 - Annual Operation Cost of DensaDeg Ballasted Flocculation Unit

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

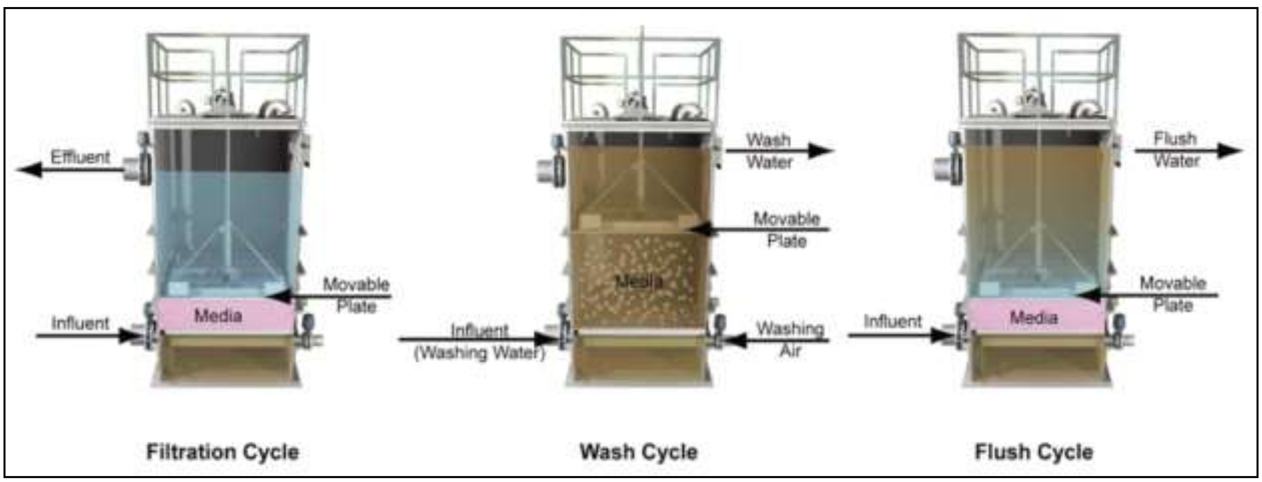
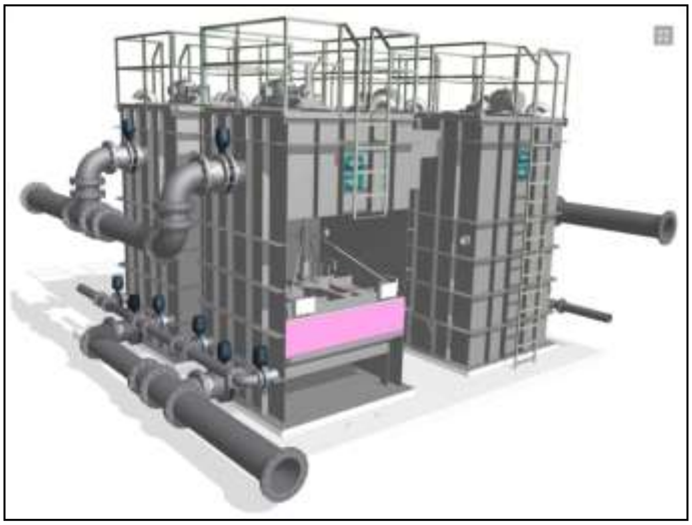


Figure 2-18 - Fuzzy Filter Unit

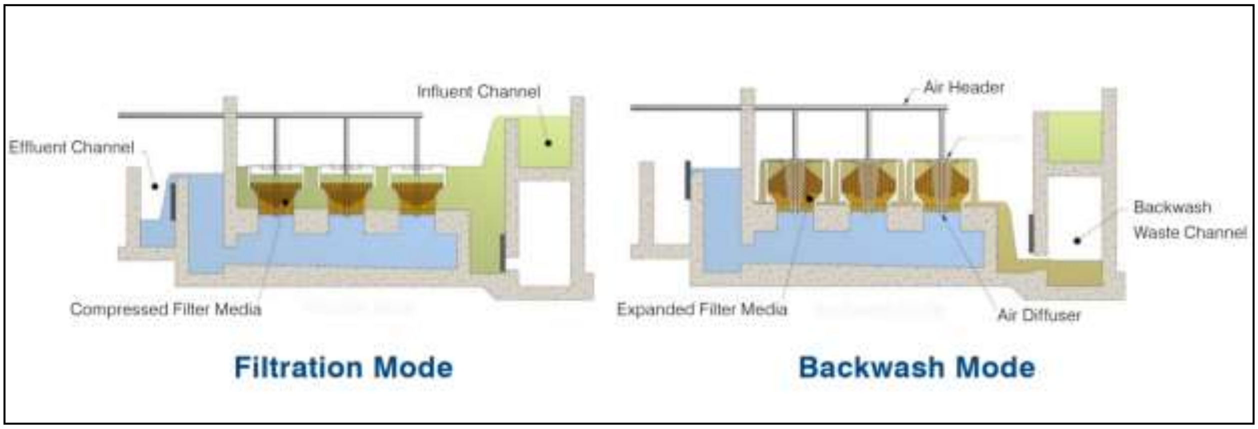


(Source: Schreiber, LLC.)

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

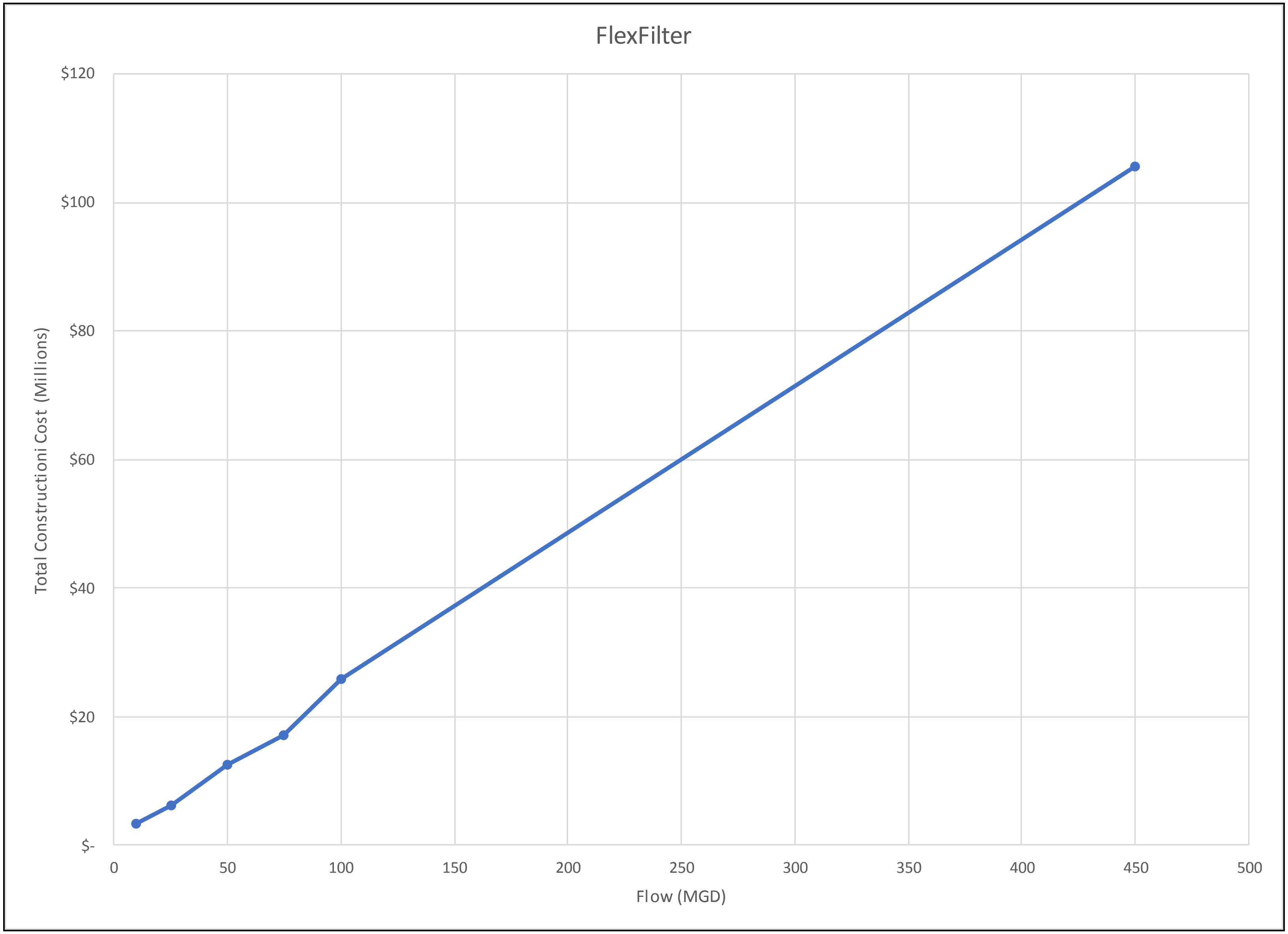


Table 2-25 - Annual Operation and Maintenance Cost of FlexFilter

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

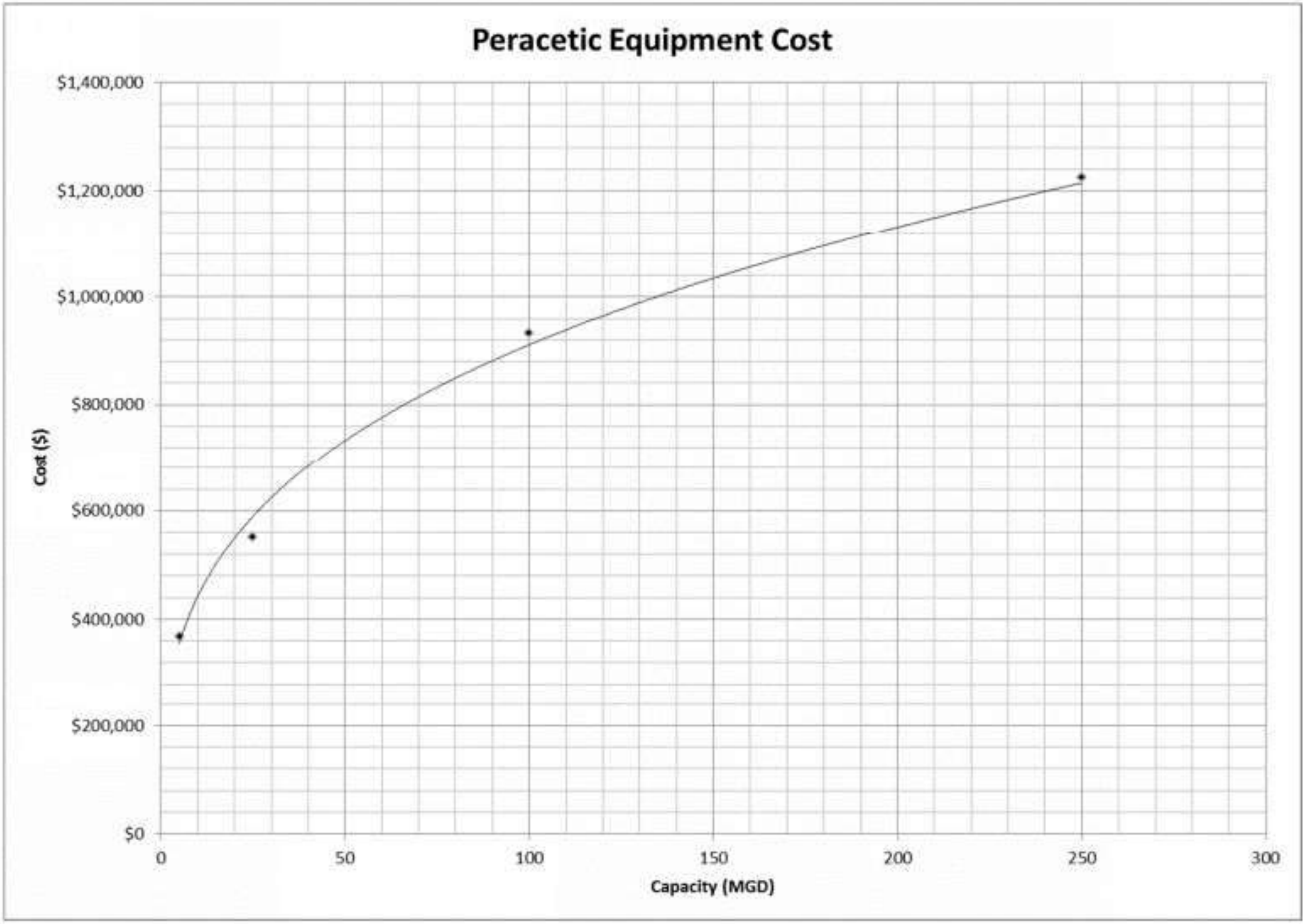
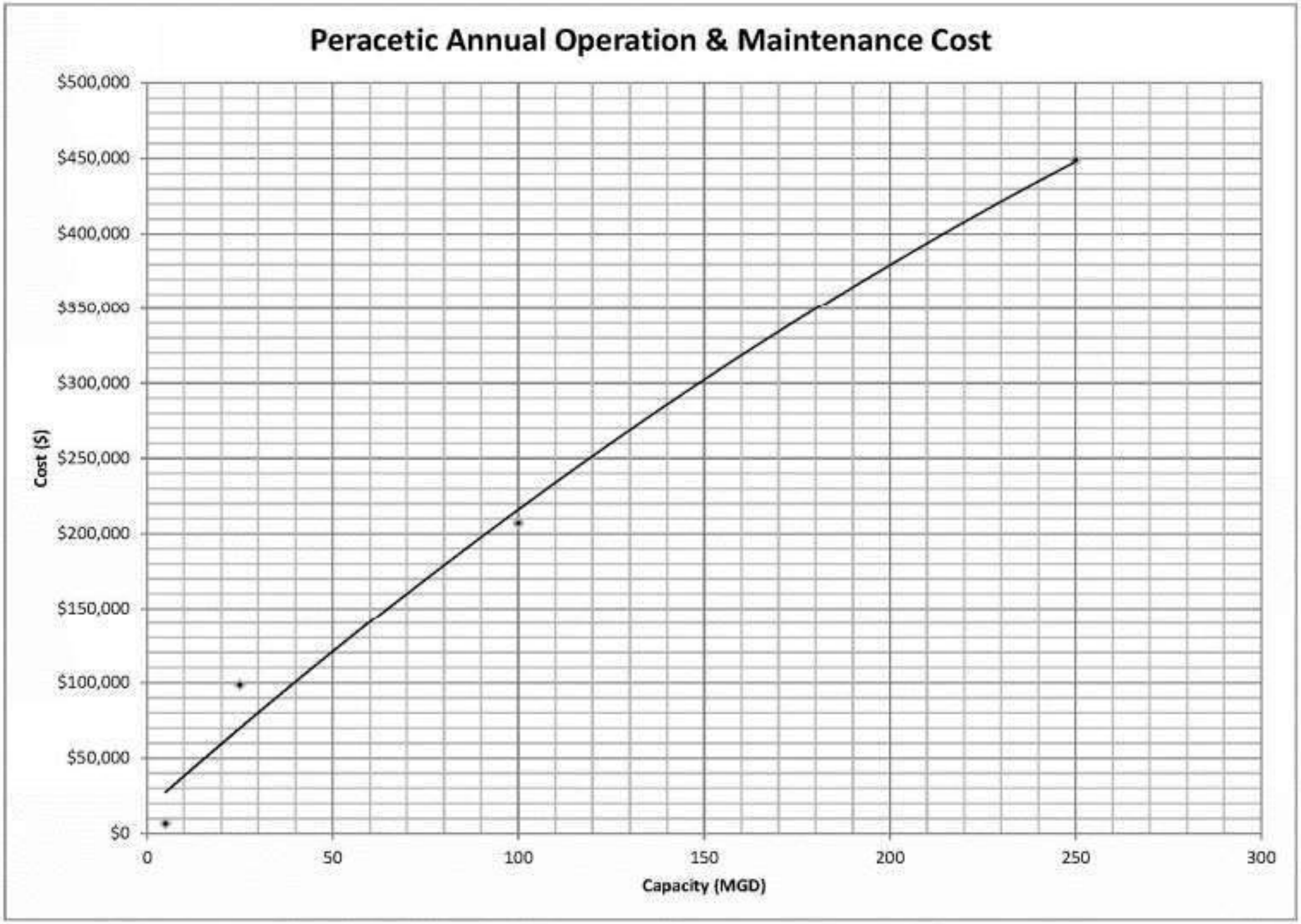


Figure 2-23 - Annual O&M Cost for Peracetic Acid System



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°C. LP-HO lamps operate at temperature range of 100 -150°C and can maintain greater stability of lamp output over a wide range of temperatures. In addition, UV output of LP-HO lamps can be modulated between 30 – 100% to adjust the UV dose.

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

A number of medium-pressure high-intensity/output UV lamps have been developed over the last decade. MP-HO lamps operate at vapor pressure of 10^2 to 10^4 mm Hg while the low-pressure lamps operating at less than 0.8 mm Hg. Also, the operation temperature of MP-HO lamps are significantly higher (600 – 800^{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.

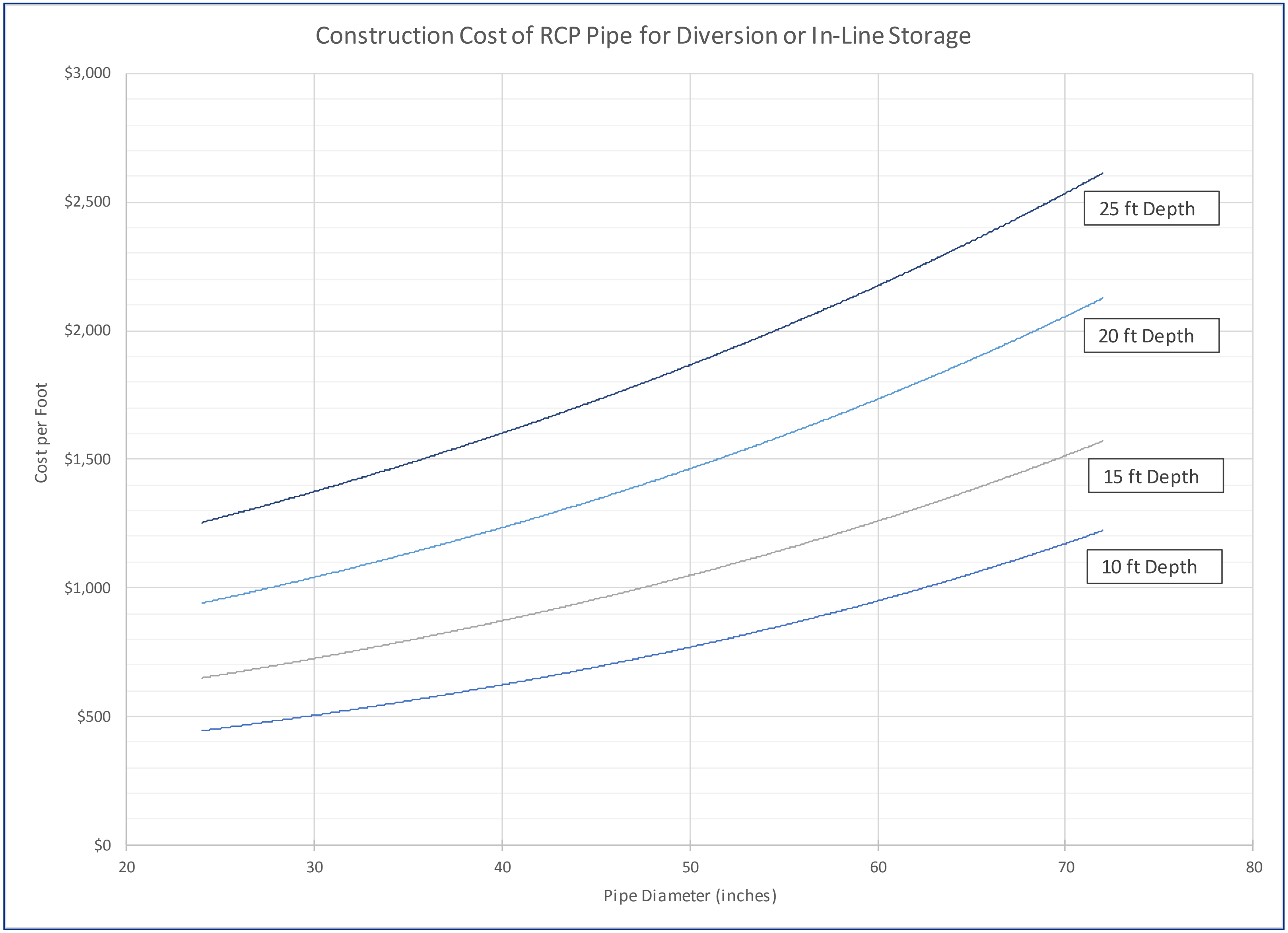
A CSO Facility Plan was completed by Killam Associates (now Mott MacDonald) in 1983 for the Passaic Valley Sewerage Commissioners on the combined sewer systems within the Cities of Newark and Paterson, and Towns of Harrison and Kearny, and the Borough of East Newark. The evaluation of in-line storage 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

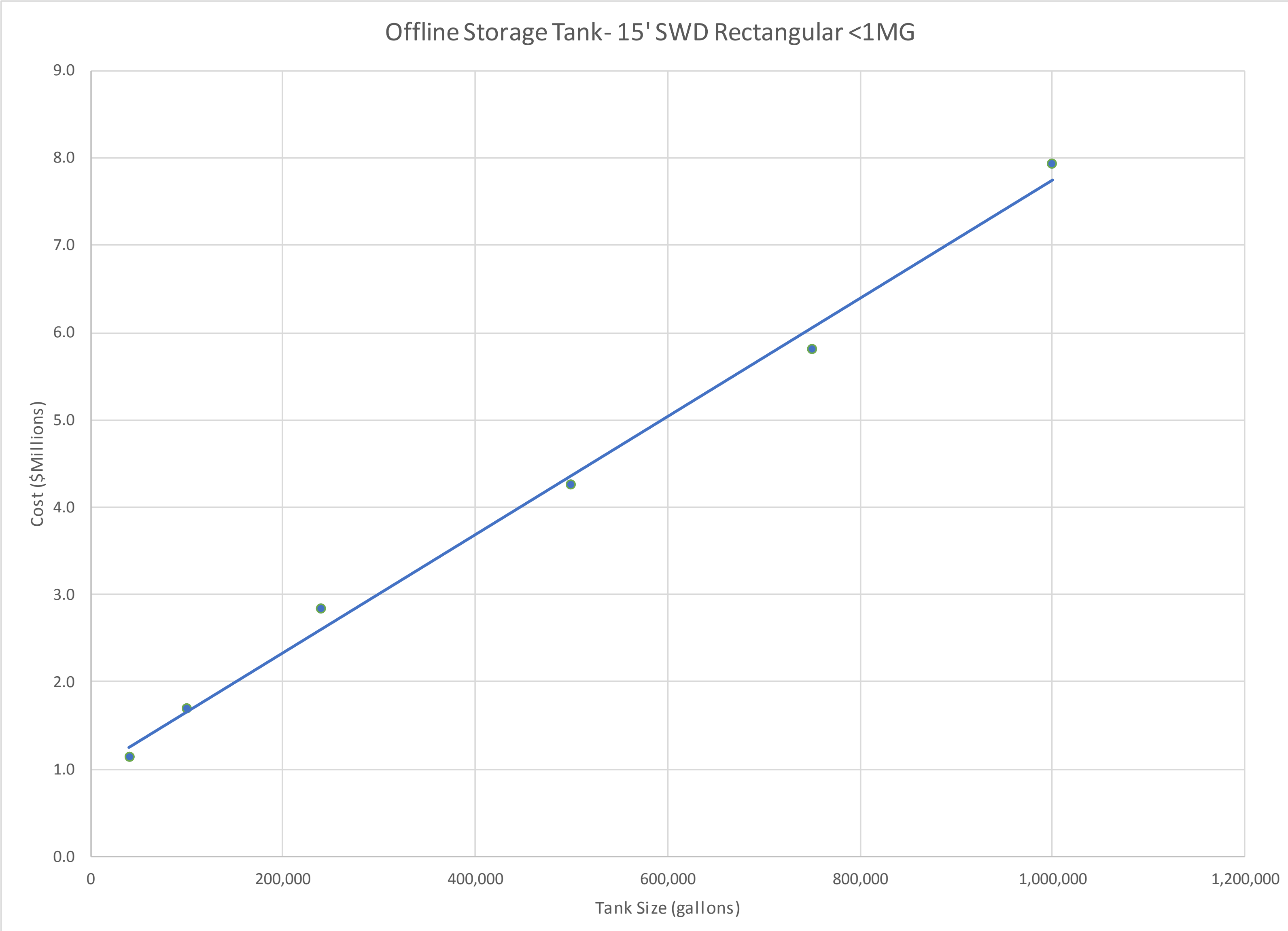


Figure 3-3 - Construction Cost Estimates for Off-Line Storage – 15’ SWD Rectangular > 1 MG

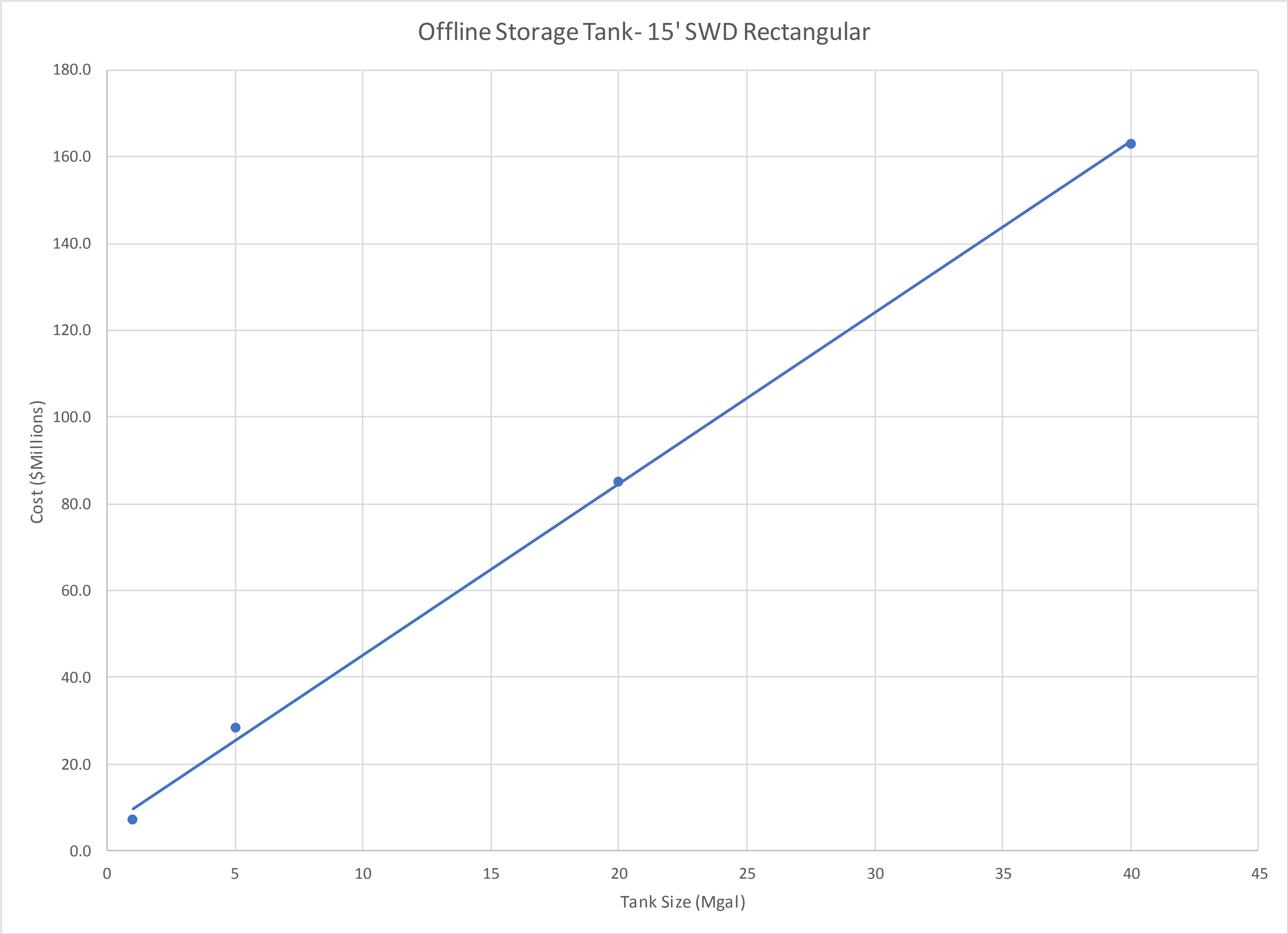


Figure 3-4 - Construction Cost Estimates for Off-Line Storage – 22’ SWD Rectangular

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

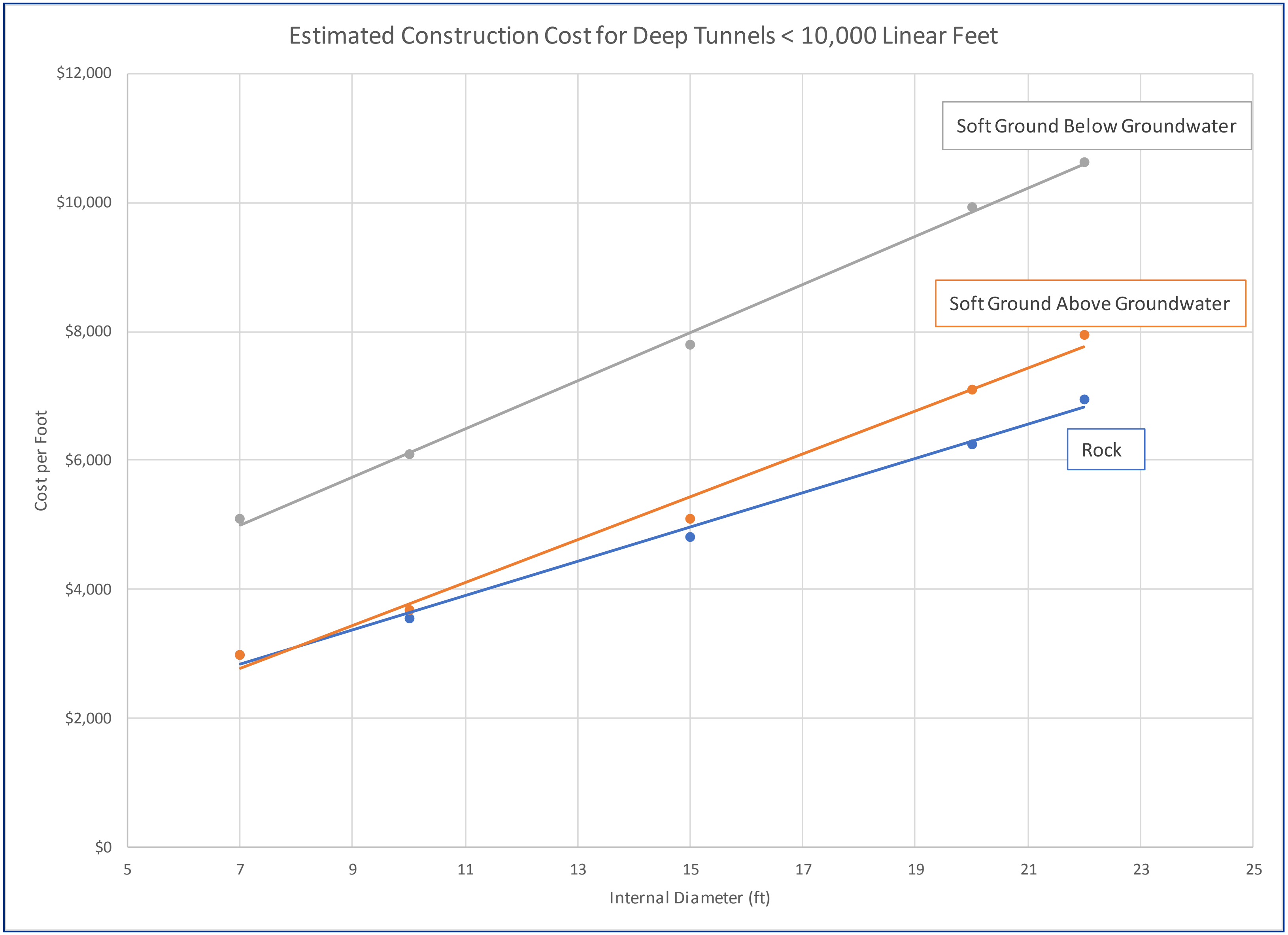


Figure 3-7 - Estimated Cost of Deep Tunnels Greater Than 10,000 Linear Feet

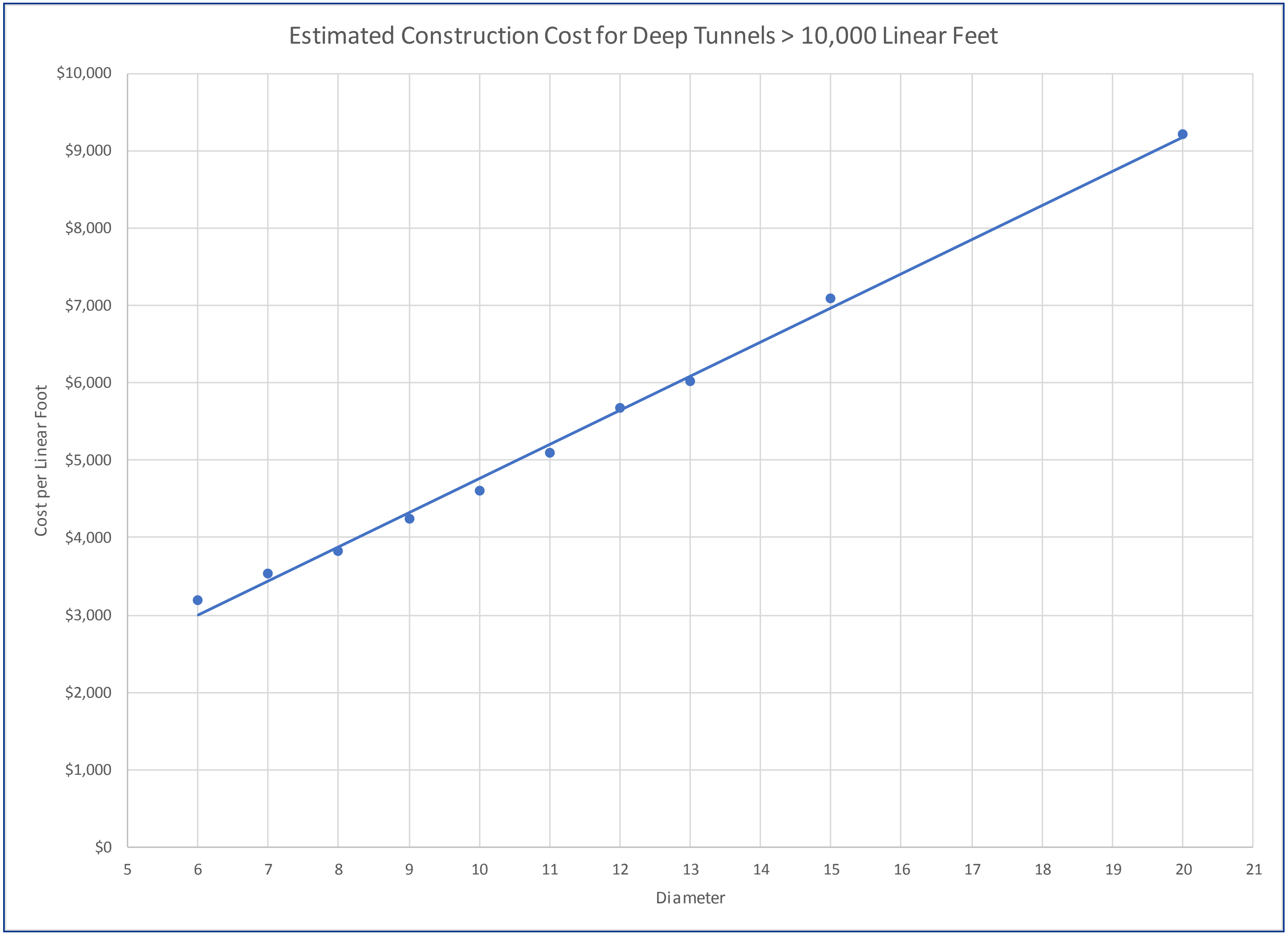
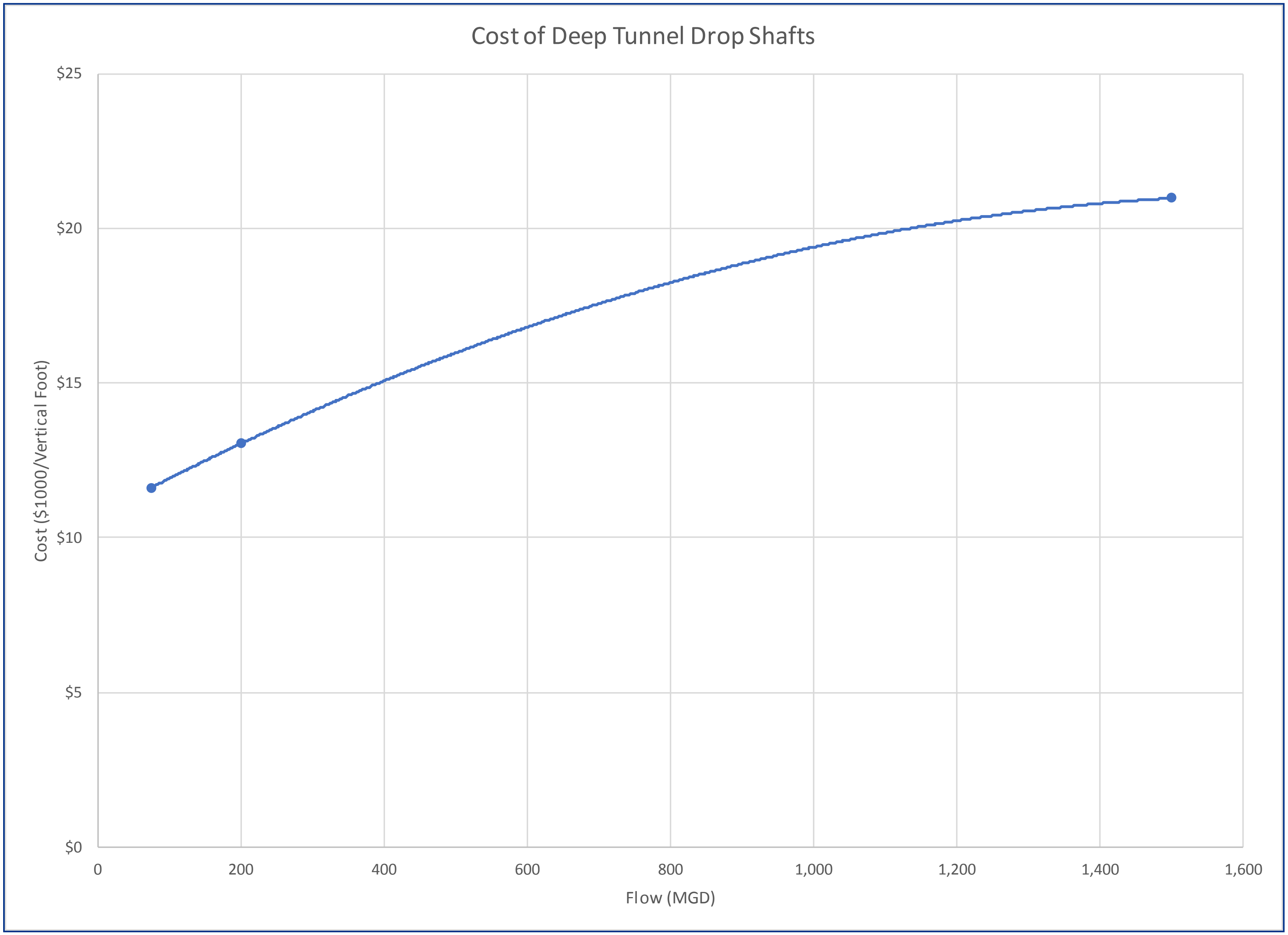


Figure 3-8 - Construction Cost Estimates for Tunnel Drop Shaft



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

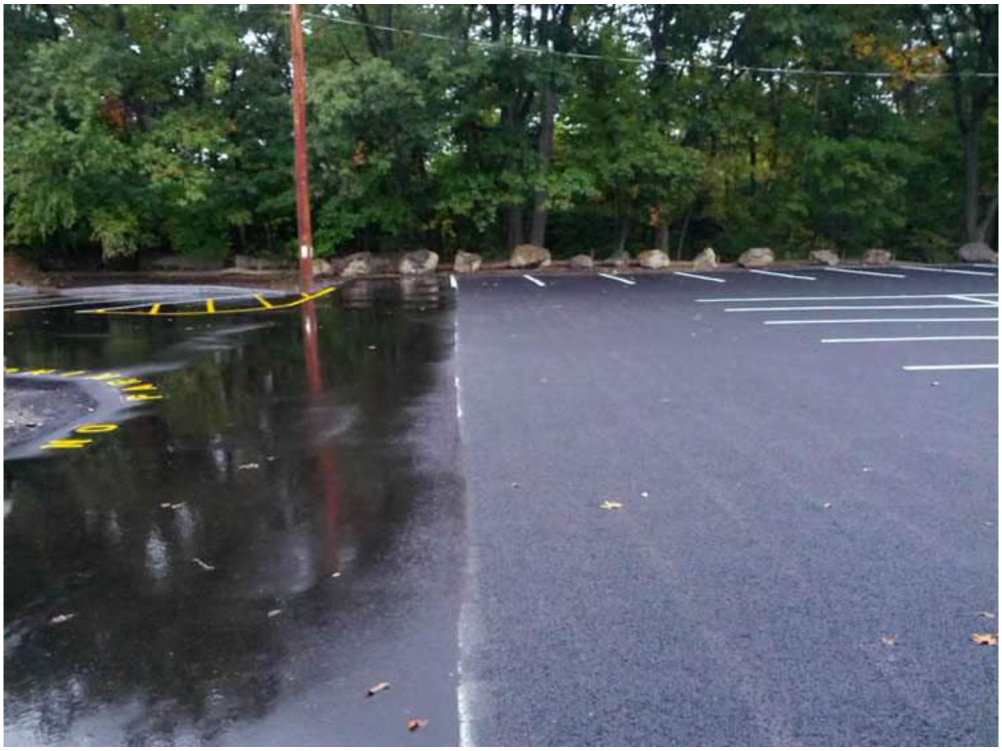
Description of Practice

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

Climber Screen® Installation List
Type IIS and IIIAS
NJ, NY, PA
2000-2015

July 2017

Serial Number	Contract#	State	Location	Name	Year	Qty	Type	Design Flow Rate	Unit of Measure	Channel Width	Channel Depth	Max. Water Depth	Clear Spacing	Channel Invert to Operating Floor	Material - Non Wetted	Material - Wetted
CS-1630	05486	NY	Onondaga County	Baldwinsville Senera Knolls	2005	1	IIS		MGD	48	66		1	360	304SS	304SS
CS-1631	05486	NY	Onondaga County	Ley Creek PS	2005	1	IIS		MGD	48	260.5		1	260.5	304SS	304SS
CS-1632	05486	NY	Onondaga County	Ley Creek PS	2005	1	IIS		MGD	48	260.5		1	260.5	304SS	304SS
CS-1633	05486	NY	Onondaga County	Metropolitan Syracuse Effluent Channel	2005	1	IIS		MGD	71	203.5		0.75	203.5	304SS	304SS
CS-1634	05486	NY	Onondaga County	Metropolitan Syracuse Effluent Channel	2005	1	IIS		MGD	71	203.5		0.75	203.5	304SS	304SS
CS-1635	05486	NY	Onondaga County	Metropolitan Syracuse Effluent Channel	2005	1	IIS		MGD	72	150.625		1.5	150.625	304SS	304SS
CS-1636	05486	NY	Onondaga County	Metropolitan Syracuse Effluent Channel	2005	1	IIS		MGD	72	150.625		1.5	150.625	304SS	304SS
CS-1650	05504	NJ	Rahway	Rahway Valley WWTP	2005	1	IIS	52.5	MGD	72	145	72	3	369	Carbon Steel	304SS
CS-1651	05504	NJ	Rahway	Rahway Valley WWTP	2005	1	IIS	52.5	MGD	72	145	72	3	369	Carbon Steel	304SS
CS-1652	05504	NJ	Rahway	Rahway Valley WWTP	2005	1	IIS	52.5	MGD	72	145	72	3	369	Carbon Steel	304SS
CS-1653	05504	NJ	Rahway	Rahway Valley WWTP	2005	1	IIS	52.5	MGD	72	145	72	3	369	Carbon Steel	304SS
CS-1654	05504	NJ	Rahway	Rahway Valley WWTP	2005	1	IIS	52.5	MGD	72	145	72	3	369	Carbon Steel	304SS
CS-1655	05504	NJ	Rahway	Rahway Valley WWTP	2005	1	IIS	52.5	MGD	72	145	72	3	369	Carbon Steel	304SS
CS-1657	05509	NY	Brooklyn	Paerdegat PS	2005	1	IIIAS	333.0	MGD	108	322	168	1.25	322	Carbon Steel	316SS
CS-1658	05509	NY	Brooklyn	Paerdegat PS	2005	1	IIIAS	333.0	MGD	108	322	168	1.25	322	Carbon Steel	316SS
CS-1659	05509	NY	Brooklyn	Paerdegat PS	2005	1	IIIAS	333.0	MGD	108	322	168	1.25	322	Carbon Steel	316SS
CS-1660	05509	NY	Brooklyn	Paerdegat PS	2005	1	IIIAS	333.0	MGD	108	322	168	1.25	322	Carbon Steel	316SS
CS-1661	05509	NY	Brooklyn	Paerdegat PS	2005	1	IIIAS	333.0	MGD	108	322	168	1.25	322	Carbon Steel	316SS
CS-1662	05509	NY	Brooklyn	Paerdegat PS	2005	1	IIIAS	333.0	MGD	108	322	168	1.25	322	Carbon Steel	316SS
CS-1690	08610	NY	Brooklyn	Newtown Creek WPCP (Replaced 86-119)	2008	1	IIIAS	100.0	MGD	78	148.5	86	1	496.5	Carbon Steel	316SS
CS-1691	08610	NY	Brooklyn	Newtown Creek WPCP (Replaced 86-119)	2008	1	IIIAS	100.0	MGD	78	148.5	86	1	496.5	Carbon Steel	316SS
CS-1692	08610	NY	Brooklyn	Newtown Creek WPCP (Replaced 86-119)	2008	1	IIIAS	100.0	MGD	78	148.5	86	1	496.5	Carbon Steel	316SS

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™ 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	JEFFERSONVI LLE	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)

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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Revised: July 13, 2017

Appendix E

SanSep Installation List

(Source: Echelon Environmental)

SANSEPtm 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

SANSEPtm 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.)

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

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

Appendix B

Wet Weather Upgrades at Delaware No. 1 WPCF, Study of Alternatives

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Wet Weather Upgrades at Delaware No. 1 WPCF

Study of Alternatives

Camden County Municipal Utilities Authority

DRAFT - May 2017
REVISED - December 2017
REVISED - June 6, 2018

GREELEY AND HANSEN

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Executive Summary

The Camden County Municipal Utilities Authority (CCMUA) commissioned a study and design of wet weather upgrades for the Delaware No. 1 Water Pollution Control Facility (WPCF) at 1645 Ferry Avenue, Camden New Jersey. The wet weather upgrades are required to increase hydraulic capacity through the plant during severe storm events to decrease combined sewer overflows in the wastewater collection system and to mitigate related stormwater flooding in portions of downtown Camden.

Based on 2016 data the annual average daily wastewater flow received and treated at the WPCF is approximately 52.6 million gallons per day (mgd). Because of the conveyance system limitations, the rated wet weather capacity of the plant is 150 mgd. Flows above 150 mgd are relieved to the Delaware River through a series of combined sewer overflows in the collection system.

Hydraulic and treatment system limitation studies determined that by improving the hydraulics through select structural modifications, flow capacity through the plant can be increased to approximately 180 mgd. Other more extensive improvements could increase flow capacity in the plant up to 240 mgd. Increasing the hydraulic and treatment capacity through the plant will decrease the number and volume of combined sewer overflows released to the Delaware River and will help to reduce stormwater flooding in low lying areas of the City of Camden.

The possible wet weather improvements at the Delaware No.1 WPCF were studied in two phases. The Phase 1 Study addressed the mechanical, electrical and structural modifications required to increase the capacity of the plant to handle wet weather flows up to 180 mgd. The Phase 1 Study determined that the existing plant could be modified to handle flows up to 180 mgd by improving the plant's hydraulics. The Phase 2 Concept Study is to increase the capacity of the plant to handle wet weather flows of 240 mgd. The Phase 2 Concept Study presents a range of wet weather treatment options that could be developed to provide wet weather treatment for flows up to 240 mgd. All of the Phase 2 Concept Study options include construction of a flow bypass and a new weather outfall. The Phase 2 Concept Study options are intended to be considered together with other ongoing CCMUA wet weather programs and the LTCP. Discussions with NJDEP regarding wet weather treatment requirements are recommended. Further study of wet weather flows, frequency and duration are also recommended.

Phase 1 Study

Investigations began with a hydraulic evaluation of the unit processes within the plant, starting with the headworks and through the outfall discharging into the Delaware River. Assuming that flow can be delivered to the unit processes throughout the plant, the hydraulic study results indicate hydraulic limitations ahead of the Primary Sedimentation Tanks and at the outfall to the Delaware River. After identifying the location or the hydraulic "bottlenecks", a detailed examination of approaches to alleviate the hydraulic restrictions was undertaken. Raising channel walls, increasing inlet gate opening sizes and adding new inlet gate openings at the Primary Sedimentation Tanks; and removal of the outfall nozzles, adding ports in the Outfall Well Chamber and providing a relief outlet stack on the outfall pipes ahead of the Outfall Well Chamber were some of the options considered.

After an examination of two (2) possible options, the addition of twenty (20) new inlet gates at the Primary Sedimentation Tanks and construction of two (2) relief stacks on the outfall pipes were the most economical and practical approach to increase flow capacity up to 180 mgd through the plant. Based on the assessment of the wastewater treatment system processes the plant with the upgrade in raw wastewater pumps be capable of treating the wastewater flows for a 180 mgd wet weather flow. Removing the plant-wide bottlenecks will increase the hydraulic capacity to 180 mgd through the plant. The estimated construction costs of the recommended Phase 1 wet weather hydraulic improvements is $1,797,000.

Phase 2 Concept Study

Various viable options to increase the treatment capacity of the CCMUA Delaware No. 1 WPCF during wet weather events are summarized in the Phase 2 Concept Study Report with probable capital and annual costs. A range of wet weather options is presented since CCMUA must consider them together with other ongoing wet weather programs, collection system enhancements, and LTCP work being done separately. Some wet weather treatment expansion options may be a better overall fit for CCMUA than others based on an evaluation of the overall wet weather program, the results of discussions with NJDEP, and the frequency and duration of various design wet weather events.

The options evaluated within the Concept Study offer different levels of treatment and system reliability at different capital and life cycle costs. Further, the options can be classified as robust, less robust and least robust. Although the least robust options (C-1 and C-2) offer the lowest construction costs and ultimately the lowest present worth, they also provide the highest risk of permit non-compliance during high wet weather flow events depending on treatment requirements established by NJDEP. The capital, annual, and life cycle costs of the viable options are provided. With this information CCMUA will be able to select a combination of options for wet weather treatment by reviewing the information presented in this Report, and considering the overall program including costs and corresponding risks. It is recommended that discussions be held with NJDEP regarding wet weather effluent limits.

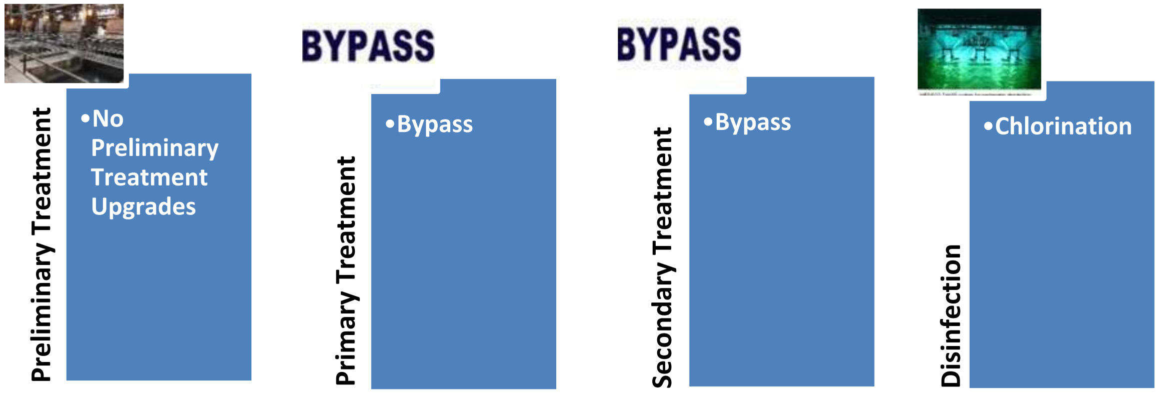
Under the Phase 2 Study, strategies to manage hydraulic flows through the plant up to 240 mgd are reviewed and included under four general option categories:

A. **Wet Weather Storage Tank:** Stores the wet weather flow volumes in excess of 150 mgd in a large underground tank until the wet weather storm event passes and the plant has capacity to treat the stored wastewater

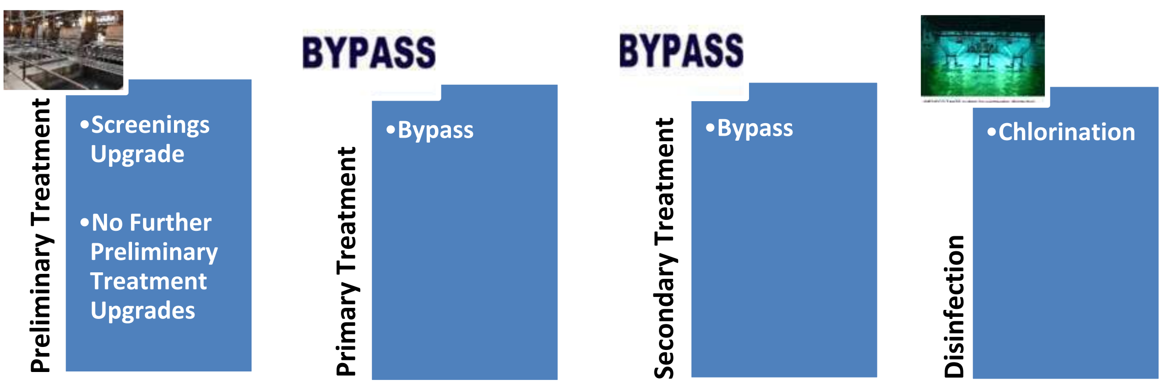
B. **All Flow Treatment:** Full treatment of wet weather flows including primary, secondary and disinfection

C. **Wet Weather Plant Bypass:** Up to 55 mgd would bypass the existing primary and secondary treatment facilities after preliminary treatment and be discharged into the Delaware River through a new wet weather outfall. This option can be developed to provide various wet weather treatment levels ranging from disinfection only to full preliminary and primary treatment, each of which are briefly described below.

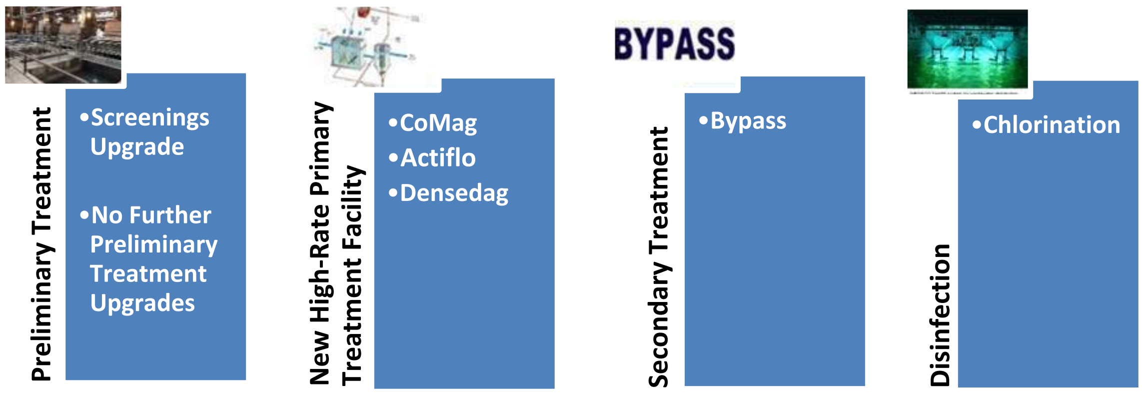
C-1) Standard-rate or high-rate disinfection with no preliminary treatment or raw sewage pumping upgrades.



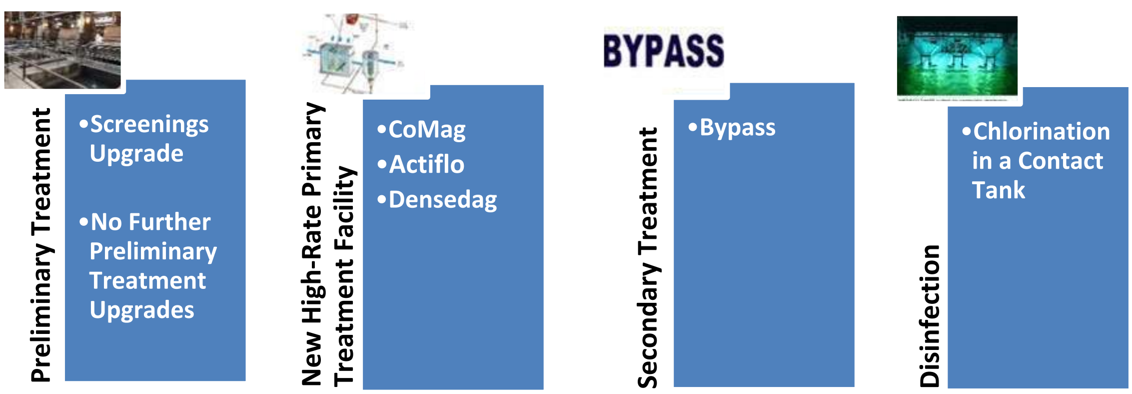
C-2) Screening of raw wastewater to 0.50 inch followed by standard-rate or high-rate disinfection with no grit removal system or raw sewage pumping upgrades.



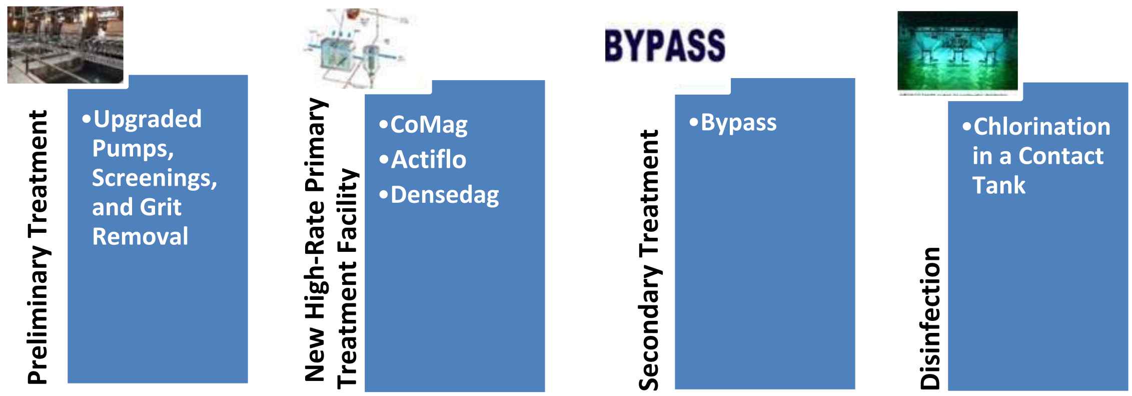
C-3) Screening of raw wastewater to 0.50 inch followed by high-rate primary treatment in area north of existing aeration/final tanks, and standard-rate or high-rate disinfection with no grit removal system or raw sewage pumping upgrades.



C-4) Sreening of raw wastewater to 0.50 inch followed by high-rate primary treatment in area north of existing aeration/final tanks, standard-rate or high-rate disinfection, and chlorine contact tank with no grit removal system or raw sewage pumping upgrades.

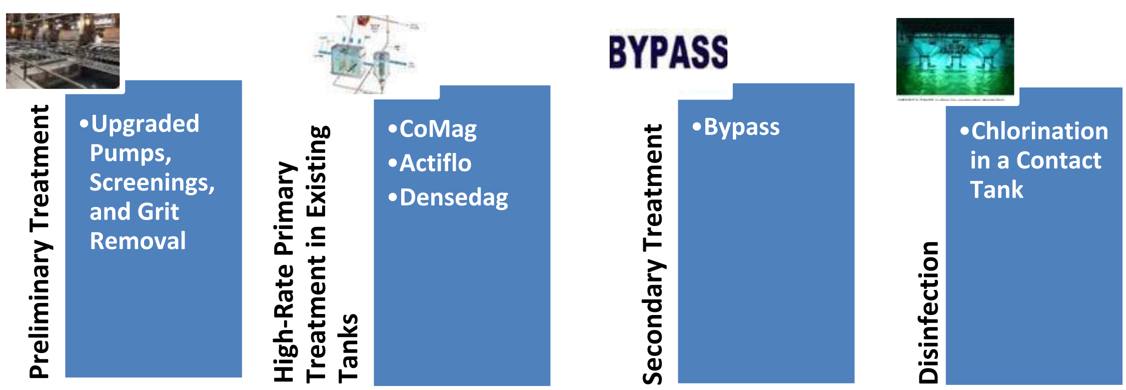


C-5) Preliminary treatment and raw sewage pumping upgrades followed by high-rate primary treatment in area north of existing aeration/final tanks, standard-rate or high-rate disinfection, and chorine contact tank to provide a firm capacity of 240 mgd.

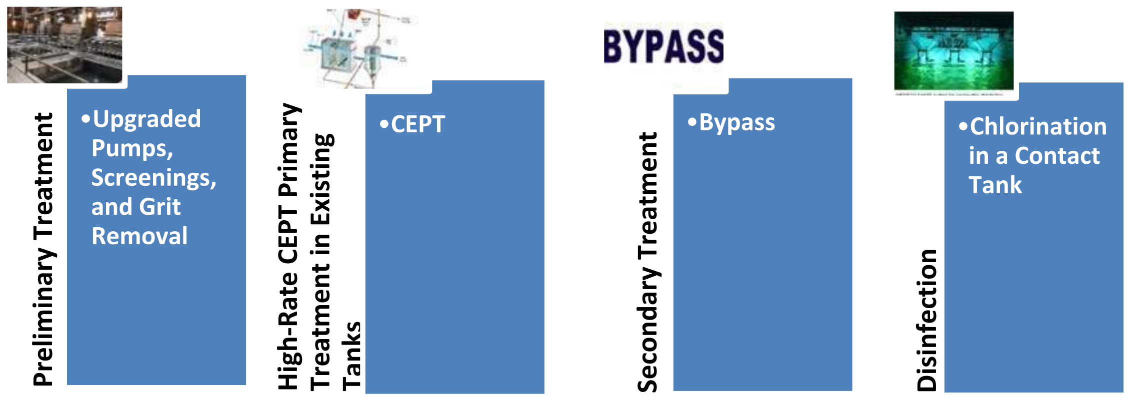


D. Secondary Bypass: Uses enhanced primary treatment system constructed within the existing Primary Sedimentation Tanks. There are two options for implementation consideration, which are briefly described below.

D-1) Preliminary treatment and raw sewage pumping upgrades followed by up to 92 mgd of high-rate primary treatment constructed within existing Primary Sedimentation Tanks followed by standard-rate or high-rate disinfection, and chorine contact tank to provide a firm capacity of 240 mgd.



D-2) Preliminary treatment and raw sewage pumping upgrades followed by Chemically Enhanced Primary Treatment in existing Primary Sedimentation Tanks followed by standard-rate or high-rate disinfection, and chorine contact tank to provide a firm capacity of 240 mgd.



Because of a lack of available space at the WPCF, construction of a new **Wet Weather Storage Tank** is not considered feasible and will not be given further consideration. With the onsite Wet Weather Storage Tank option considered infeasible, the study concentrates on the remaining options.

Providing **All Flow Treatment** with primary and secondary treatment with disinfection for the full 240 mgd peak wet weather flow is relatively complex. The high-rate secondary treatment processes reviewed are not well suited for intermittent operations and these systems would need to be in service at all times. Consequentially, a significant operations change and major capital expense are required to provide new secondary treatment facilities for the peak wet weather flows. Further from an engineering perspective, changing most of the existing wastewater treatment process to accommodate intermittent wet weather flows was considered unnecessary. For these reasons the All Flow Treatment approach was removed from further consideration.

The **Wet Weather Plant Bypass** provides a range of wet weather treatment capacity expansion options for consideration where planning and implementation could be considered in steps, with progressively higher levels of performance and reliability ranging from the least level of treatment Option C-1 to the highest level of treatment Option C-5.

Under options for **Wet Weather Plant Bypass** (bypass primary and secondary treatment) for wet weather flows above 180 mgd up to 240 mgd, there are three general system classifications for the options that can be implemented. The first classification is the "least robust". In the first classification, flows above 180 mgd bypass primary and secondary treatment and receive disinfection only. This includes Options C-1 and C-2. These options may need a regulatory wet weather effluent standard when plant flows are above 180 mgd as they do not provide primary treatment for wet weather flows and require all of the preliminary treatment facility equipment to be available for operation.

The second classification is considered "less robust" and provides high-rate primary treatment of flows above 180 mgd, but does not upgrade the existing capacities of the raw wastewater pumping system or the preliminary treatment systems (screening and grit removal systems). Under the second classification the system can treat flows up to 240 mgd if all unit process systems are available (firm capacity is 180 mgd). This classification includes Options C-3 and C-4. These options may need a regulatory wet weather effluent standard as they require all preliminary treatment systems to be operational with no backup capacity.

The third classification is "robust", which provides high-rate primary treatment of flows above 180 mgd and upgrades the capacities of the raw wastewater pumping and preliminary treatment systems (screening and grit removal systems) to treat wet weather flow with a firm capacity of 240 mgd. This classification includes Option C-5. This option is expected to meet the requirements of the current effluent discharge permit.

Both options under the **Secondary Bypass** are classified as "robust" with a firm capacity to treat 240 mgd of wet weather flow and they are expected to meet the current plant effluent discharge permit. Both of these options use high-rate primary treatment technologies installed with the existing primary tanks, followed by a secondary treatment bypass with disinfection prior to discharge through a new wet weather outfall.

Option D-1 utilizes high rate primary treatment systems constructed within the existing Primary Sedimentation Tanks 9 and 10, followed by chlorine disinfection in the secondary treatment bypass channel. The high rate primary treatment system would be sequentially placed into service when flows exceed 148 mgd.

Option D-2 utilities Chemical Enhanced Primary Treatment in three of the Primary Sedimentation Tanks followed by chlorine disinfection in the secondary treatment bypass. The chemical enhanced primary treatment would be sequentially placed into service when flows exceed 129 mgd.

The probable construction costs for implementation of the Options C-1 through C-5 and Options D-1 and D-2 range from $816,000 to $31,478,000 excluding engineering and CCMUA's administrative costs. A brief wet weather system description along with the capital, annual and life cycle cost for each option is provided in **Table ES-1**.

Table ES-1: Wet Weather Options Summary

	Firm Capacity 180 MGD*			
	Bypass Primary & Secondary Treatment			
Options	**C-1**	**C-2**	**C-3**	**C-4**
System Classification	**Least Robust**	**Least Robust**	**Less Robust**	**Less Robust**
Wet Weather Treatment Systems Description	• 55 mgd Bypass of Primary and Secondary Treatment through New Wet Weather Outfall • Standard or High-Rate Disinfection	• 55 mgd Bypass of Primary and Secondary Treatment through New Wet Weather Outfall • Upgrade Screenings • Standard or High-Rate Disinfection	• 55 mgd Bypass of Primary and Secondary Treatment through New Wet Weather Outfall • Upgrade Screenings • Add 55 MGD High-Rate Primary Treatment • Standard or High-Rate Disinfection	• 55 mgd Bypass of Primary and Secondary Treatment through New Wet Weather Outfall • Upgrade Screenings • Add 55 MGD High-Rate Primary Treatment • Standard or High-Rate Disinfection • Add Disinfection Capacity
Capital Cost**	$816,000	$2,447,000	$10,602,000	$13,048,000
Annual Cost	$100,000	$200,000	$300,000	$300,000
Life Cycle Cost	$2,066,000	$4,947,000	$14,352,000	$16,798,000

	Firm Capacity 240 MGD		
	Bypass Primary & Secondary Treatment	Bypass Secondary Treatment	
Options	**C-5**	**D-1**	**D-2**
System Classification	**Robust**	**Robust**	**Robust**
Wet Weather Treatment Systems Description	• 55 mgd Bypass of Primary and Secondary Treatment through New Wet Weather Outfall • Expand Preliminary Treatment • Add 55 MGD High-Rate Primary Treatment • Standard or High-Rate Disinfection • Add Disinfection Capacity	• 55 mgd Bypass of Secondary Treatment through New Wet Weather Outfall • Expand Preliminary Treatment • Add 92 MGD High Rate Primary Treatment in Exist. PSTs • Standard or High-Rate Disinfection • Add Disinfection Capacity	• 55 mgd Bypass of Secondary Treatment through New Wet Weather Outfall • Expand Preliminary Treatment • Add Chemically Enhanced Primary Treatment in Exist. PSTs • Standard or High-Rate Disinfection • Add Disinfection Capacity
Capital Cost**	$24,954,000	$31,478,000	$23,323,000
Annual Cost	$900,000	$900,000	$900,000
Life Cycle Cost	$36,204,000	$42,728,000	$34,573,000

Notes: * Plant can treat 240 MGD if all components of each unit process are available for service.
** Excludes engineering and CCMUA's administration fees

Section 1 Introduction

Project Overview

The objective of the Wet Weather Upgrades Project is to develop the conceptual designs for upgrading the wet weather treatment capacity of the Camden County Municipal Utilities Authority's (CCMUA's) Delaware No. 1 Water Pollution Control Facility (WPCF) to enable the plant to provide sufficient hydraulic capacity to treat the flow generated by a ten (10) year storm, while complying with applicable requirements and regulations of the New Jersey Department of Environmental Protection (NJDEP).

The plant's wastewater treatment process is comprised of a raw sewage pump station, preliminary treatment facility with bar screens and grit removal systems (detritus tanks), ten primary sedimentation tanks, eight aeration tanks, eight final sedimentation tanks, two chlorine contact tanks, and an outfall system discharging into the Delaware River. The plant is located on the Delaware River south of downtown Camden, New Jersey (See **Appendix A** - Vicinity Map). In order to evaluate proposed upgrades, an understanding of the hydraulics that exist within the plant is required. To accomplish the hydraulic capacity of the plant, a hydraulic model was developed and calibrated for the Delaware No. 1 WPCF. The model identified existing hydraulic bottlenecks and was used to evaluate alternatives to increase the WPCF's overall hydraulic capacity. Bottlenecks were defined as locations where the water surface encroached within 6-inches of the freeboard (below the conduit or tank walls) or where weirs became submerged.

Proposed wet weather upgrades are to occur in two phases. Phase 1 upgrades will allow the WPCF to receive wet weather flows up to 180 MGD using the existing facilities without major revisions to existing structures, and without construction of new structures or unit processes. Proposed Phase 2 upgrades will allow the plant to treat up to 240 MGD of wet weather flows through the construction of new facilities or new unit processes.

Section 2 Background

The CCMUA WPCF receives wastewater influent from three (3) sources, 1) the City's 72-inch Combined Sewage system main, 2) the County's 96-inch Sanitary Sewer main, and 3) the 36-inch Baldwin Run force main. These pipes merge at an existing junction structure near the northeasterly side of the plant.

The 2016 annual average flow rate of wastewater through the plant was approximately 52.6 million gallons per day (MGD), while the plant is rated to treat an average annual flow of up to 80 MGD. The existing raw sewage pumps have a firm pump capacity of 150 MGD, which is the rated wet weather capacity of the treatment plant. A related project is currently underway to increase the firm capacity (largest pump out of service) of the raw sewage pumps to approximately 180 MGD.

The actual range of flows to the plant depend on how the wastewater collection system is operated. A new Junction Structure is being designed to allow approximately 190 MGD of wastewater flow to enter into the plant. **Table 2-1** lists the peak flow from each of the identified sources.

Table 2-1: Flows to CCMUA Delaware No. 1 WPCF

Flow	Flow Rate (MGD)
96" County Interceptor	90
72" City Interceptor	80
36" Baldwin Run Force Main	20
Total Maximum Flow to Plant	**190**

With other improvements being discussed for the collection system, the maximum flows to the plant could potentially reach 240 MGD. The limitation in bringing additional flows to the plant through the collection system are being addressed by others.

2.1 Project Goals

In consultation with the CCMUA, the following goals have been identified for the overall Wet Weather Upgrades Project:

Wet Weather Improvement Goals

- Increase wet weather treatment capacity of the CCMUA's WPCF to provide sufficient capacity to treat the 10-year storm for at least a portion of Camden's combined sewer system, and to foster community redevelopment within the City of Camden by reducing flooding.
- Reduce overflows from the combined sewer system to improve water quality in the receiving waters to protect the Delaware River.
- Maximize cost effectiveness without sacrificing environmental performance.
- Minimize community impacts during construction and operation.

- Maintain Plant Operations during construction, and design new facilities to not negatively impact existing plant operations.

2.1.1 Phase 1 – Baseline Design for Wet Weather Capacity Upgrades

Phase 1 of the project includes baseline designs for wet weather capacity upgrades to the existing plant that can be implemented using existing facilities without major revisions to existing structures, and without construction of new structures or unit processes to treat up to 180 MGD of wet weather flow.

There are two primary goals to Phase 1 of the Project:

Phase 1 Goals

- Develop a Hydraulic Model for the Delaware No. 1 WPCF and determine the wet weather hydraulic capacity of the existing plant.
- Increase Delaware No. 1 WPCF hydraulic capacity to 180 MGD using the existing facilities. Flow rate is based on the use of the combined operation of three of the four raw sewage pumps, each operating at 60 MGD.

2.1.2 Phase 2 – Construction of New Facilities for Wet Weather Capacity Upgrades

Phase 2 of the project includes three alternatives for evaluation that involve the construction of new facilities or unit processes at the WPCF to allow the plant to treat up to 240 MGD of wet weather flow. The Delaware No. 1 WPCF is expected to treat up to 240 MGD when four raw sewage pumps, each with 60 MGD capacity, are operating together. The four pumps are anticipated to operate during a 25 – year storm event.

Wet Weather Capacity Upgrade Alternatives

1. Construction of a complete onsite parallel wet weather treatment facility using a combination of existing and new wet weather treatment facilities, such as:
   a. Construction of a new wet weather headworks and degritting facility.
   b. Increasing raw sewage wet weather pumping capacity.
   c. Equipping existing primary tanks with new high-rate clarification system.
   d. Construction of new wet weather outfall with a high-rate disinfection system.
2. Construction of various treatment plant upgrades designed to meet a specific design storm or flow using a combination of existing and new wet weather treatment facilities.
3. Construction of a wet weather storage facility for wet weather flow equalization.

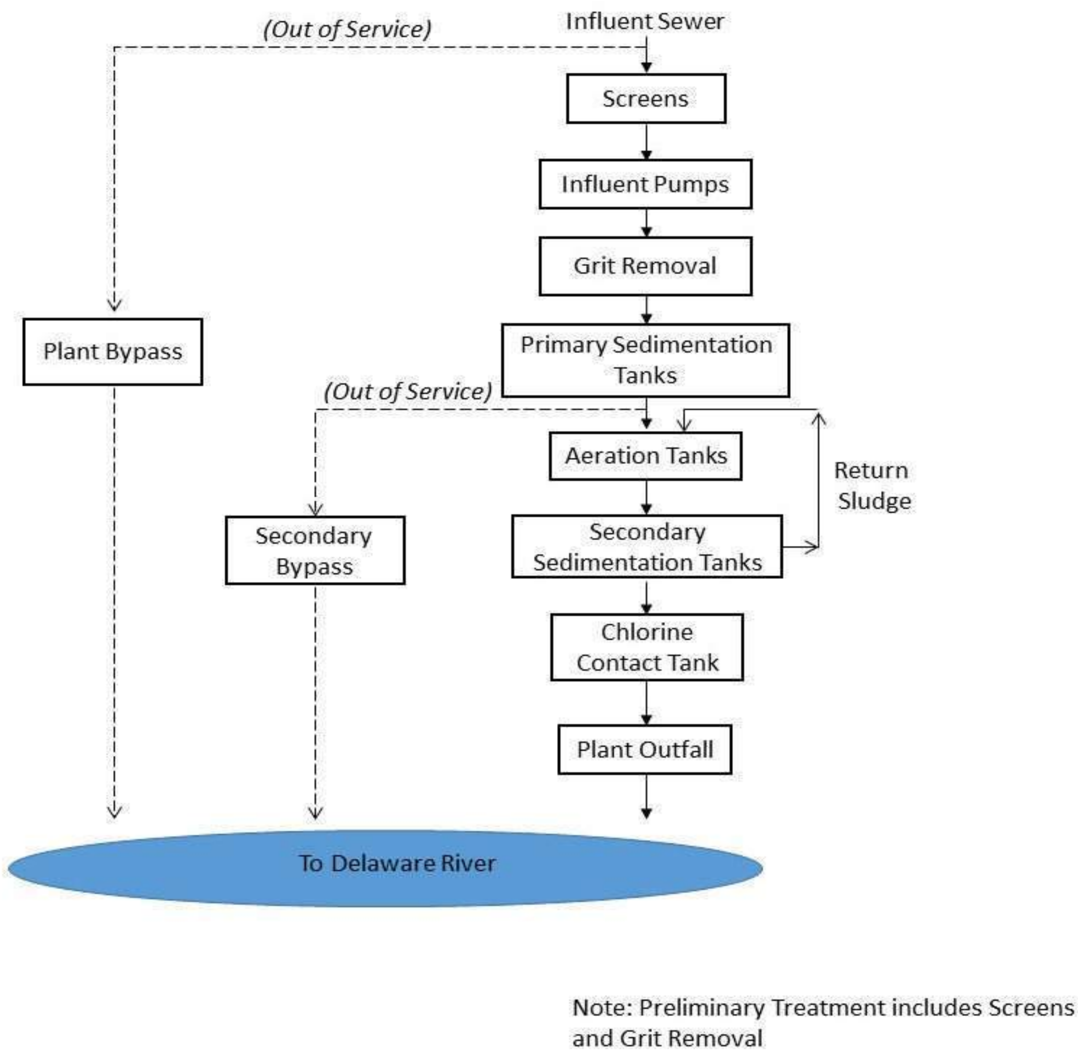
2.2 General Information

A process flow diagram for the treatment plant showing the major unit processes and flow streams is presented in **Figure 2-1**. Currently the Long-Term Control Plan (LTCP) intends to achieve an 85% capture rate for treatment of the annual Combined Sewer Overflow (CSO) volume to meet the requirements of the "Presumption Approach," versus a maximum of 4 to 6 overflow events per year. The treatment plant is rated as follows:

CCMUA WPCF Hydraulic Capacities

- Rated Wet Weather Capacity: 150 MGD
- Rated Annual Average Capacity: 80 MGD
- 2016 Annual Average Daily Flow: 52.6 MGD

Figure 2-1: Delaware No. 1 WPCF Treatment Process Flow Diagram



Section 3 Hydraulic Model

3.1 General Description of the Hydraulic Model

The hydraulic model developed for CCMUA's Delaware No. 1 WPCF was primarily based on the As-Built design record drawings for the plant. The plant consists of 103 separate hydraulic elements identified between the grit tank influent channel and the outfall discharge. Each hydraulic element represents an existing hydraulic structure including but not limited to tanks, channels, conduits, pipes or weirs. The goal for the development of the steady-state hydraulic model was to provide CCMUA with an accurate prediction of the hydraulic grade line and projected water surface elevations throughout the WPCF for a range of flow conditions and process configurations. To achieve this goal, the following steps in creating the model were performed:

- Defined model boundaries and assumptions.
- Breakdown of existing hydraulic structures into separate elements that can be represented by standard hydraulic equations.
- Identified dimensions and elevations from record drawings and site visits.
- Developed a user interface for the plant modification and expansion alternatives and output display (hydraulic profile).
- Calibrated the hydraulic model using data from field measured water surface elevations.

3.2 Bases of Hydraulic Model

The design criteria for the hydraulic model were as follows:

- Predict the hydraulic grade line (HGL) water surface elevation at various design flow conditions.
- Calibrate the model HGL to within a tolerance of (+/-) 2-inches of the field measured water surface elevations.
- Incorporate into the hydraulic model proposed modifications and plant expansion alternatives for evaluation.

The hydraulic principles that were used in the hydraulic model development are listed as follows:

- Manning's Equation for open channels and conduits.
- Weir equations – Sharp-crested rectangular weirs; "V" notched weirs; and rectangular spillway weirs.
- Villemonte's Equation for submerged weirs.
- Velocity head loss coefficients from Chicago Pump Handbook; King; ASCE Manual No. 19; Schroder and Dawson.
- Orifice, Slide Gates and Sluice Gates, Valve head loss from *Manufacturer's Data and Graphs.*

3.3 Hydraulic Analysis

The hydraulic model calculates energy and hydraulic grade line elevations upstream and downstream of hydraulic control points in the treatment plant such as weirs, tanks, channels and conduits. Hydraulic model development began with the Delaware River water surface elevation, which represents the farthest downstream hydraulic elevation. Model development proceeded upstream from the river surface elevation, one element at a time.

The Delaware River's maximum water surface elevation is 10.0-feet to which 1.5-feet was added to account for predicted Climate Change effects through 2050. This brings the maximum river water surface elevation to 11.5-feet. River water surface elevations at the time of the field measurements were obtained from the National Oceanic and Atmospheric Administration (NOAA) website. The Tidal Station No. 8545240 Philadelphia, PA is located across the Delaware River from the plant. The predicted Mean Higher High Water (MHHW) elevation through 2050 of 11.5-feet was used as a reference point in the hydraulic model. See **Appendix B** for NOAA tidal station data. **Table 3.1** lists the various water levels evaluated for the Delaware River.

Table 3-1: Delaware River Water Levels

Water Levels	Elevation (feet)
Mean Higher High Water (MHHW)	10.0
Mean Higher High Water + 18" for Future Climate Change Through 2050	11.5
Mean High Water (MHW)	4.0
Mean High Water + 18" for Future Climate Change Through 2050	5.5
Mean Low Water (MLW)	-2.0

The governing criterion for the hydraulic model is to contain and treat the peak wet weather flow, for the Phase I alternatives assuming all treatment process units are in service, while maintaining a minimum freeboard of 6-inches throughout the plant. For the purpose of this project, peak flow is described as the maximum wet weather discharge to the Delaware River at the current MHHW river elevation of 10.0 feet plus the 1.5 foot additional allowance for the Climate Change effects. Model inputs can be modified to include different plant flow rates, number of process units in service, tide elevations, and additional parameters.

3.4 Hydraulic Model Assumptions

The hydraulic model was prepared with the following assumptions:

- All facilities are in service during the peak wet weather flow scenarios
- Mean Higher High Water (MHHW) level elevation is 11.5-feet, this incorporates the 1.5 feet for predicated Climate Change effects by 2050.
- Equal flow spilt between the north and south sides of the plant
- Return activated sludge (RAS) flow rates are included in the flow to Aeration Tanks and removed prior to Final Sedimentation effluent weir
- The RAS flow rates were assumed to be 30% (per CCMUA website) of the inflow rates.
- All gates are fully open.
- Flows at steady state.
- Flow is laminar and surfaces are quiescent.
- Effect of contractions from the sides of weirs are negligible.
- All elevations are based on the Plant Datum.
- No hydraulic effect on aerated surfaces.

3.5 Model Calibration

The hydraulic model was calibrated by comparing the model predictions to water surface measurements taken in the field. Water surface elevations were taken on December 5, 2016 between 3:32 pm and 4:38 pm. The flow rate at the plant during the time of the field measurements was 57 MGD. The NOAA tidal information was obtained during field measurements and shows the average river water level was at an elevation of approximately 1.6-feet at the time the field measurements were taken. The calibrated hydraulic model includes the HGL, Weir, Channel and Tank elevations. A summary of the plant hydraulics is included in **Table 3-2**. The yellow cells in the table show locations where the hydraulic model predicted the flows to either overflow or submerges weirs. There were three areas of significant difference between the hydraulic model and the observed field measurements. The South Old Effluent Channel had over a 1.0-foot difference between the hydraulic model and the field measurements. In our model, it is assumed that flow is split equally between the North and South treatment trains. This difference could be attributed to an obstruction within the South Effluent Channel. The North and South Splitter Box Troughs each had approximately a 0.5 foot difference between the hydraulic model and the field measurements. The 0.5 foot difference could be attributed to debris build-up within each trough. Both Chlorine Contact Tank Effluents had over a 5.0-foot difference between the hydraulic model and the observed field measurements. At the time, the hydraulic drop over the effluent weir was approximately 10-feet resulting in a turbulent water surface making it difficult to obtain accurate water surface elevations at that location.

Table 3-2: Hydraulic Model Calibration Table

CCMUA

Model/Field Measurement Comparison

Collected 12/5/16 between 3:32p-4:38p

ID #	Location	Top of Tank Elev. (ft)		WSEL Model Prediction @ 57 mgd (ft)	Weir Elev. (ft)	WSEL Field Measurement @ 57 mgd (ft)	Difference (ft)
	Grit tanks						
1	Influent Channel	34.08		26.96		26.96	0.00
2A	Grit Tank Surface	34.08		26.86	26.10	26.96	0.10
2	Effluent Channel	34.08		25.34		25.54	0.20
	Primary Tanks						
3	West Influent Channel	26.65		25.23		25.09	-0.14
3A	East Influent Channel	26.65		25.22		25.05	-0.17
4	Influent Channel Near Tank #10	26.65		24.97	24.58/ 24.62	24.98	0.01
5A	PST Tank #10 Water Surface	26.65		24.67		24.69	0.02
5	South Old Effluent Channel	26.65		23.18		24.49	1.31
6A	PST Tank #1 Water Surface	26.65		24.67		24.69	0.02
6	North Old Effluent Channel	26.65		23.18		23.44	0.26
16	North New Effluent Channel	26.65		22.61		22.56	-0.05
17	South New Effluent Channel	26.65		22.61		22.55	-0.06
	Aeration Tanks						
7	North Splitter Box Surface	26.00		22.57	22.25	22.57	0.00
7A	North Splitter Box Trough	26.00		19.64		20.07	0.43
8	South Splitter Box Surface	26.00		22.57		22.54	-0.03
8A	South Splitter Box Trough	26.00		19.64		20.17	0.53
	Aeration Tanks	26.00		18.73	18.50		
10A	North Effluent Channel	20.00		17.65		18.04	0.39
9A	South Effluent Channel	20.00		17.65		-	-
	Final Tank Influent Channel						
10	North	20.00		17.61		17.68	0.07
9	South	20.00		17.61		17.82	0.21
	Final Sedimentation Tanks						
11	North Effluent Trough	20.00		15.96		15.69	-0.27
11A	North FST Water Surface	20.00		17.60	17.50	17.54	-0.06
11B	North Effluent Channel	20.00		15.80		15.91	0.11
12	South Effluent Trough	20.00		15.96		15.58	-0.38
12A	South FST Water Surface	20.00		17.60	17.50	17.50	-0.10
12B	South Effluent Channel	20.00		15.80		15.81	0.01
	Chlorine Contact Tanks						
13	North Contact Tank Surface	20.00		15.75	14.84	15.67	-0.08
14	North CT Effluent Below Weir	20.00		2.58		7.60	5.02
15	South Contact Tank Surface	20.00		15.75	14.84	15.52	-0.23
14A	South CT Effluent Below Weir	20.00		2.58		8.38	5.80

Section 4 Flow Management Strategies

4.1 Existing Hydraulic Bottlenecks

Analysis of the model results for the HGL between each hydraulic element at different flow rates and river elevations allowed for the identification of hydraulic bottlenecks within the plant. Bottlenecks were identified at the locations where the water surface either encroached into the 6-inch freeboard or submerged weirs. From the plant hydraulic analysis the following hydraulic bottlenecks were identified.

Identified Hydraulic Bottlenecks

At Primary Sedimentation Tanks

- Constriction where Grit Tank Effluent Channel makes a 90-degree turn and merges into Primary Sedimentation Tank (PST) East and West Influent Channels.
- Inlet Ports into each of the PSTs.

At Outfall

- Limited capacity of dual 60” Diameter Outfall Pipes.
- Three 90^{o} Elbows between Outfall Well Chamber and Chlorine Contact Tank (CCT) Weir Chamber.
- 45^{o} Elbow at Outfall Discharge.
- 72” x 54” Diameter Concentric Reducer Nozzle at the Outfall Discharge.

4.2 Potential Concerns and Improvements

The identified bottlenecks outlined above cause overflows within the treatment plant process train under certain conditions. Hydraulic elevations were calculated using the mean high water (MHW) river elevation of 4.0-feet, which is the average high river elevation. The identified bottleneck locations are summarized below:

- Raw Sewage Pump Discharge Pipes become submerged and the Raw Sewage Pump Effluent Discharge Channel overflows at plant flows above 180 MGD.
- PST Influent Channels overflow at plant flows above 180 MGD.
- Treatment processes between the Aeration Tanks (ATs) and CCTs overflow at plant flows above 220 MGD.

At a MHW river elevation of 5.5-feet (average high water elevation of 4.0 feet with allowance for the anticipated 1.5 foot tide increases Climate Change by 2050):

- Raw Sewage Pump Discharge Pipes become submerged and the Raw Sewage Pump Effluent Discharge Channel overflows at plant flows above 180 MGD.
- PST Influent Channels overflow at plant flows above 180 MGD.
- PSTs and the PST Effluent Channels overflow at 220 MGD.
- Treatment processes between the ATs and CCTs overflow at plant flows above 220 MGD.
- Outfall Well Chamber overflows at plant flows above 180 MGD.

At a mean higher high water (MHHW) river elevation of 10.0-feet (maximum storm tide elevation):

- Raw Sewage Pump Discharge Pipes become submerged and the Raw Sewage Pump Effluent Discharge Channel overflows at plant flows above 180 MGD.
- PST Influent Channels overflow at plant flows above 180 MGD.
- PSTs and the PST Effluent Channels overflow at 220 MGD.
- Treatment processes between the ATs and CCTs overflow at plant flows above 220 MGD.
- Outfall Well Chamber overflows at plant flows above 180 MGD.

At a MHHW river elevation of 11.5-feet (maximum storm tide elevation with allowance for the anticipated 1.5 foot tide increases Climate Change by 2050):

- Raw Sewage Pump Discharge Pipes become submerged and the Raw Sewage Pump Effluent Discharge Channel overflows at plant flows above 180 MGD.
- Grit Tank Effluent Channel overflows at flows above 220 MGD.
- PST Influent Channels overflow at plant flows above 180 MGD.
- PSTs and the PST Effluent Channels overflow at 220 MGD.
- Treatment processes between the ATs and CCTs overflow at plant flows above 150 MGD.
- Outfall Well Chamber overflows at plant flows above 150 MGD.

Output tables for each of the modeled river elevations at varying flow rates and tide levels are found in **Appendix C**.

4.3 Wet Weather Scenarios

The Delaware No. 1 WPCF currently has a maximum wet weather hydraulic capacity of 150 MGD. The project goal of the Phase 1 wet weather improvements investigations is to increase the Delaware WPCF No. 1 wet weather capacity to treat up to 180 MGD. The Phase 2 project goal is to increase the wet weather capacity up to 240 MGD.

As noted in Section 2, with improvements to the junction chamber at the head of the plant, flows into the plant could be increased up to 190 MGD. Currently, Greeley and Hansen is upgrading the raw sewage pumping facility to increase plant's firm pumping capacity from 150 to 180 MGD. With other improvements being discussed for the collection system, the maximum flows to the plant could potentially reach 240 MGD based on the probable maximum capacity of the existing sewer infrastructure with modifications to increase the delivery of wastewater to the plant.

As noted in sections above, the Phase 2 alternatives to achieve additional wet weather capacity through the plant include four alternatives that were evaluated in two (2) phases.

Phase 1 Alternative

Phase 1 of the project involves the following alternative:

1. Increase wet weather capacity at CCMUA's Delaware No. 1 WPCF through upgrades to the existing plant systems without foundation revisions to existing structures and without construction of new structures or unit processes. This work focuses on removal of hydraulic bottlenecks identified in the hydraulic model. Only Alternative No. 1 is addressed in Phase 1 of the Concept Study.

Phase 2 Alternatives

Phase 2 of the project addresses Alternatives No. 2, 3, 4 and 5 below.

2. Wet Weather Storage Tank: Construct underground storage tanks to serve as a flow equalization facility *(on plant site)*, with the capacity to store flows in excess of 150 MGD until the wet weather event passes and the plant has the capacity to process the stored raw wastewater.
3. All Flow Treatment: Full treatment of wet weather flows including primary and secondary treatment, as well as disinfection.
4. Wet Weather Plant Bypass: Up to 55 MGD bypasses the existing primary and secondary treatment facilities after preliminary treatment and is discharged into the Delaware River through a new wet weather outfall. This option can be implemented in various forms or steps as follows:
   - Standard-rate or high-rate disinfection with no preliminary treatment or raw sewage pumping upgrades.
   - Screening of raw wastewater to 0.50 inch followed by standard-rate or high-rate disinfection with no grit removal system or raw sewage pumping upgrades.
   - Screening of raw wastewater to 0.50 inch followed by high-rate primary treatment in area north of existing aeration/final tanks, and standard-rate or high-rate disinfection with no grit removal system or raw sewage pumping upgrades.
   - Screening of raw wastewater to 0.50 inch followed by high-rate primary treatment in area north of existing aeration/final tanks, standard-rate or high-rate disinfection, and chlorine contact tank with no grit removal system or raw sewage pumping upgrades.
   - Preliminary treatment and raw sewage pumping upgrades followed by high-rate primary treatment in area north of existing aeration/final tanks, standard-rate or high-rate disinfection, and chorine contact tank to provide a firm capacity of 240 MGD.

5. Secondary Bypass – Uses enhanced primary treatment system constructed within the existing Primary Sedimentation Tanks:
   - Preliminary treatment and raw sewage pumping upgrades followed by up to 92 MGD of high-rate primary treatment constructed within existing Primary Sedimentation Tanks followed by standard-rate or high-rate disinfection, and chorine contact tank to provide a firm capacity of 240 MGD.
   - Preliminary treatment and raw sewage pumping upgrades followed by Chemically Enhanced Primary Treatment in existing Primary Sedimentation Tanks followed by standard-rate or high-rate disinfection, and chorine contact tank to provide a firm capacity of 240 MGD.

4.4 Hydraulic Profiles of Evaluated Scenarios

The hydraulic grade profiles from the hydraulic model with and without the proposed baseline design improvements evaluated for the Phase 1 alternatives are found in **Appendix D**.

Section 5 Existing Wet Weather Process Capacity

5.1 Existing Plant Capacity

In 2016, the Delaware No. 1 WPCF treated an annual average flow rate of approximately 52.6 MGD and the plant is rated to treat 80 MGD. The four (4) existing raw sewage pumps together can provide a firm capacity (largest pump out of service) of 150 MGD, which is the maximum wet weather capacity treatment at the plant. As mentioned previously, a related project is currently underway to increase the firm pumping capacity at the Raw Sewage Pumps to approximately 180 MGD. The basic treatment plant processes include preliminary treatment, primary sedimentation, aeration, final sedimentation, and disinfection.

The Preliminary Treatment Facility contains three (3) Detritus Grit Tanks with a combined tank area of 4,357 ft^2. Once the wastewater flows through the grit tanks, flow enters an effluent channel then to the primary sedimentation facility. The Primary Sedimentation Facility includes 10 PSTs with a combined surface area of 93,000 ft^2. The PSTs receive grit tank effluent flows through the East and West PST influent channels, of which, the latter has a lesser hydraulic capacity as predicted by the hydraulic model. Specifically, the plant staff reports that the flow split is approximately 75% to the East Channel and 25% to the West Channel.

Flow exits the PSTs into an effluent channel that directs flow into one of two (2) flow splitter chambers that divides flow into the north and south aeration systems. The aeration facility includes eight (8) aeration tanks with a combined tank volume of 14.79 million gallons. Flow then exits the aeration tanks into effluent channels which direct flow to the Final Sedimentation Tanks (FSTs). The final sedimentation facilities include eight (8) tanks with a combined tank surface area of 158,656 ft^2.

Flow from the Final Sedimentation Tanks (FSTs) enters into effluent channels that direct the flow into the chlorine contact tanks. The disinfection facilities include two (2) chlorine contact tanks with a combined volume of 2.76 million gallons. Outfall from the CCTs is discharged into the Delaware River through a series of outfall pipes ranging from 60-inches to 72-inches in diameter.

5.2 Wet Weather Issues

During extreme wet weather events, the maximum storm tide elevation for the Delaware River based on the plant datum is currently expected to be 10.0-feet and at 11.5-feet by the year 2050 due to the predicted effects of Climate Change. All CCMUA Delaware No.1 WPCF treatment processes were evaluated at the tidal elevation of 11.5-feet and the results are included in **Appendix E**.

Particle size removal efficiencies were evaluated at the grit removal and handling facility for varying flow rates at the predicted future (2050) maximum storm tide elevation of 11.5 feet. Particle size removal ranges from 100+ (mesh) at 57 mgd to 65-80 (mesh) at 180 mgd, which indicates small particle carryover during the high flow events. Typically, Grit Tanks are designed for particle size removal in the range of 100-150 mesh. The frequency of high flow events is anticipated to be low, and further, are not expected to last for prolonged periods of time. Thus, the decrease in mesh removal efficiency at higher flow rates should not have a significant impact on the overall treatment performance of the WPCF.

Recommended surface overflow rates (SORs) for the Primary Sedimentation Tanks should not exceed 2,000 gallons per day/square foot in accordance with the recommendations of the 10 State Standards. These overflow rates were evaluated for the PSTs at varying flow rates and the predicted future maximum storm tide elevation. The 10 State Standards Surface overflow rates, a key parameter in sedimentation performance, are met at flow rates up to 180 mgd.

Similarly, recommended detention times were evaluated for the Aeration Tanks at varying flow rates at the predicted future maximum storm tide elevation. The recommended detention time of 1.5 hours from the 10 States Standards is met at flow rates up to 180 mgd. The detention time, however, is not met at 240 mgd**.**

Surface overflow rates for the Final Sedimentation Tanks should not exceed 1,200 gallons per day/square foot as recommended by the 10 State Standards. Overflow rates were evaluated for the Final Sedimentation Tanks at varying flow rates at the predicted future maximum storm tide elevation. The recommended SORs are met at flow rates up to 180 mgd, but are exceeded at flow rates above 180 mgd.

Detention times were evaluated for the disinfection facilities at varying flow rates at the predicted future maximum storm tide elevation of 11.5 feet. The recommended minimum detention time of 15 minutes, as recommended by the 10 State Standards, will not be met at 240 mgd.

Section 6 Phase 1 Capacity Improvements

6.1 Existing Hydraulic Bottlenecks

Addressing the hydraulic bottlenecks can correct the existing overflow issues and increase the plant's capacity to treat flows up to 180 MGD during significant wet weather events with river storm tide elevations of both 10.0-feet and 11.5-feet. The following are potential Phase 1 plant improvements that would produce the most significant impact in increasing the overall hydraulic capacity of the plant:

Plant Capacity Improvements Alternatives

At the Outfall:

- Remove the 45-degree elbow and the 54-inch by 72-inch dispersion nozzle at the end of the 72-inch outfall pipe; however, this option is not recommended based on the conclusion of the Plant Effluent Dispersion Report (a.k.a. The Dilution Study).
- Realign and reconstruct the dual 60-inch outfall pipes with dual 72-inch pipes. Include removal of three 90-degree elbows (two at the CCT outfall and one at the wet well chamber). This option is not recommended based on the high cost and complexity of constructing the outfall system without elbows.
- Construct an emergency overflow system at the Outfall Well Chamber.
- Use the existing dual 36-inch blind flange outfall access points on top of each of the dual 60-inch outfall pipes with vertical risers for hydraulic relief.

At the Primary Sedimentation Tanks:

- Increase PST inlet port size width from 15-inches to 24-inches.
- Provide two additional inlet ports into each PST.
- Increase PST Channel Wall Height for East and West Influent Channels by 12-inches.
- Decrease PST Effluent Weir Height by 6-inches.

6.2 Proposed Mitigation at Outfall

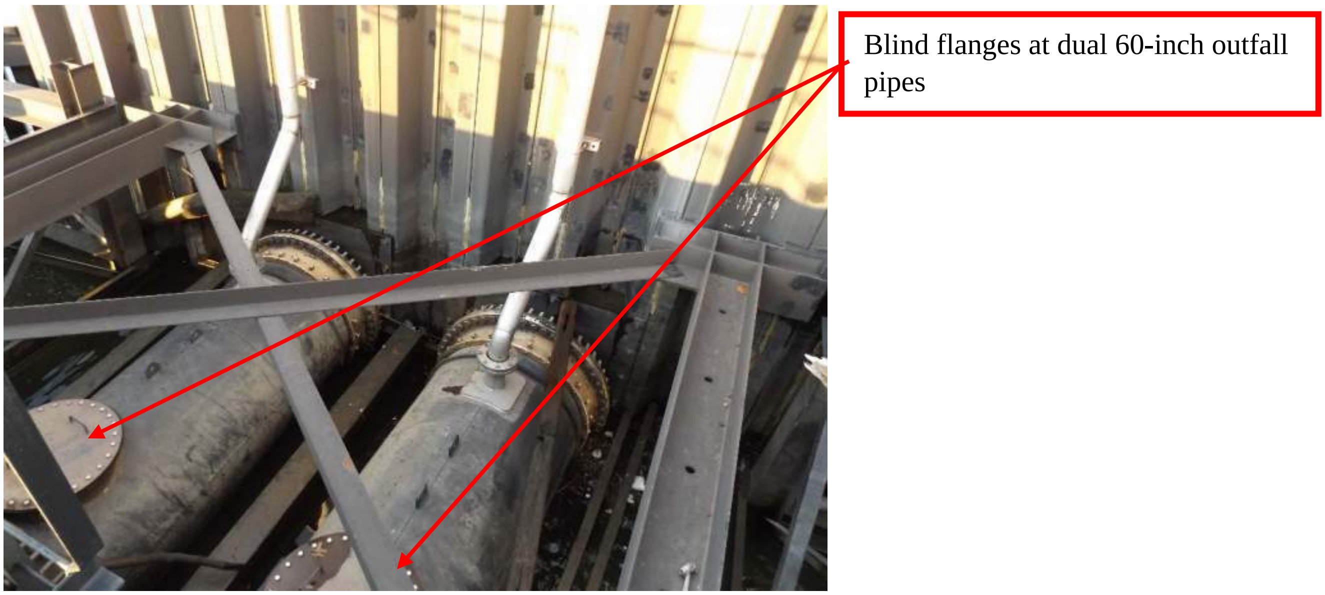
Two viable alternatives for addressing existing bottlenecks at the outfall are:

- Construct an outfall hydraulic relief weir system at the Outfall Well Chamber.
- Use the existing dual 36-inch blind flanged connections on top of each of the 60-inch outfall pipes with vertical riser as an outfall hydraulic relief system.

The first viable alternative consists of modifying the existing outfall to include an Outfall Relief Structure that would relieve flows when the total plant flow exceeds 150 MGD, and would prevent upstream overflows. This modification would consist of opening the west and south walls of the existing Outfall Well Chamber structure and installing two weir structures. The weir elevations would be placed at least 1.5-feet above the MHHW elevation of 10-feet, to prevent backflow into the chamber, to be below the top of weir elevation of the CCTs, and to account for the 1.5 feet predicted rise in river tide elevations due to Climate Change by 2050.

The second viable alternative is to modify the dual 36-inch blind flanged connections on each of the outfall pipes as shown in **Figure 6-1**. This modification would consist of either installing a vertical riser or horizontal flap gate system that would relieve flows to the river when the plant flow exceeds 75 MGD.

Figure 6-1: Existing Blind Flanges on Outfall Pipes



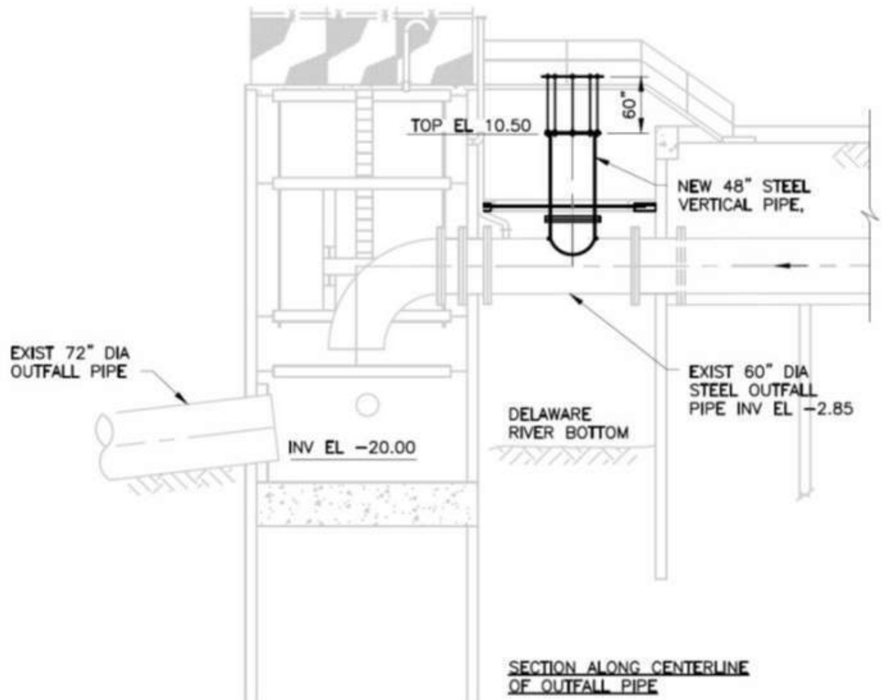
6.2.1 Emergency Overflow System

An emergency overflow system at the existing Outfall Well Chamber was considered to relieve plant flows exceeding 150 MGD. Upon further evaluation of this option, it was determined as unfeasible due to the significant costs required to retrofit the existing Outfall Well Chamber with such a system. The retrofit would have required removal of the existing top deck structure, significant structural modifications to the Outfall Well Chamber wall systems, and additional structural supports for the new roof deck system.

6.2.2 Vertical Riser System

Headloss from the Vertical Riser System is relatively straight forward. The effluent rises in the vertical pipe section as the plant flow hydraulic grade line and river elevation rise. When the Hydraulic Grade Line of the outfall pipes, governed by the river tide elevation and plant flow rates, exceeds the top of the riser pipe, flow discharges from the riser pipe at a rate that is proportional to flow over a weir of a length equivalent to the circumference of each riser pipe. The top of the riser pipe is set at 10.50 – feet, which is 0.5 – feet above the current maximum storm tide elevation of 10 – feet. Additional segments can be added through flanged spool pipe segments to increase the height of the riser pipe as necessary to account for increasing maximum storm tide elevations due to climate effects. **Figure 6-2** illustrates that vertical riser system concept.

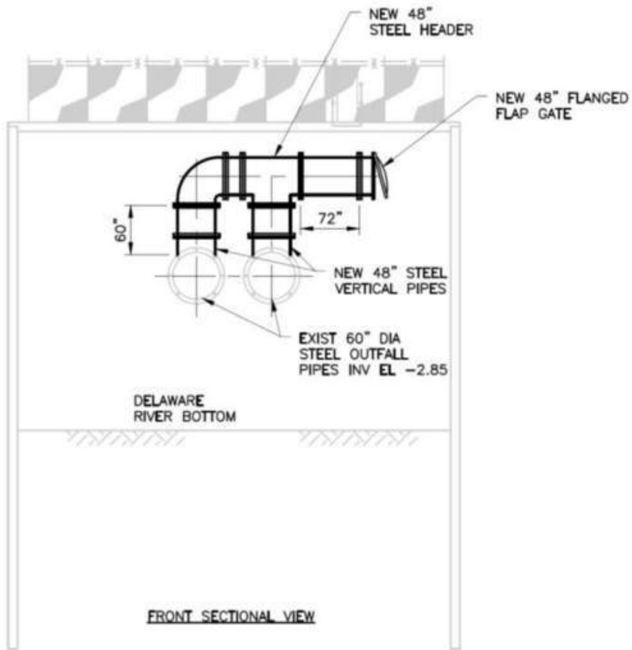
Figure 6-2: Vertical Riser System Concept



6.2.3 Flap Gate System

Headloss of the Flap Gate System is a major factor in the performance of the proposed relief system. The flap gate system concept is illustrated in **Figure 6-3**. Head loss studies of flap gates have been performed by a number of researchers. Comparison of the studies demonstrate consistent relationships between headloss and flow rate of varying sizes of flap gates. The relationship demonstrates a rapidly increasing headloss, up to a peak headloss at a critical flow rate, followed by a gradually decreasing headloss as flow rates exceed the critical flow. The critical flow and peak headlosses are a function of the flap gate size.

Figure 6-3: Flap Gate System Concept



A 2003 American Society of Agricultural Engineers (ASAE) study formulated the following mathematical relationship for flap gates:

Flap Gate Headloss Equation

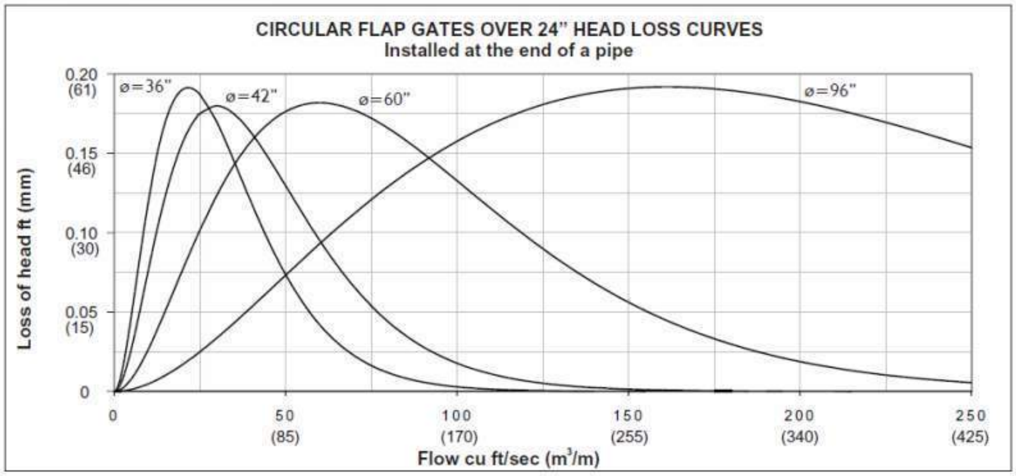
$$\frac{H_L}{D} = \frac{\frac{1}{176} g D^5}{Q^2}$$

Where, H_L is headloss in feet,
D is gate diameter in feet,
g is the gravitational constant,
Q is the flow rate in ft^3/s.

(https://naldc.nal.usda.gov/download/54857/PDF)

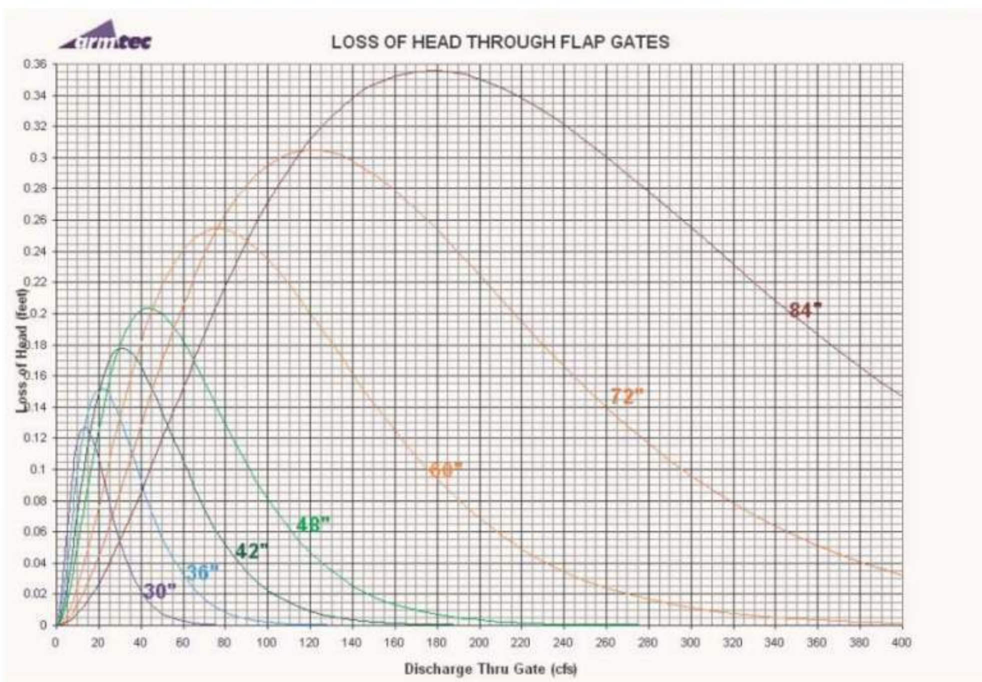
Neenah Foundry, a flap gate manufacturer, estimates headloss to be equal to half of the diameter of the flap gate opening, and states that “maximum headloss occurs when the pipe is flowing full.” (http://www.nfco.com/municipal/products/valves-gates/flap-gates). This information is in contrast to other cited studies. The Neenah Foundry approach appears to relate headloss empirically to gate size, whereas the cited studies determines headlosses through experimental results. Flap gate headloss as a function of flow rate and gate size are demonstrated in **Figures 6-4 and 6-5**. The hydraulic model for the CCMUA Delaware No. 1 WPCF outfall utilized the ASAE flap gate headloss approach because of the confidence provided by experimental validation of results, and confirmation by two independent studies.

Figure 6-4: Fontaine Headloss through Circular Flap Gates



http://www.hfontaine.com.br/images/PDF/flap_serie_60.pdf

Figure 6-5: Armtec Headloss through Flap Gates



[https://www.armtec.com/wp-content/uploads/Water-Control-Products-flap-gate-head- loss-charts.pdf](https://www.armtec.com/wp-content/uploads/Water-Control-Products-flap-gate-head-loss-charts.pdf)

6.2.4 Outfall Systems Evaluation

Performance complexities of a new vertical riser system and a new flap gate system were compared to evaluate advantages and disadvantages for the outfall relief design. The advantages and disadvantages of each system are summarized below.

	Vertical Riser System	Flap Gate System
System Advantages	Reliable Operation	Protected from Debris
	Free Discharge to River	Protected from Backflow
	Minimal operation parameters	
	Predictable headloss	
	Minimal Associated Headlosses	
	Immediate Discharge of Excess Flows	
	Lower Associated Construction Costs	
System Disadvantages	Becomes submerged at maximum storm tide elevations	Reliability degrades with time (i.e. corrosion, malfunction of gate mechanism)
	Minimal operation parameters	Uncontrollable variables such as degree of gate submergences leads to unpredictable hydraulic behavior
		Higher associated minor headlosses with design
		Increase in system head is required to activate flap gate and begin relief.
		Higher Associated Construction Costs

6.2.5 Recommendation for Outfall Improvements

The goal of the outfall modifications is to reduce the overall water elevation between the Outfall Well Chamber and the PST influent channels. By reducing the water elevation, potential overflow conditions at the Outfall Well Chamber, CCT and CCT effluent channels, FST and FST influent and effluent channels, the AT effluent channels, and the PSTs and PST influent channels are mitigated or eliminated. Of the three viable outfall alternatives evaluated, the third alternative to modify the dual 36-inch blind flange connections is preferred and recommended as a wet weather flow relief option for the CCMUA Delaware No. 1 WPCF outfall. This is due to the significantly higher costs associated with the modifications of the Outfall Well Chamber discussed in Section 6.2.1, compared to the cost for modifications to the blind flange connections. Furthermore, the vertical riser system has less unforeseen construction costs and potential failures and adverse hydraulic effects versus the flap gate system.

6.3 Proposed Mitigation at Primary Sedimentation Tanks

Improving the hydraulic capacity of the PSTs could be achieved by any or some grouping of the following:

- Increase all PST Influent Port widths from 15-inches to 24-inches.
- Provide two (2) additional influent ports into each tank.
- Increase the PST Channel Wall Height for the East and West Influent Channels by 12-inches. However, this option interferes with existing railings, conduits, actuators, access points and other existing features that complicates the work.
- Decrease the PST Effluent Weir Height by 6-inches.

Enlarging the width of the existing inlet ports from 15-inch to 24-inch, or adding two new inlet ports, could reduce the upstream water levels by 6-inches at flow rates up to 180 mgd, which would reduce the PST influent channels overflow potential at higher flow rates during wet weather events. Adding new influent ports into each Primary Sedimentation Tank for operation above plant flows of 150 mgd is the most cost-effective method to relieve upstream hydraulic pressures and allow passage of flow up to 180 mgd into the PSTs without disturbing the existing infrastructure.

Increasing the PST Channel Wall Height for the East and West Influent Channels is not recommended as this option has conflicts with the existing infrastructure. Additionally, this option is more structurally complicated as the additional weight of adding height to the wall would need to be evaluated with the existing foundation system. Decreasing the PST Effluent Weir Height was not recommended, as such a reduction in height would likely cause the weirs to become flooded at flow rates of approximately 150 mgd, resulting in a loss of hydraulic control over the PSTs.

6.4 Summary of Design Improvements

Of the proposed mitigation measures reviewed, the following are recommended for implementation:

- Provide two additional PST Inlets into each PST.
- Use the dual 36-inch blind flanged connections on top of the existing 60-inch outfall pipes with vertical risers as an Outfall Hydraulic Relief Overflow location.

6.5 Construction Cost Estimates for Phase 1 Improvements

An Opinion of the Probable Construction Costs was prepared for the recommended Phase 1 improvements. The construction cost estimates are conceptual level estimates intended to provide an estimate of the project's overall cost. Detailed construction cost estimates for the selected improvements will be prepared as part of the detailed design phase. An Opinion of Probable Construction Costs for the recommended Phase 1 improvements is provided in **Appendix F**.

Section 7 Process Review of Existing Facilities

Before evaluating the Phase 2 Options to expand the existing wastewater treatment capacity up to 240 mgd, the existing treatment facilities in the WPCF were reviewed to understand the current treatment capacities in order to provide efficient and cost-effective alternatives.

7.1 Existing Combined Sewer Systems Overview

The current rated wet weather capacity of the WPCF is 150 mgd. As part of another CCMUA project, the existing sewer system is being investigated to identify and remove hydraulic bottlenecks to increase hydraulic capacity up to 190 mgd. The hydraulic capacity of the sewer collection system would need further improvements to enable the system to convey wet weather flows up to 240 mgd.

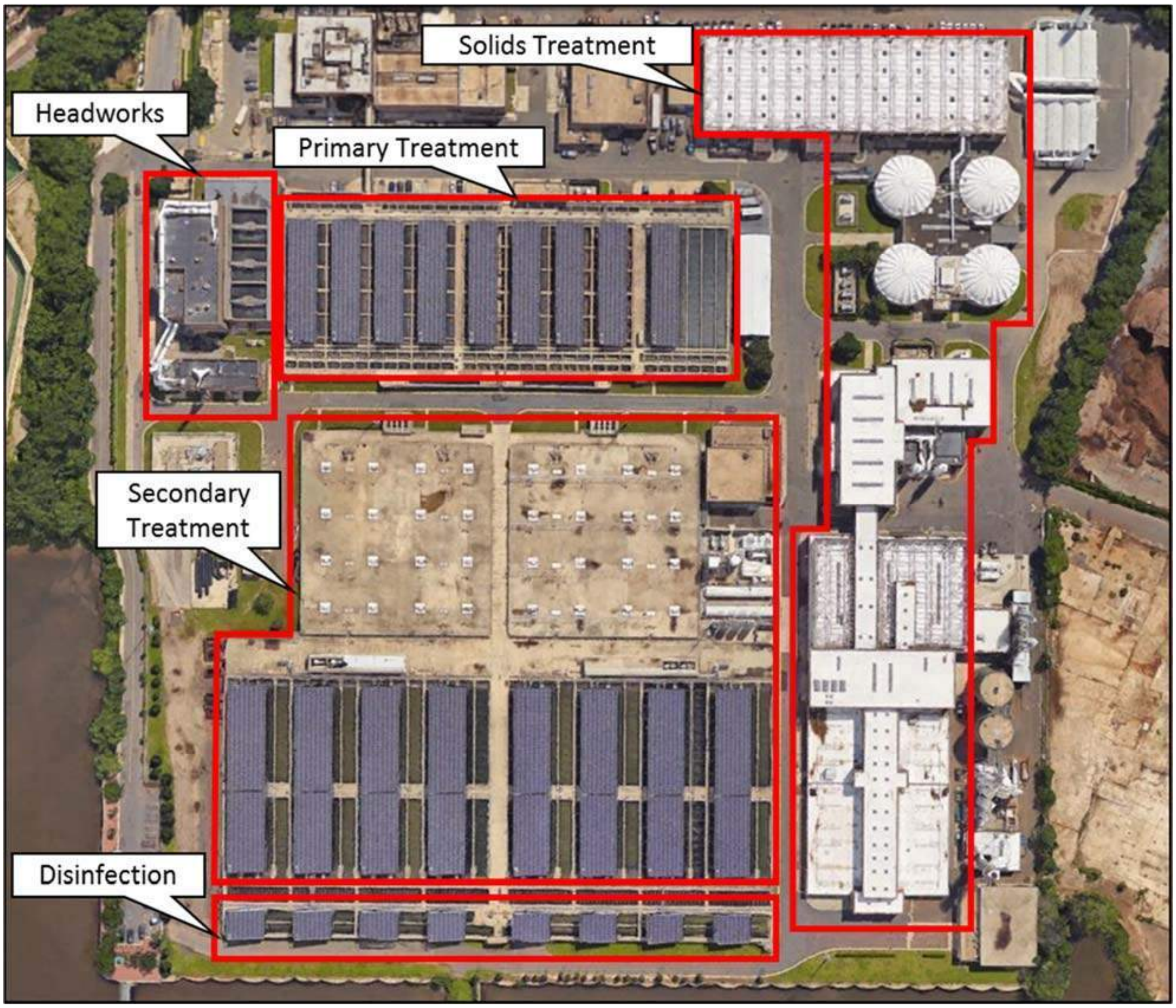
7.2 Overview of Existing Wastewater Treatment Plant Systems

The Delaware No.1 Water Pollution Control Facility was designed with an annual average treatment capacity of 80 mgd. The rated wet weather hydraulic capacity of the plant is 150 mgd. The WPCF is comprised of the following principal unit processes.

- Headworks (Preliminary Treatment)
  - Raw wastewater pumps, screens and grit removal system
- Primary Treatment
- Secondary Treatment
  - Activated sludge process (oxygen activated sludge process with mechanical mixers)
  - Final Sedimentation Tanks
- Effluent Chlorination (with sodium hypochlorite)
- Solids Treatment
  - Primary sludge storage tanks
  - Waste activated sludge storage tanks
  - Strain presses
  - Gravity belt thickeners
  - Thicken sludge feed tank
  - Screw thickeners
  - Anaerobic digester tanks
  - Belt filter presses
  - Digested sludge holding tank
  - Cake dryer

The WPCF facility plan and the areas of the major unit processes is shown on **Figure 7-1**.

Figure 7-1: WPCF Facility Plan



7.3 WWTP Inflow Characteristics

It is important to understand the characteristics of the wastewater influent at the Delaware No.1 WPCF so that the recommended wet weather upgrades will effectively and efficiently treat the wet weather flow and minimize the release of BOD, TSS, and bacterial load into the Delaware River. Currently, the plant has no discharge requirements which limits the release of nutrients like nitrogen and phosphorous into the Delaware River. Therefore the main goal of the treatment plant is to reduce BOD, TSS, and bacterial load per the requirements of the plant's current NPDES permit.

7.4 Hydraulic Evaluation and Capacity

A detailed evaluation of the hydraulic capacity was performed as stated in previous sections of this Report. According to the evaluation, the preliminary, primary, and secondary treatment systems are hydraulically capable of handling flows up to 180 mgd with minor modifications. However, in order to provide treatment capacities for flows up to 240 mgd, the hydraulic capacity of various channels, pipes, and conduits will need to be expanded.

7.5 Bar Screens and Raw Wastewater Pumps

There are three (3) Mechanical Bar Screens each rated for 60 mgd at average conditions with an upper limit of 140 mgd each. Without additions or modifications the Mechanical Bar Screens can handle 240 mgd with two screens in service.

The raw wastewater pumping system is being upgraded to deliver 240 mgd with all four pump units in service, with a firm capacity of 180 mgd (3 raw sewage pumps in service).

7.6 Grit Removal

The existing grit removal system contains three (3) Detritus Grit Removal Tanks with a combined tank surface area of 4,357 ft^2, which translates into grit tanks with a rated capacity to capture 95 percent of 100 mesh grit and coarser grit material at 70 mgd. At wet weather flows of 240 mgd the grit tanks performance would be degraded slightly, but would perform within their hydraulic capacity limits.

According to the WEF Manual of Practice No. 8 for average flow conditions, the recommended SOR is 32,300 gpd/ft^2 for 95% removal of 100 mesh particles in a Detritus Grit Removal Tank. A safety factor of “2” is recommended for calculating the design SOR in order to take into account inlet and outlet turbulence, short circuiting and hydraulic inefficiency. Calculated SORs for different Detritus Grit Removal Tank influent flowrates are shown in **Table 7-1**.

Table 7-1: Surface Overflow Rate of Existing Grit Removal System

Item Description	Existing Facilities			
Wastewater Flow, mgd	57	150	185	240
Number of Tanks	3	3	3	3
Total Operating Tank Area, ft^2	4,357	4,357	4,357	4,357
Theoretical Surface Overflow Rate Required, (SOR), gpd/ft^2	32,300	32,300	32,300	32,300
Surface Overflow Rate, (SOR), gpm/ft^2	13,084	34,430	42,464	55,089

7.7 Primary Sedimentation

Wastewater flows from the Detritus Grit Tanks to the Primary Sedimentation Tanks. The Primary Sedimentation Tanks include ten (10) PSTs with a combined surface area of 93,000 ft^2. The PSTs receive Grit Removal Tank effluent through the East and West PST influent channels. Flow exits the PSTs into an effluent channel that directs the flow into one of two (2) flow splitter chambers which divides the flow for entry into the North and South Aeration Systems.

The surface overflow rate of existing Primary Sedimentation tanks is insufficient to meet the Ten States Standards of 240 mgd. With the recommended modifications in Phase 1 the hydraulic capacity of the PST influent channel will be sufficient for flows up to 180 mgd. Additional hydraulic improvements would be required to achieve a flow of 240 mgd into and through the PSTs.

The existing PSTs were evaluated based on different flowrates and the calculated surface overflow rates. The results are provided in in **Table 7-2.** According to the calculated values, the recommended SOR of 2,000 gallons per day per square foot (gpd/ft^2) will not be exceeded with influent flow of 180 mgd), but will be exceeded at 240 mgd (2,581 gpd / sf, or 29% above recommended SOR)

Table 7-2: Surface Overflow Rate of Primary Clarification

Item Description	Existing Facilities			
Wastewater Flow, mgd	57	150	180	240
Number of Tanks	10	10	10	10
Tank Surface Area, ft^2	9,300	9,300	9,300	9,300
Total Tank Surface Area, ft^2	93,000	93,000	93,000	93,000
Surface Overflow Rate, (SOR), gpd/ft^2	613	1,613	1,935	2,581
Recommended SOR, gpd/ft^2	2000	2000	2000	2000
Is Recommended SOR Exceeded?	No	No	No	Yes

7.8 Biological Treatment and Secondary Clarification

The aeration facility (bioreactors) includes eight (8) aeration tanks divided equally into a North and South batteries, with a combined tank volume of 14.79 million gallons. Flow enters the aeration tanks from the Primary Sedimentation Tanks and discharges into effluent channels which direct flow to the Final Sedimentation Tanks. The final sedimentation facilities include eight (8) tanks with a combined tank surface area of 158,656 ft^2. With the hydraulic modifications to the Primary Treatment system stated in the previous sections of the Report, the secondary treatment and clarifiers are able to handle flows up to 180 mgd.

7.9 Disinfection

Disinfection facilities include two (2) Chlorine Contact Tanks with a combined volume of 2.69 million gallons. Outfall from the CCTs is discharged into the Delaware River through a dual outfall pipe system ranging from 60-inches to 72-inches in diameter.

Detention times were evaluated for the disinfection facilities at varying flow rates and at the maximum storm tide elevation. The recommended minimum detention time of 15 minutes, as recommended by the 10 State Standards, will not be met at 240 mgd without expansion of the Chlorine Contact Tanks.

7.10 Solids Handling

The performance of the existing solids handling facilities for a maximum month flow of 60 mgd and 80 mgd was shown in a Camden Bioenergy, LLC, Block Flow Diagram dated, May13, 2016. According to the block diagram information with all digesters in operation, the digester units provide 21 days of retention time for maximum month flow of 60 mgd (existing flow) and 16 days of retention time for a maximum month flow of 80 mgd (future flow). Solids retention times greater than 15 days will meet the regulatory requirements for good practice of anaerobic digestion of sludge, however, the retention time will unlikely be met with the infrequent wet weather flows to the plant upwards to 240 mgd, because of higher solids capture to meet permit requirements.

Section 8 Technology Review of Phase 2 Options

8.1 Overall Treatment Options

The first step in the evaluations of wet weather treatment options is to fully consider various potentially viable options and to develop appropriate and cost-effective recommendations for reliably meeting treatment goals using new and existing facilities at the Delaware No. 1 WPCF. These should be considered together with information from CCMUA's other ongoing wet weather programs. Discussions with NJDEP are recommended to determine wet weather treatment requirements. The Phase 2 alternatives to increase the Delaware No. 1 Water Pollution Control Facility wet weather hydraulic capacity up to 240 mgd are:

A. **Wet Weather Storage Tank:** Stores the wet weather flow volumes in excess of 150 mgd in large tanks until the wet weather storm event passes and the plant has capacity to process the stored raw wastewater

B. **All Flow Treatment:** Full treatment of wet weather flows including primary, secondary and disinfection

C. **Wet Weather Plant Bypass:** Up to 55 mgd bypasses the existing primary and secondary treatment facilities after preliminary treatment and is discharged into the Delaware River through a new wet weather outfall. This option can be planned to provide different levels of treatment as follows:

C-1) Standard-rate or high-rate disinfection with no preliminary treatment or raw sewage pumping upgrades.

C-2) Screening of raw wastewater to 0.50 inch followed by standard-rate or high-rate disinfection with no grit removal system or raw sewage pumping upgrades.

C-3) Screening of raw wastewater to 0.50 inch followed by high-rate primary treatment in area north of existing aeration/final tanks, and standard-rate or high-rate disinfection with no grit removal system or raw sewage pumping upgrades.

C-4) Screening of raw wastewater to 0.50 inch followed by high-rate primary treatment in area north of existing aeration/final tanks, standard-rate or high-rate disinfection, and chlorine contact tank with no grit removal system or raw sewage pumping upgrades.

C-5) Preliminary treatment and raw sewage pumping upgrades followed by high-rate primary treatment in area north of existing aeration/final tanks, standard-rate or high-rate disinfection, and chorine contact tank to provide a firm capacity of 240 mgd.

D. Secondary Bypass: Up to 55 mgd bypasses the existing secondary treatment facilities and is discharged into the Delaware River through a new wet weather outfall. These two options use enhanced primary treatment systems constructed within the existing Primary Sedimentation Tanks:

D-1) Preliminary treatment and raw sewage pumping upgrades followed by up to 92 mgd of high-rate primary treatment constructed within existing Primary Sedimentation Tanks followed by standard-rate or high-rate disinfection, and chorine contact tank to provide a firm capacity of 240 mgd.

D-2) Preliminary treatment and raw sewage pumping upgrades followed by Chemically Enhanced Primary Treatment (CEPT) in existing Primary Sedimentation Tanks followed by standard-rate or high-rate disinfection, and chorine contact tank to provide a firm capacity of 240 mgd.

Because of a lack of available space at the WPCF, construction of a new Wet Weather Storage Tank is not considered feasible and will not be given further consideration. With the onsite Wet Weather Storage Tank option considered infeasible, this study concentrates on the remaining options. These options are outlined in more details in the following Report sections.

8.1.1 Option B: All Flow Treatment

In this option, high-rate primary, high-rate secondary, and standard-rate or high-rate disinfection systems are integrated to increase the treatment capacity of the wastewater treatment plant. A major drawback of any high-rate secondary treatment system is the time for system acclimation, which means the systems cannot be brought into service rapidly and for a short period of time. As such, if this option is used at the plant, the system would need to remain in service at all times. In addition, the high-rate secondary option is likely to produce high volumes of waste primary and secondary sludge that would need to be treated in the solids treatment processes.

Available high-rate primary, secondary and disinfection systems are shown in **Figure 8-1.** For high-rate primary treatment CoMag®, Actiflo®, Densedag®, and Chemically Enhanced Primary Treatment (CEPT) are examined. For high-rate biological treatment systems BioMag® and Bio-Actiflo® technologies are options. Lastly, for high-rate disinfection; chlorination, ozone, UV, peracetic acid, and E-Beam are reviewed.

The “All Flow Treatment” option requires the expansion of the preliminary treatment systems to handle increased flows above 180 mgd on firm capcity basis. On an installed capacity basis, the preliminary treatment system can handle flows up to 240 mgd if all system units are available, no units out of service. In addition, at a minimum a portion of the Primary Sedimentation Tanks will need to be converted into a high-rate treatment technology such as Chemically Enhanced Primary Treatment. Further, the areation tanks and secondary clarifiers would be converted to one of the high-rate biological treatment systems such as Bio-Actiflo®. Lastly, standard-rate or high-rate disinfcetion would provide further treatment to the wet weather flows. A schematic of the proposed option is shown in **Figure 8-2.**

Figure 8-1: Available High Rate Primary, Secondary and Disinfection Technologies for Option B

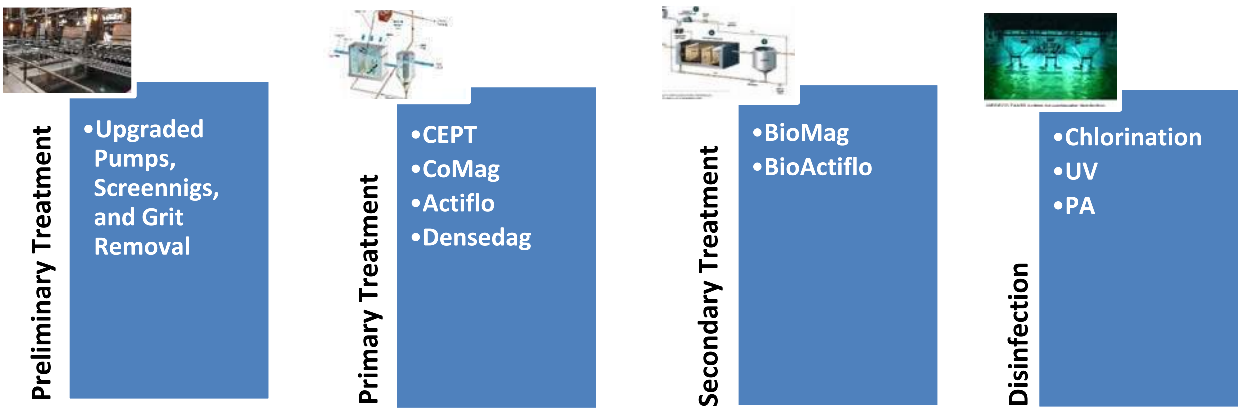
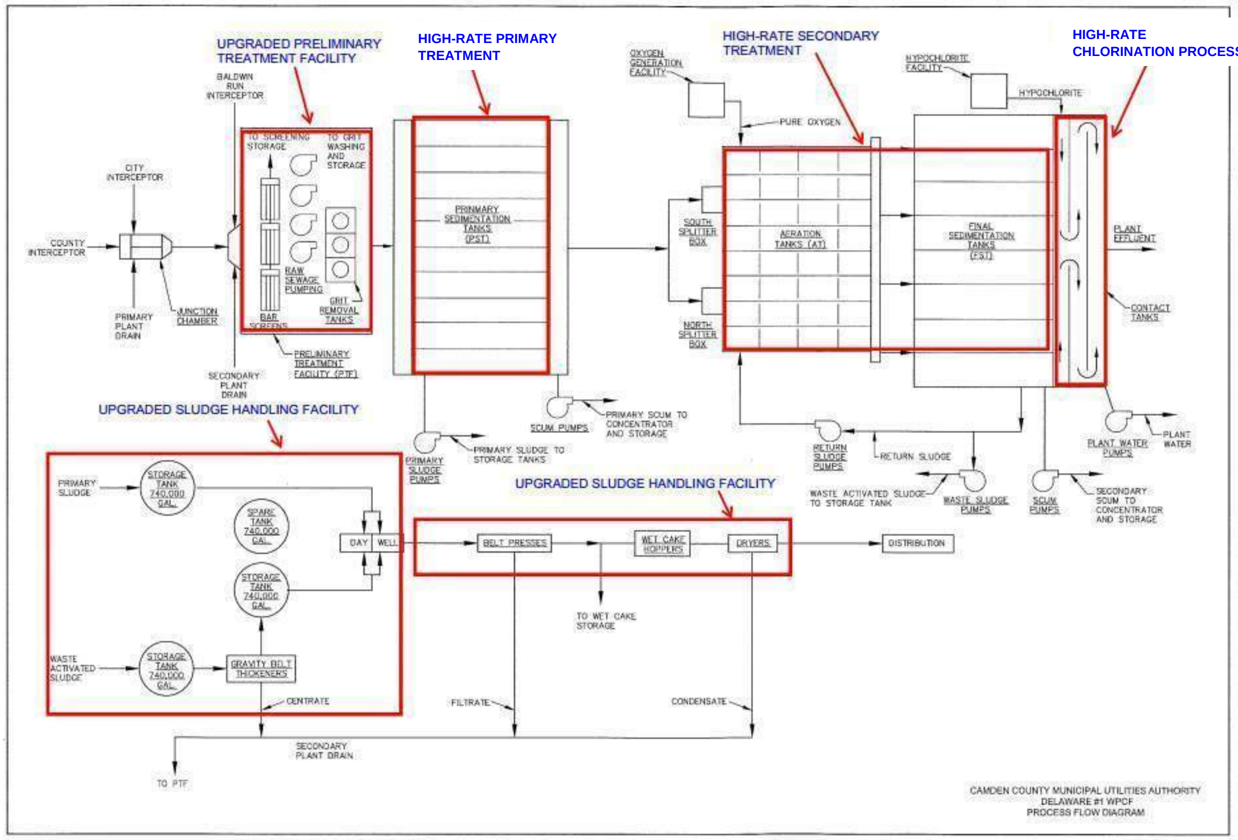


Figure 8-2: "All Flow Treatment" Option



(Note: Sludge Handling Facility may require upgrade.)

8.1.2 Option C: Wet Weather Plant Bypass

As discussed in the previous sections of the Report, the Plant capacity can be increased up to 180 mgd with some hydraulic modifications. In order to further increase the treatment capacity up to 240 mgd, a parallel on-site wet weather treatment facility can be constructed. The new 55 MGD treatment facility would bypass the existing primary and secondary treatment facilities after preliminary treatment and discharge into the Delaware River through a new wet weather outfall. This option can be implemented in various combinations as illustrated in Options C-1 to C-5 below.

Option C-1: In this option, the existing preliminary treatment systems including raw sewage pumps, screenings, and grit removal will be used with no or minimal upgrades to provide preliminary treatment for wet weather flows up to 240 mgd. This option does not account for any unit process element being out of service.

A new 55 mgd bypass conduit will be used to discharge the flow to the Delaware River through a new wet weather outfall. Standard-rate or high-rate disinfection would follow to lower the bacterial load of the discharged flow. It should be noted that the disinfection of preliminary treated wet weather flows with relatively high total organic carbon (TOC) and total suspended solids (TSS) values may decreases the efficiency of the process and may increases the chance of formation of undesirable by-products (e.g., trihalomethanes [THMs] and haloacetic acids [HAAs]). A schematic layout and a process flow diagram for Option C-1 are shown in **Figure 8-3** and **Figure 8-4**.

Figure 8-3: Option C-1 Layout with Available Technologies for Disinfection

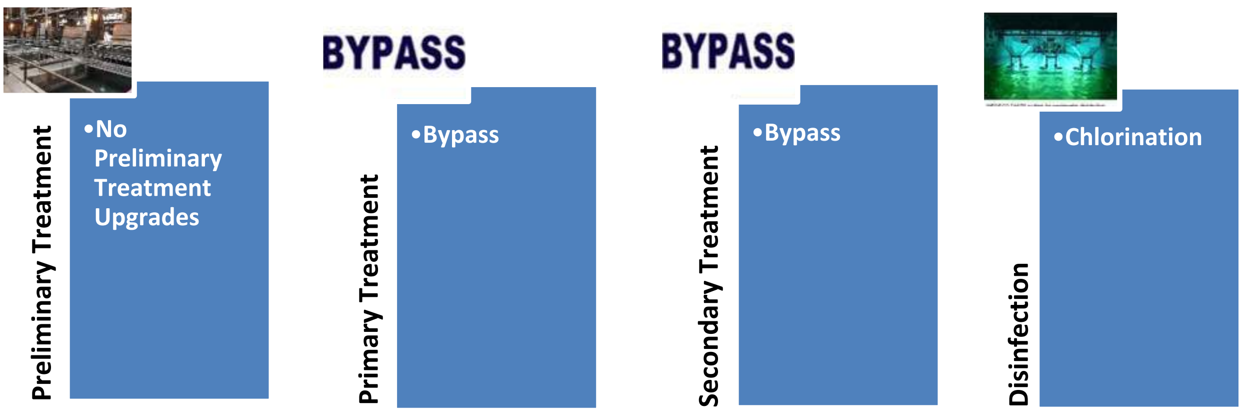
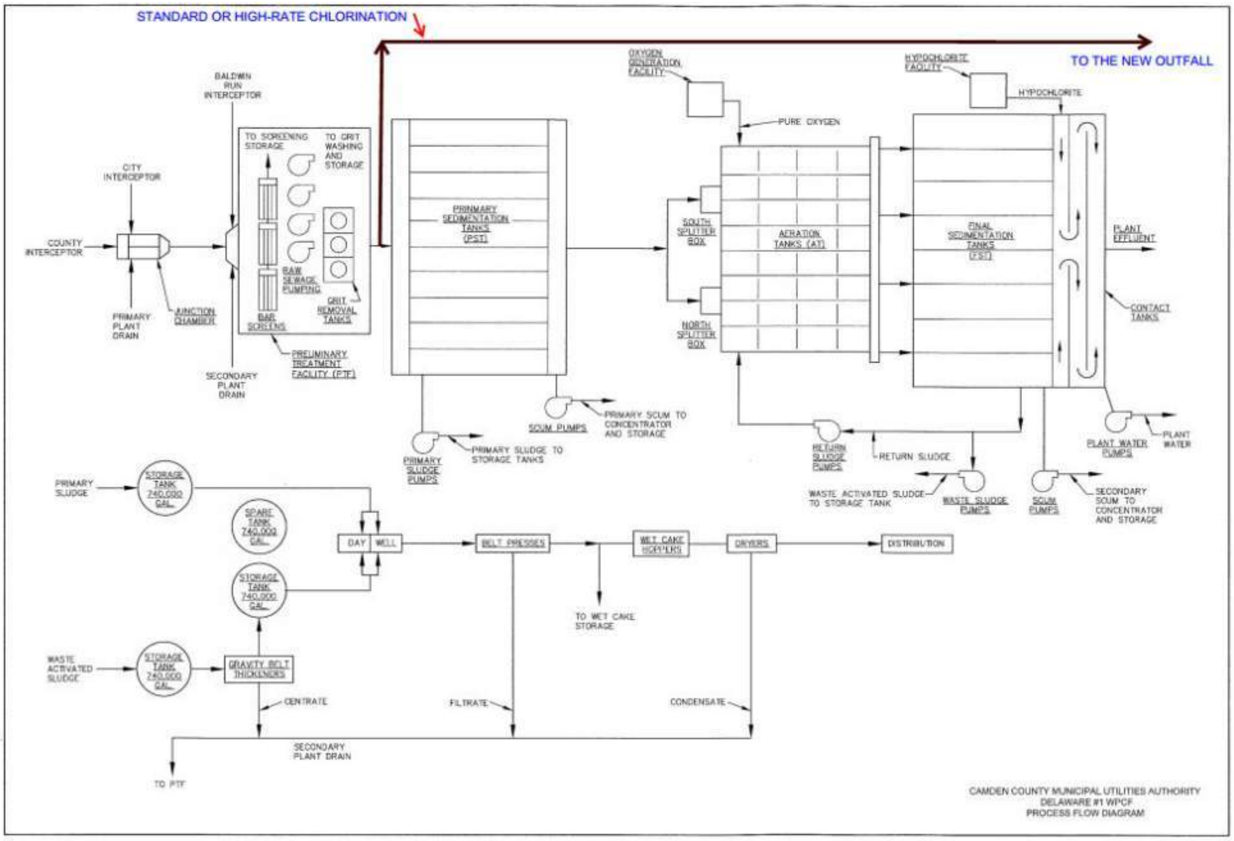


Figure 8-4: Schematic Diagram of the Proposed Upgrades in Option C-1



Option C-2: This option includes the Option C-1 wet weather disinfection and adds a fine screening system. Therefore, the existing raw sewage pumps and grit removal will be used with no upgrades. This option does not account for any unit process element being out of service.

Similar to Option C-1, a new 55 mgd bypass conduit will be used to discharge the flow to the Delaware River through a new wet weather outfall. Standard-rate or high-rate disinfection would follow to lower the bacterial load of the discharged flow. A schematic layout and the process flow diagram for Option C-2 are shown in **Figure 8-5 and Figure 8-6**.

Figure 8-5: Option C-2 Layout with Available Technologies for Disinfection

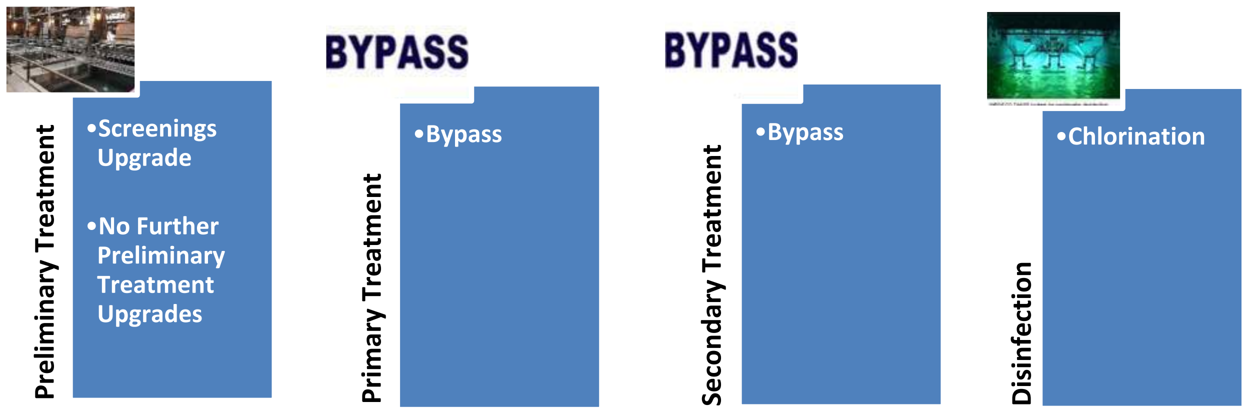
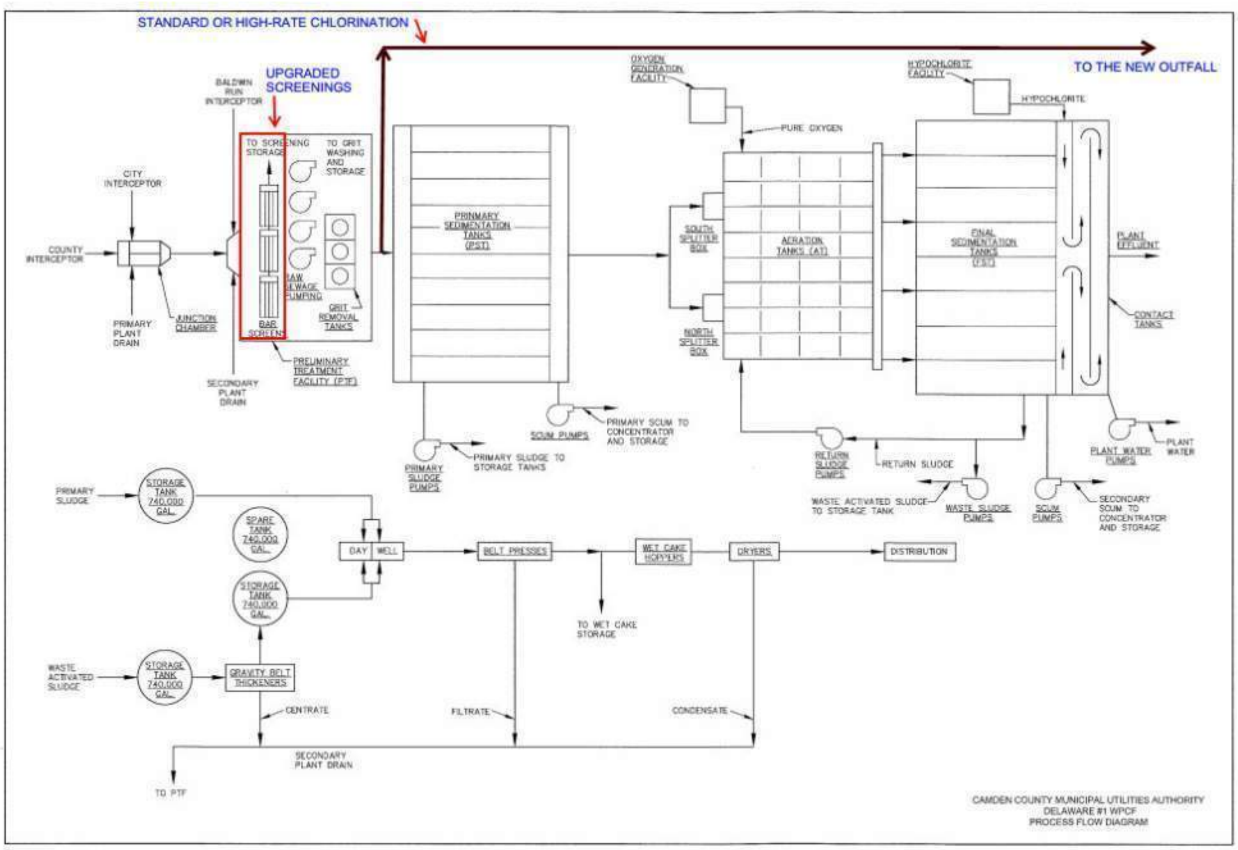


Figure 8-6: Schematic Diagram of the Proposed Upgrades in Option C-2



Option C-3: This option proposes the implementation of Option C-2, plus a new high-rate primary treatment system in the area north of the existing aeration/final tanks. This new high-rate system will provide primary treatment for the wet weather flows up to 55 MGD. Like options C-1 and C-2, this option does not account for any unit process element in the preliminary treatment system being out of service.

Similar to the previous options, a new 55 mgd bypass conduit will be used to discharge the flow to the Delaware River through a new wet weather outfall. Standard rate or high rate disinfection would follow to lower the bacterial load of the discharged flow. A schematic layout and the process flow diagram for Option C-3 are shown in **Figure 8-7** and **Figure 8-8**.

Figure 8-7: Option C-3 Layout with Available Technologies for Primary Treatment and Disinfection

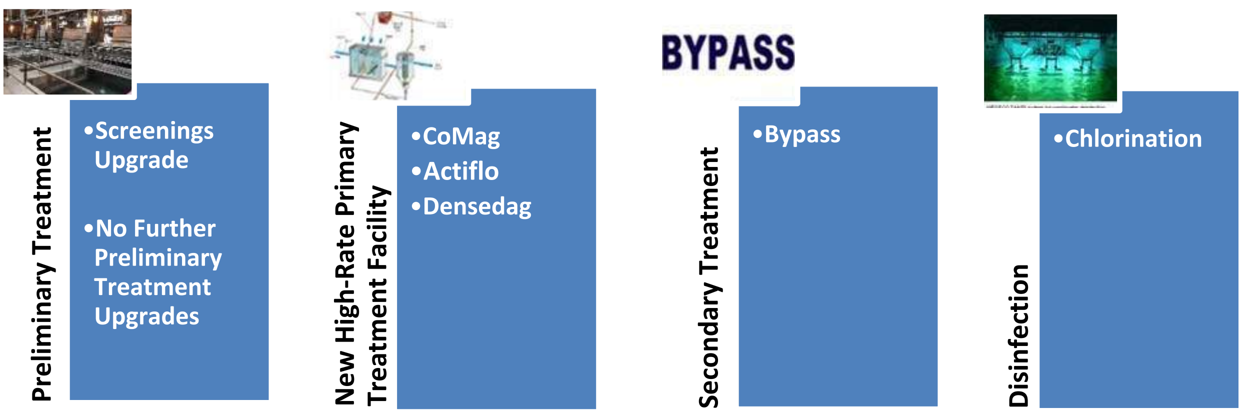
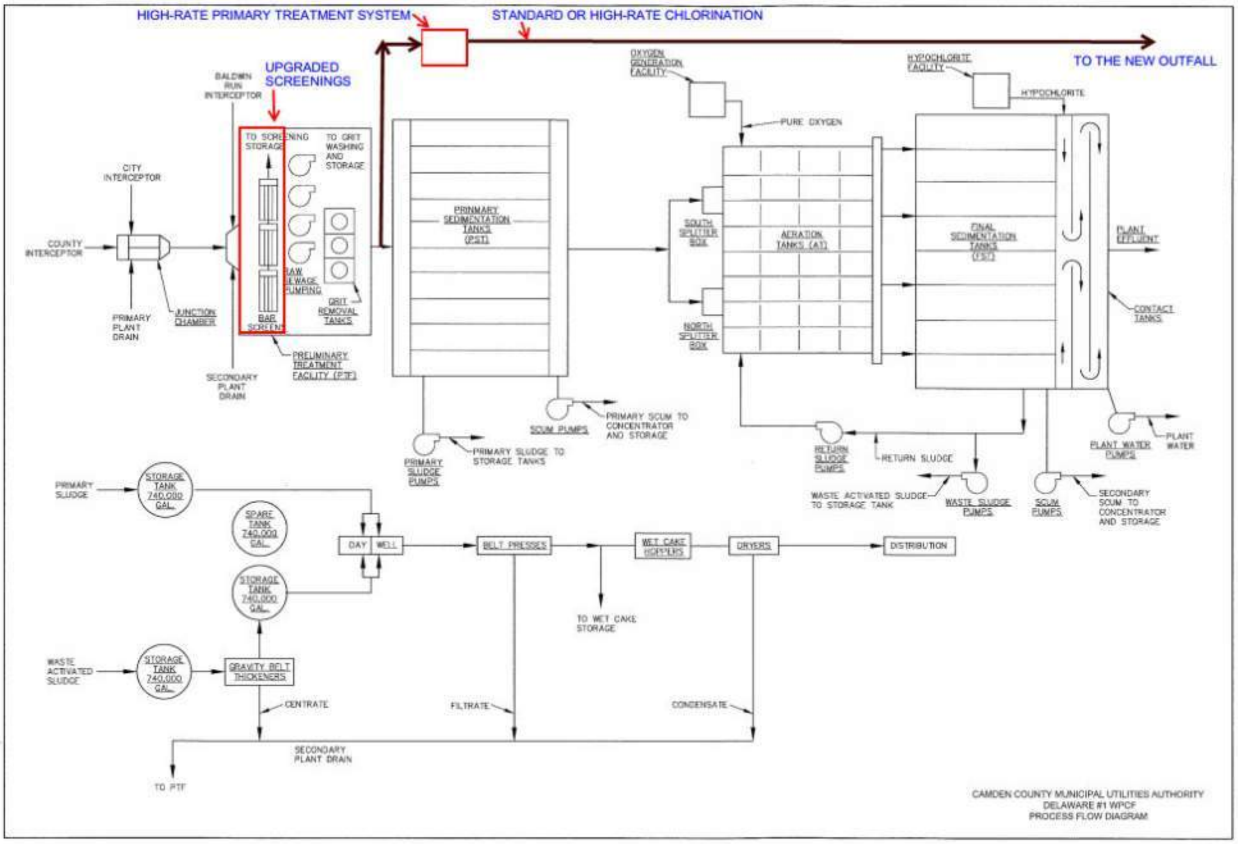


Figure 8-8: Schematic Diagram of the Proposed Upgrades in Option C-3



Option C-4: Option C-4 uses the same concept that was presented for Option C-3, however this option uses a chlorine contact tank (if chlorine is selected as the disinfectant) to provide disinfection to the wet weather flows. Like options C-1 and C-2 and C-3, this option does not account for any unit process element in the preliminary treatment system being out of service.

The construction of a new chlorine contact tank not only increases the disinfection process efficiency by lowering the required chlorine dosage, but also is in compliance with the Recommended Standards for Wastewater Facilities. A minimum of 15 minutes contact time is required to achieve the treatment goals. Therefore, the required chlorine contact tank should provide 15 minutes of contact time for the maximum flow of 55 mgd.

Similar to the previous three options, a new 55 mgd bypass conduit will be used to discharge the flow to the Delaware River through a new wet weather outfall. A schematic layout and process flow diagram for Option C-4 are shown in **Figure 8-9** and **Figure 8-10**.

Figure 8-9: Option C-4 Layout with Available Technologies for Primary Treatment and Disinfection

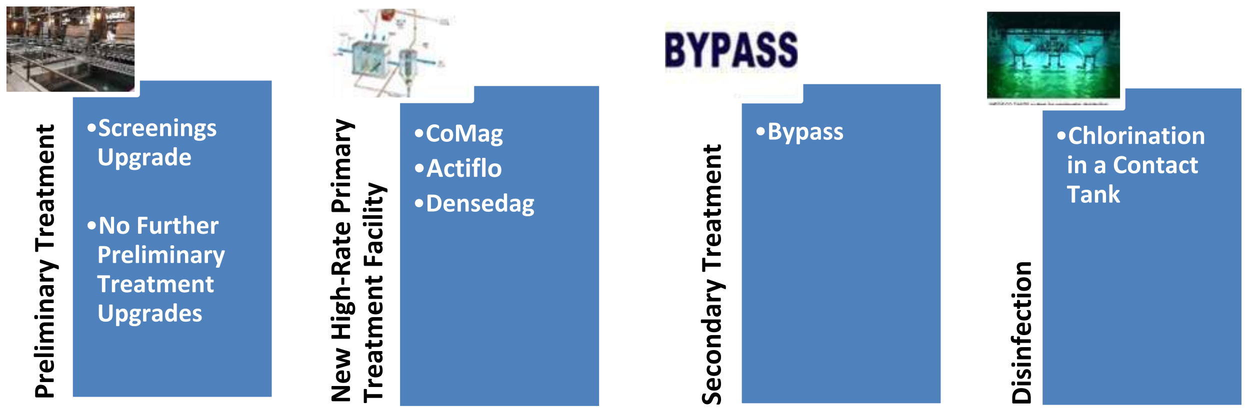
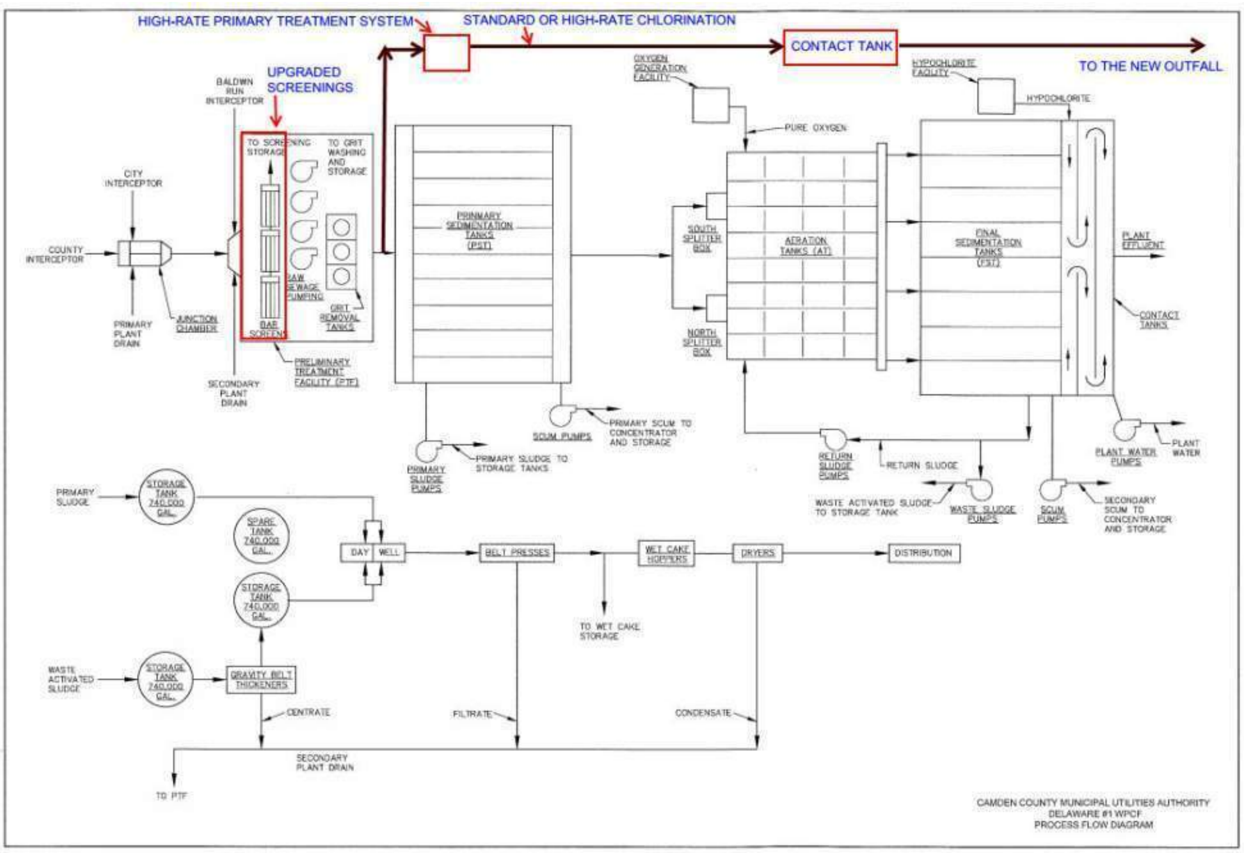


Figure 8-10: Schematic Diagram of the Proposed Upgrades in Option C-4



Option C-5: This option recommends a full upgrade to the existing preliminary treatment systems including the raw sewage pumps, screenings, and grit removal units as well as construction of a new 55 mgd high-rate primary treatment system in the area north of the existing aeration/final tanks. This option will increase the firm capacity of the Plant to 240 mgd.

Standard-rate or high-rate disinfection in a contact tank (if chlorine is selected as disinfectant) will be provided to disinfect the wet weather flows. The construction of a new chlorine contact tank not only increases the disinfection process efficiency by lowering the required chlorine dosage, but also is in compliance with the Ten States Standard.

Similar to the previous options, a new 55 mgd bypass conduit will be used to discharge the flow to the Delaware River through a new wet weather outfall. A schematic layout and process flow diagram for Option C-5 is shown in **Figure 8-11** and **Figure 8-12**. This option will increase the firm capacity of the Plant to 240 mgd.

Figure 8-11: Option C-5 Layout with Available Technologies for Primary Treatment and Disinfection with Full Preliminary Upgrades

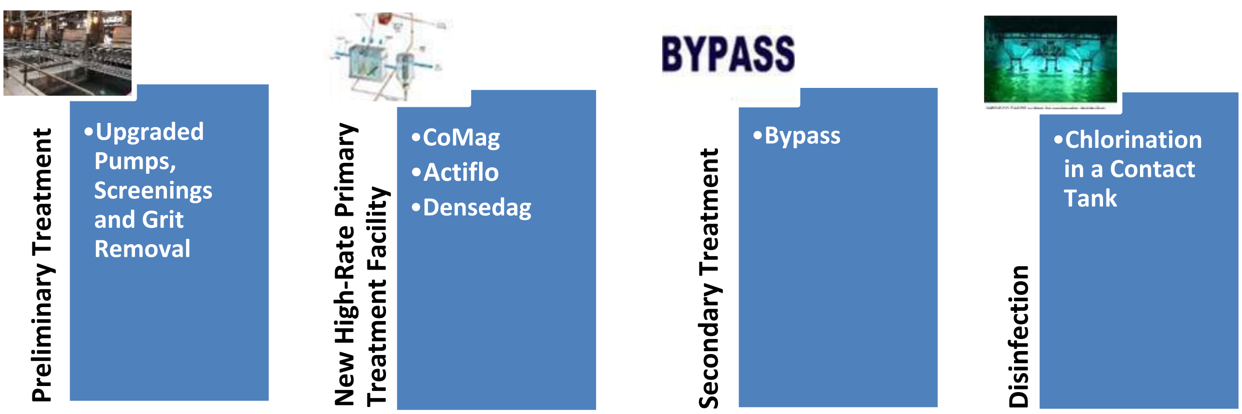
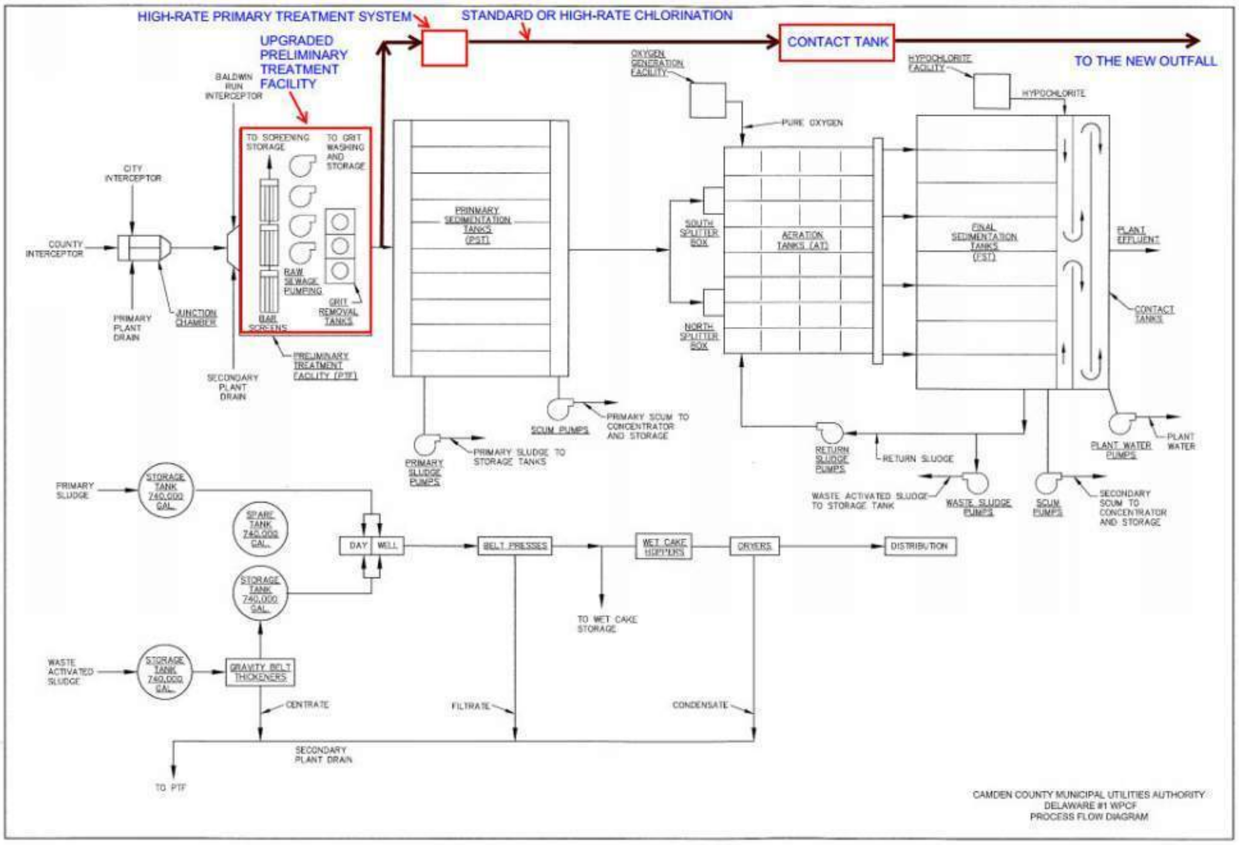


Figure 8-12: Schematic Diagram of the Proposed Upgrades in Option C-5



8.1.3 Option D: Secondary Bypass

The secondary bypass option proposes the use of existing preliminary treatment with some modifications and upgrades as well as the upgraded primary treatment tanks. The upgraded Primary Sedimentation Tanks will provide capacity for the treatment of wet weather flows up to 240 mgd. The primary effluent flows up to 180 mgd will be sent to the existing Secondary Treatment Tanks and the remaining flows up to 55 mgd will be directed to a new bypass conduit. Therefore, this option eliminates the complexities of maintining a high-rate secondary treatment system in service. A new 55 mgd conduit will be used to bypass the secondary treatment and to discharge the flow to the Delaware River through a new wet weather outfall. Standard-rate or high-rate disinfection will be implemented to lower the bacterial load of the discharged flow. This option can be implemented in two ways:

Option D-1: The existing primary treatment system consists of ten Primary Sedimentation Tanks that can provide treatment for wet weather flows up to 180 mgd. In this Option, two Primary Tanks will be upgraded to a new high-rate primary treatment system to increase the treatment capacity up to 240 mgd. The remaining Primary Tanks (eight tanks) can provide treatment for dry and wet weather flows up to 148 mgd and the new high-rate treatment will provide additional 92 mgd treatment capacity and can be brought into service for flows higher than 148 mgd and up to 240 mgd.

A new 55 mgd conduit bypasses the existing secondary treatment and discharges the wet weather flow to the Delaware River after proper disinfection and through a new wet weather outfall. A new chlorine contact tank will provide a minimum of 15 minutes contact time according to the Recommended Standards for Wastewater Facilities. All the other treatment components (e.g. influent pump station and grit removal system) will be upgraded and the rated treatment capacity of the WPCF will be increased up to a firm capacity of 240 mgd. Option D-1 is shown in **Figure 8-13** and **Figure 8-14**.

Figure 8-13: Option D-1 Layout with Available High-Rate Primary Treatment Technologies and Disinfection

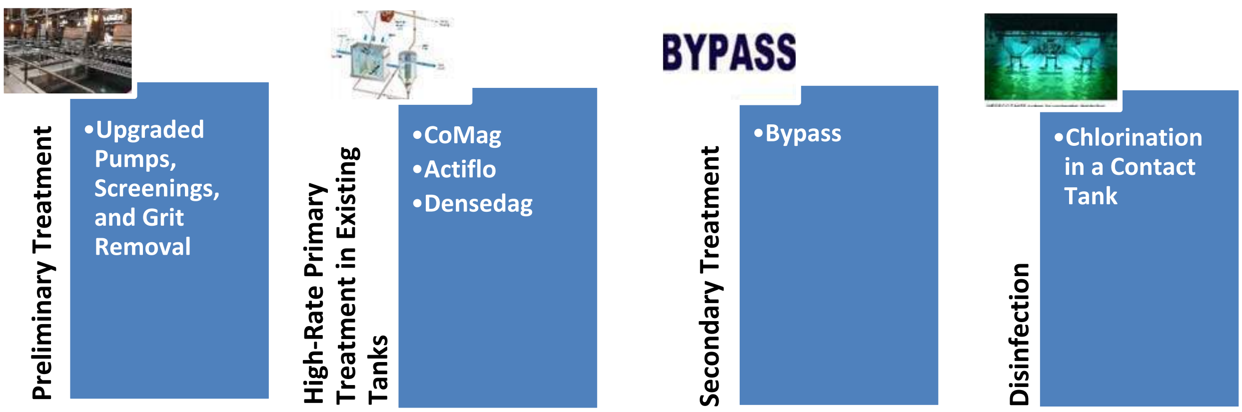
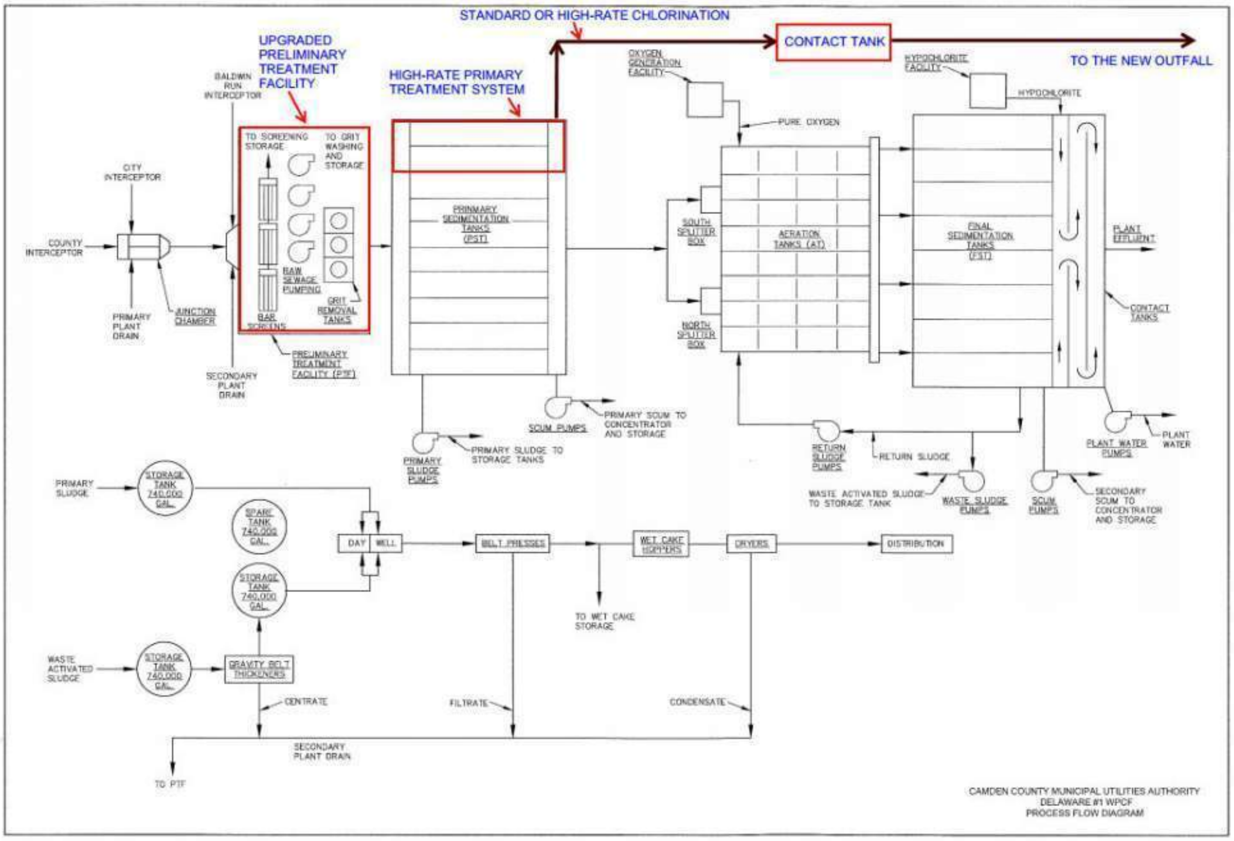


Figure 8-14: Schematic Diagram of the Proposed Upgrades in Option D-1



Option D-2: This Option converts the existing Conventional Primary Treatment system to the Chemically Enhanced Primary Treatment system. CEPT system provides for a higher surface overflow rate, which enhances the settling characteristics of the wet weather flows and increases the treatment capacity.

The primary effluent flows up to 180 mgd will be sent to the existing Secondary Treatment Tanks and the remaining flows up to 55 mgd will be directed to a new bypass conduit. A new 55 mgd conduit bypasses the existing Secondary Treatment and discharges the wet weather flow to the Delaware River after proper disinfection and through a new wet weather outfall. A new chlorine contact tank will provide a minimum of 15 minutes contact time according to the Recommended Standards for Wastewater Facilities. All the other treatment components (e.g. influent pump station and grit removal system) will be upgraded and the rated treatment capacity of the WPCF will be increased up to firm cacicity of 240 mgd. Option D-2 is shown in **Figure 8-15** and **Figure 8-16**.

Figure 8-15: Option D-2 Layout with High-Rate Chemically Enhanced Primary Treatment and Disinfection

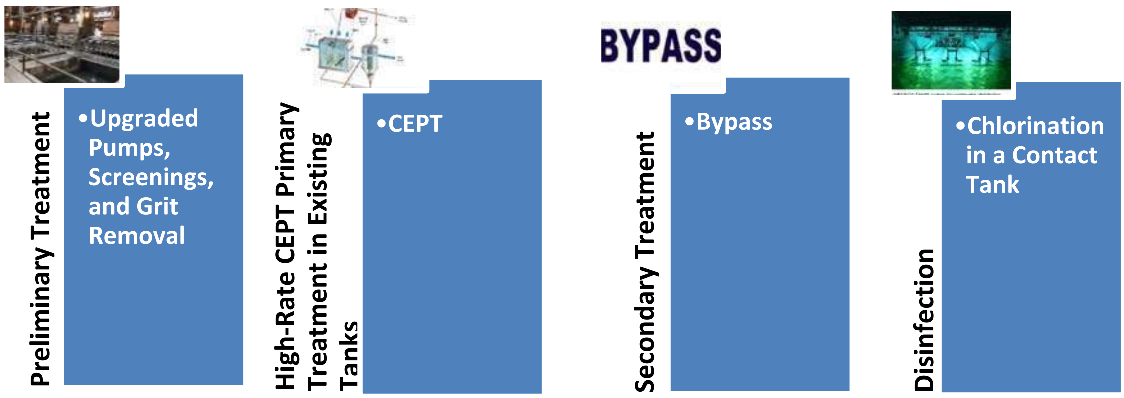
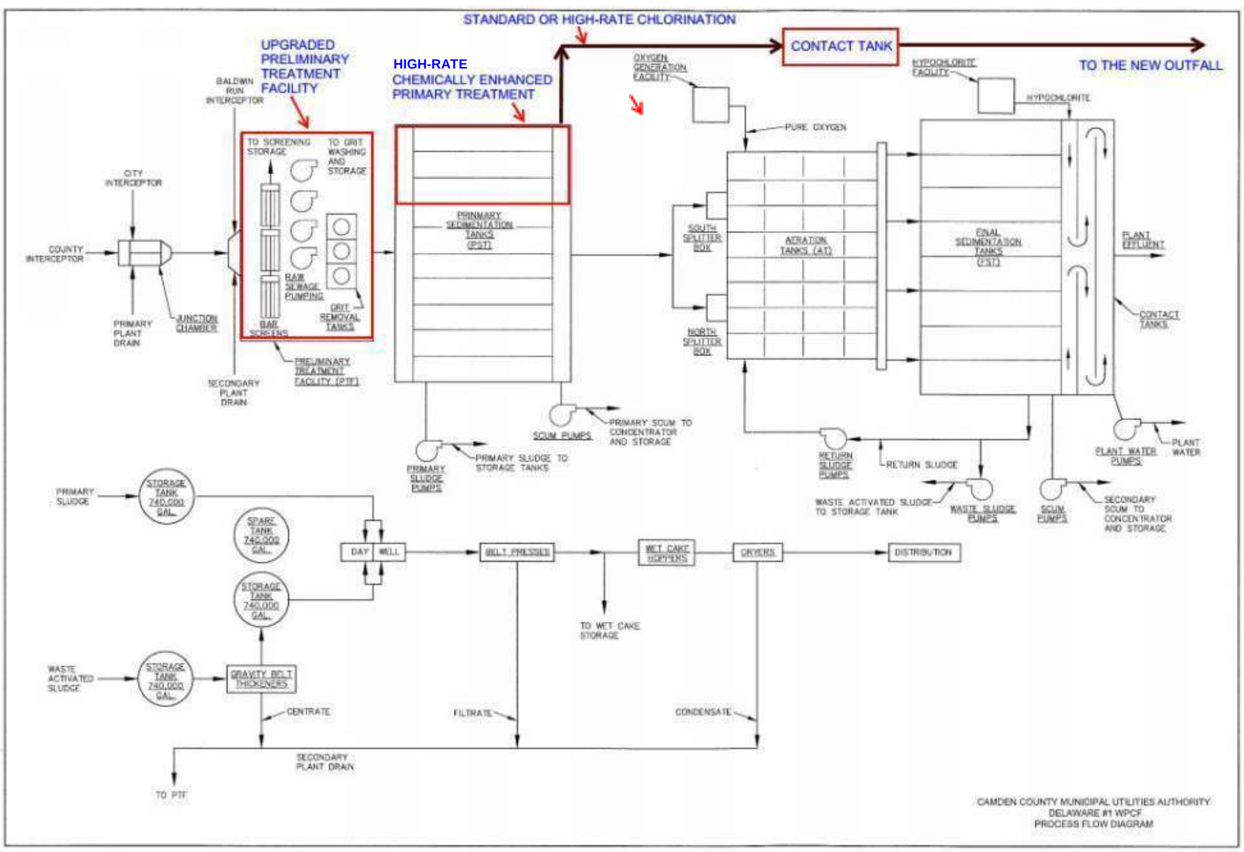


Figure 8-16: Schematic Diagram of the Proposed Upgrades in Option D-2



8.2 Wet Weather Solids Handling

Application of high-rate primary or secondary treatment technologies results in a higher sludge production rates than existing operations due to the higher TSS and BOD removal efficiencies. Currently, it is expected that the existing solids handling facility has enough capacity to store and treat infrequent wet weather flows; however, the solids handling facility may require an upgrade if more frequent wet weather flow occurs in the future.

8.3 High-Rate Primary Treatment Alternatives

High-rate primary treatment (especially high-rate clarification) technologies have been widely tested and used at WWTPs to manage wet weather flows. A higher settling rate in primary clarifier treatment may be possible with use of either chemically enhanced treatment or by using high-rate settling technologies. Each of these options are discussed later in this section of the Report.

8.3.1 Chemically Enhanced Primary Treatment

Chemically Enhanced Primary Treatment is used to enhance removal of TSS and associated BOD through chemical coagulation and flocculation. Typically, metal salts and/or polymers are added to the primary sedimentation basins. Typical chemicals include ferric chloride ($FeCl_3$) or aluminum sulfate ($Al_2(SO_4)_2$) with possible performance enhancement with polymers. CEPT allows the primary clarifiers to operate at higher overflow rates, while maintaining high removal rates of TSS and BOD. Higher primary effluent water quality has an additional benefit of reducing the disinfectant dose in the downstream disinfection facility.

Advantages of CEPT:

- High removal efficiency
- High surface overflow rate
- Consistent performance
- Low construction cost

Disadvantages of CEPT:

- Increased mass of primary sludge
- Production of sludge that is more difficult to thicken and dewater
- Potential adverse effects on biological treatment if too much alkalinity or phosphorous is removed.

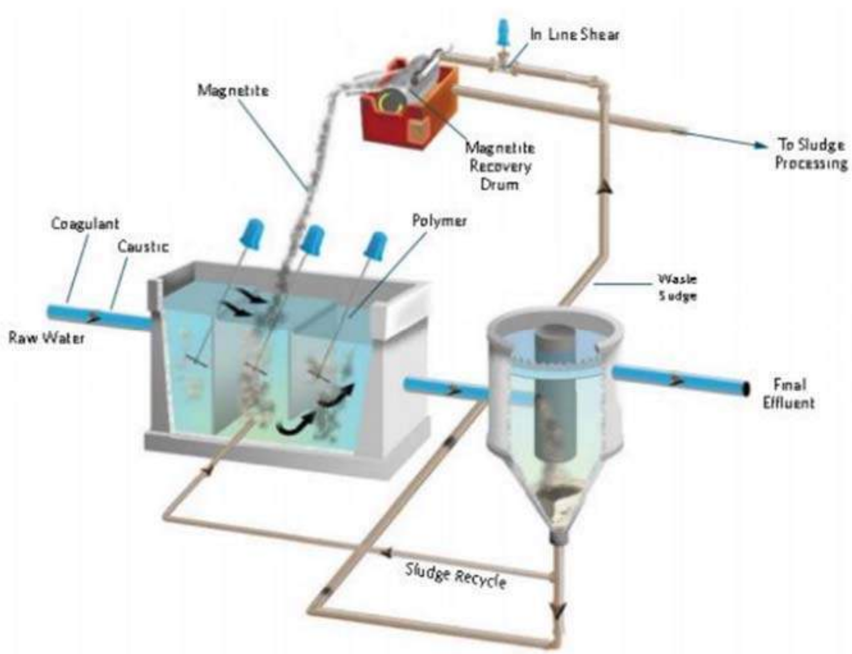
8.3.2 CoMag®

CoMag® is a high-rate ballasted settling system that uses magnetite (Fe_3O_4) particles to provide enhanced settling. CoMag® is based on conventional coagulation and flocculation but uses the magnetite particles that have a relatively high specific gravity to ballast the floc thus providing faster settling and thus reducing footprints due to a small settling zone. **Figure 8-17** provided by Evoqua Water Technologies, the supplier for CoMag®, shows the overall system components.

The system requires pretreatment with coagulant, such as polymer, iron salts, alum, or a combination of these chemicals. Caustic may be added if pH adjustment and alkalinity control is needed for downstream process considerations. Polymer and magnetite are added in different zones to enhance the coagulation process and improve settling characteristics. The ballasted mix goes to a polymer dosing section and into a clarifier. The ballasted floc allows the clarifier to be reduced in size compared to traditional clarifiers. A potential side-effect of the system is that organic phosphorous can be removed. The enhanced settling characteristics of the CoMag® system can:

- Increases solids overflow rate up to 10 times
- Increases solids loading rate up to 20 times
- Decreases Fecal Coliform concentration to 200 cfu/100 mg/L (plant disinfection standard)
- Reduce turbidity to less than 1 NTU

Figure 8-17: CoMag® Process Flow Diagram



A few factors affect the amount of magnetite required, but according to Greeley and Hansen's previous experiences, it is estimated that at least 10 pounds of magnetite per million gallons of treated water is required. Magnetite cost is roughly $0.20 to $0.50 per pound delivered. It should be noted that the magnetite is removed from the floc by magnet systems mounted in a drum separator. The separation efficiency is estimated to be 99%, therefore the cost of magnetite makeup is minimal.

Advantages:

- System provides 90% TSS removal and 50% BOD removal.
- System partially decreases copper, aluminum, and arsenic, although these are not required by CCMUA permit
- Unit can be used to supplement primary treatment during low-flow conditions.
- System potentially can remove portions of the phosphorous loading, although this is not required by CCMUA's permit.

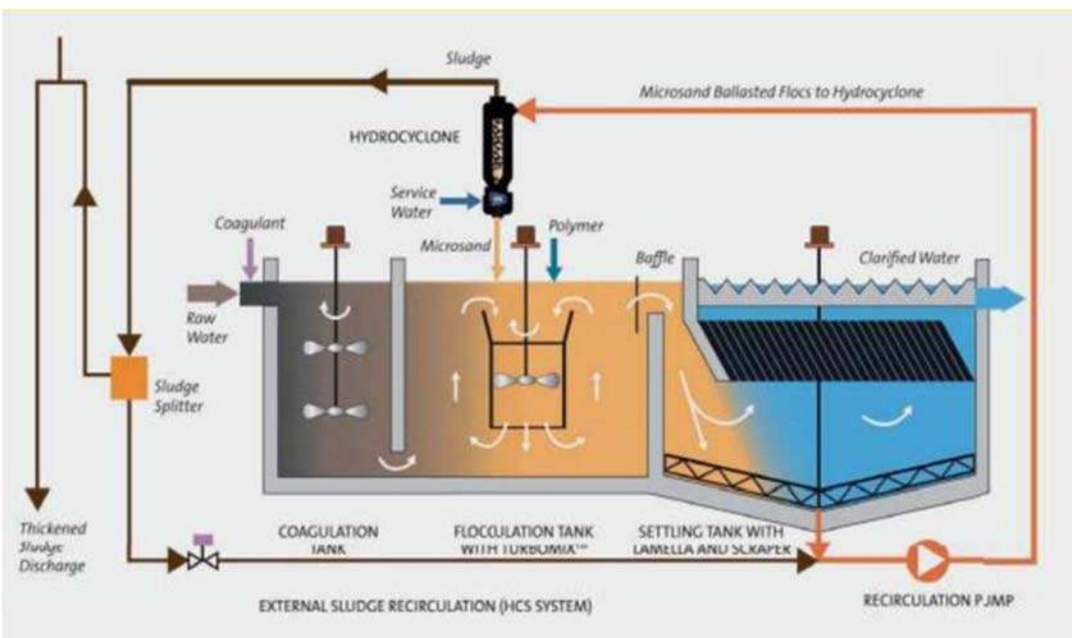
Disadvantages:

- System requires the use of magnetite. Although readily available from Evoqua, volumes loss during operation is small. But represent an operating consumable.
- Process increases the degree of operational complexity.
- System requires coagulant and polymer additions.
- System produces an additional waste sludge which will have to be processed and disposed.

8.3.3 Actiflo®

Actiflo® is similar to CoMag® and relies on ballasted sedimentation, however, the ballast is microsand rather than magnetite. The clarifier section differs from the CoMag® system in that Actiflo® uses lamellar tubes to enhance settling. A simple flow diagram of an Actiflo® system is shown in **Figure 8-18**.

Figure 8-18: Actiflo® Process Flow Diagram



Similar to the CoMag® system, Actiflo® uses coagulant and a coagulation contact tank followed by polymer in a floc tank but uses lamella plate assisted clarification. Sludge is recycled through a hydrocyclone to recapture the microsand. Sludge is wasted back to the treatment plant with a portion recycled to the clarifier. The site layout for the Actiflo® system is similar to that for the CoMag® system. Like CoMag®, the anticipated performance is 90% TSS removal and 50% BOD removal. Actiflo® can be brought into service relatively quickly (in less than 1 hour).

Advantages:

- Reduced primary clarification footprint.
- System removes portions of the phosphorous loading.

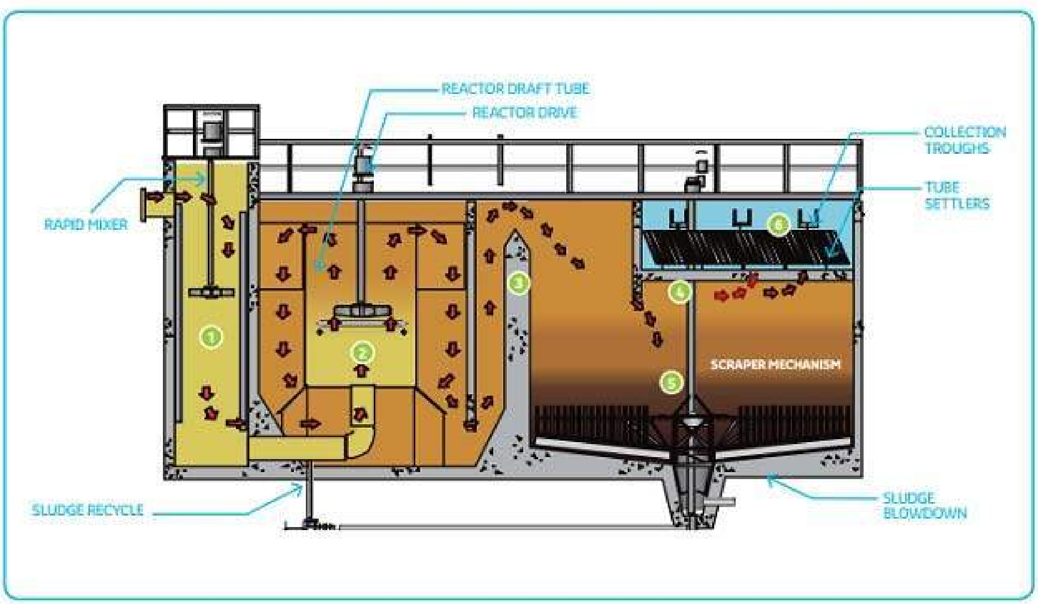
Disadvantages:

- System requires use of microsand. Although readily available from Veolia, volume losses during operation are small, but represent an operating consumable.
- Process increases the degree of operational complexity.
- System requires a coagulant and polymer.
- System produces additional waste sludge that will have to be processed and disposed.

8.3.4 DensaDag® and DensaDag® XRC

DensaDeg® is a high-rate clarifier/thickener system based on recycling portions of the primary sludge blanket. It is a high-rate solids contact clarifier and uses flocculation and internal sludge recirculation followed by tube settlers. **Figure 8-19,** provided by Infilco, shows the overall system components.

Figure 8-19: DensaDag® Process Flow Diagram



The initial rapid mix stage is used to flash mix raw wastewater and coagulant. The coagulated wastewater flows to a reactor zone where flow is combined with return solids in an axial- turbine mixing zone and then flows up through a transition zone where additional flocculation takes place. Polymer is added in the reactor zone to assist flocculation and formation of dense floc particles. The floc flows to the settling stage that includes clarification settling through upflow tube settlers before sludge collection and removal.

The process is similar to Actiflo® except the settling is not ballasted and uses tube settlers. The DensaDeg® system relies on the recycle of primary solids and dense floc formation.

Advantages of DensaDeg®:

- Reduced footprint primary clarification
- System removes portions of the phosphorous loading
- Does not use a ballast for settling sludge

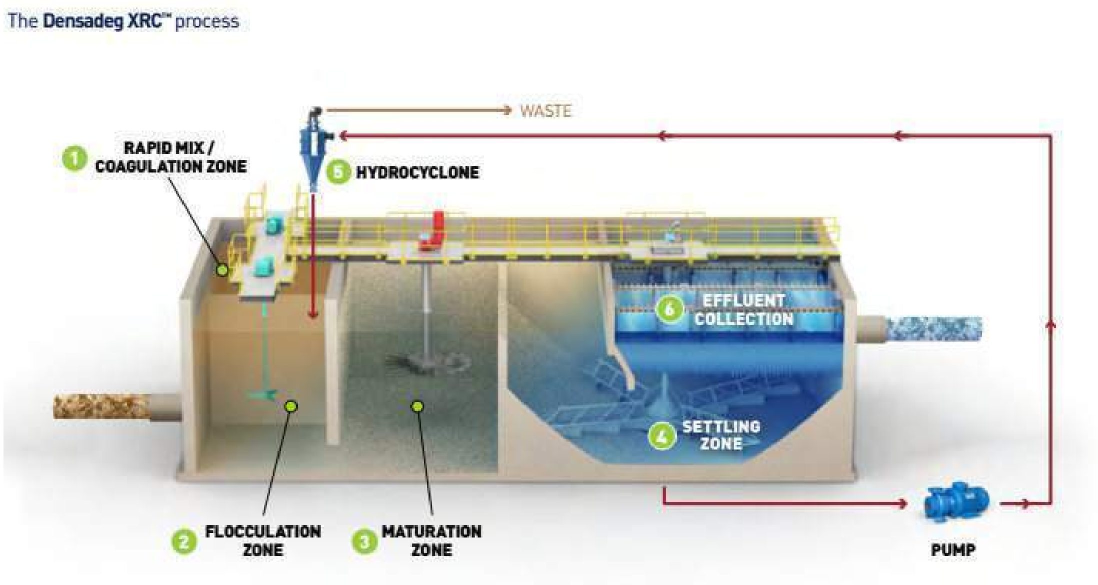
Disadvantages of DensaDeg®:

- System requires internal recirculation of sludge
- Process increases the degree of operational complexity
- System requires a coagulant and polymer
- System produces additional waste sludge that will have to be processed and disposed

DensaDeg® also has process that utilizes a ballast as a settling aid. This process is the DensaDeg® XRC. A schematic diagram of DensaDeg® XRC is presented in **Figure 8-20**. Compared to the previously described DensaDeg®, the new DensaDeg® XRC unit introduces a ballast media, garnet, to enhance sedimentation. This is very similar to the Actiflo® system but using dense garnet particles rather than sand

The six steps in the DensaDeg® XRC process include: combination of solids contact, ballast addition, and solids recirculation to provide extreme high-rate clarification. In Step 1, raw water enters into the rapid mix/coagulation zone where coagulant is added. In Step 2, floc particles begin to form where ballast media and polymer are added and stirred by a vertical turbine mixer. In the maturation zone (Step 3), flocculated particles are grown into large particles. Then flocculated solids flow into the settling zone, where heavy suspended solids settle to the bottom of the clarifier (Step 4). In Step 5, ballast media is separated by hydrocyclones and recycled to a flocculation zone and settled solids are sent to solids handling facility for further treatment. Strep 6 is the collection of effluent for discharge.

Figure 8-20: DensaDeg® XRC Process Flow Diagram



Advantages of DensaDeg® XRC:

- Extreme loading rates
- Compact footprint
- Ability to retrofit existing basins
- Fast start-up time

Disadvantages of DensaDeg® XRC:

- System requires garnet sand as a ballast. Volumes of operating loss are small, but represent an operating consumable
- Process increases the degree of operational complexity
- System requires a coagulant
- System produces additional waste sludge that will have to be processed and disposed
- Relatively high construction challenges

A comparison summary of advantages and disadvantages of each potential high-rate primary treatment system is provided in **Table 8-1**.

Table 8-1: Overall Comparison of High-Rate Primary Treatment Alternatives

High-Rate Primary Treatment	Advantages	Disadvantages
CoMag®	• System provides 90% TSS removal and 50% BOD removal. • System partially decreases copper, aluminum, and arsenic. • Unit can be used during low flow conditions. • System potentially can remove phosphorous.	• System requires use of magnetite. • Process increases the degree of operational complexity. • System requires coagulant and polymer additions. • System produces an additional waste sludge.
Actiflo®	• Reduced primary clarification footprint. • System removes portions of the phosphorous loading.	• System requires use of microsand. • Process increases the degree of operational complexity. • System requires a coagulant and polymer. • System produces additional waste sludge.
DensaDeg®	• Reduced primary clarification footprint. • System removes portions of the phosphorous loading. • Does not use a ballast for settling sludge.	• System requires internal recirculation of sludge. • Process increases the degree of operational complexity. • System requires a coagulant and polymer. • System produces additional waste sludge.
DensaDeg® XRC	• Extreme loading rates. • Compact footprint. • Ability to retrofit existing basins. • Fast start-up time.	• System requires garnet sand as a ballast. • Process increases the degree of operational complexity. • System requires a coagulant.

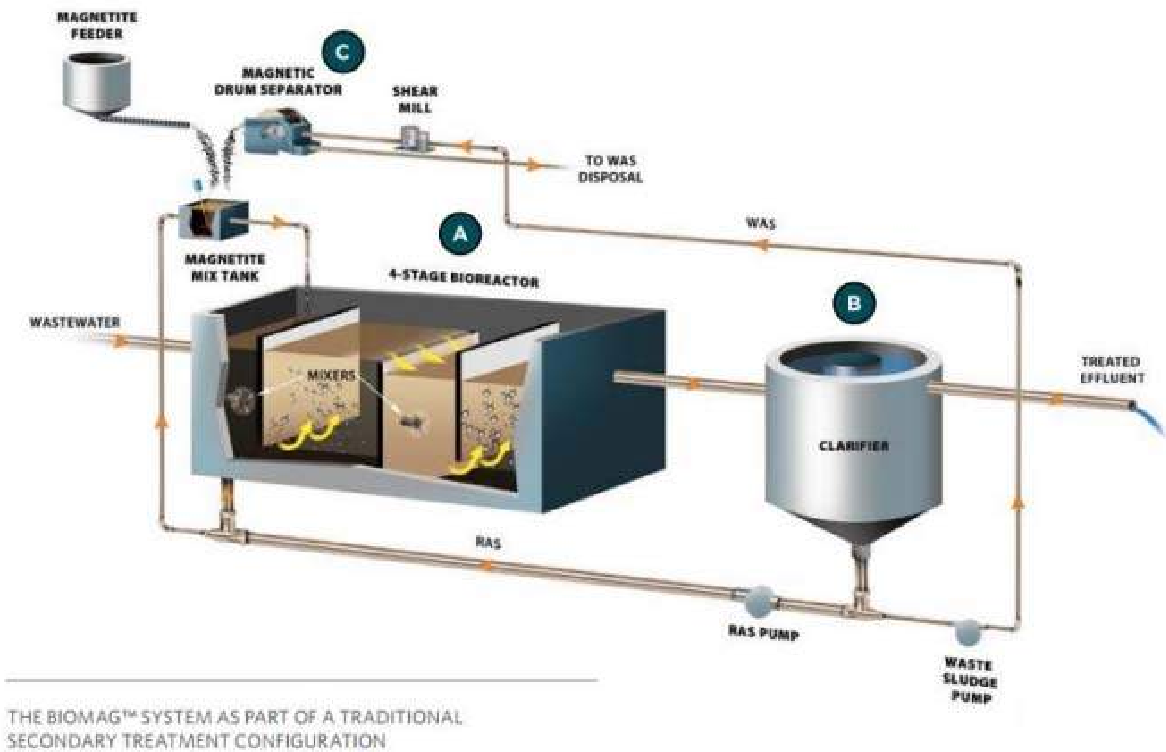
8.4 High-Rate Secondary Treatment Alternatives

For high-rate secondary treatment there are two technologies that can be applied at the CCMUA plant. These are BioMag® and Bio-Actiflo®, which are summarized below.

8.4.1 BioMag®

Components of the BioMag® system are similar to the CoMag® system. The BioMag® system, however, includes the use of mixed liquor that is cultured and supported in the BioMag® reactor. The BioMag® mixed liquor is separate from the base plant mixed liquor. This system uses magnetite to increases the settling characteristics of mixed liquor and reduces the size of the clarifiers. The flow schematic of BioMag® system is shown in **Figure 8-21**.

Figure 8-21: BioMag® Process Flow Diagram



BioMag® includes a 4-stage bioreactor and has a larger footprint than CoMag®. The BioMag® takes time to acclimate and cannot be brought into service rapidly. As such, if this system were used it would remain in service at all times to reduced startup and acclimation issues.

Advantages:

- Reduced secondary clarification footprint
- Increased capacity of treatment

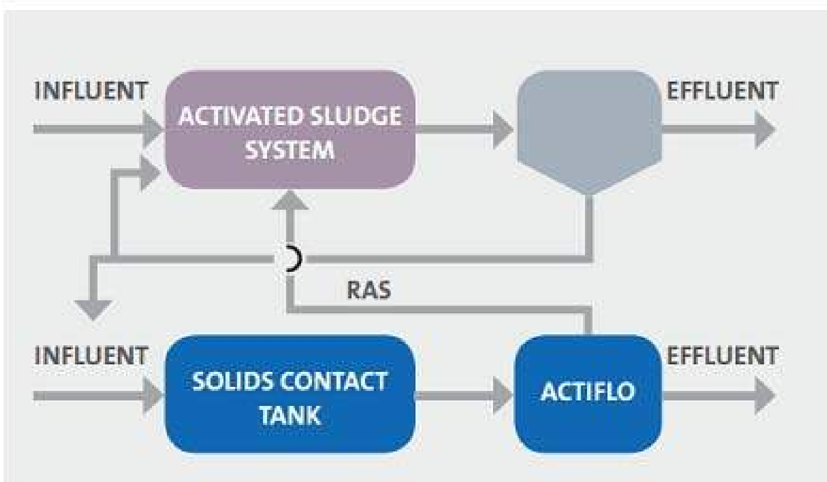
Disadvantages:

- Process increases the degree of operational complexity.
- System requires magnetite.
- System produces additional waste sludge
- Needs to be in operation during dry weather

8.4.2 Bio-Actiflo®

Bio-Actiflo® includes similar treatment process components to the Actiflo® system, but Bio-Actiflo® includes a contact stabilization tank as the first step of the reactor. The stabilization tank borrows mixed liquor wasted from the main treatment plant process to provide contact stabilization where the mixed liquor absorbs portions of the soluble BOD. The raw wastewater and contact stabilized mixed liquor mix then pass through the Bio-Actiflo® system providing rapid TSS removal and enhanced BOD removal. A flow diagram for Bio-Actiflo® working in parallel to the wastewater treatment plant is shown in **Figure 8-22**.

Figure 8-22: Bio-Actiflo® Process Flow Diagram



The contact stabilization tank, or solids contact tank, needs to be large enough to provide 20 minutes of contact time. The system provides a high degree of TSS and BOD removal, but little reduction of nitrogen-ammonia due to the short aeration period in the contact tank. The nitrogen-ammonia removal is not a permit concern at the CCMUA plant. Furthermore, the entire system is relatively complex to bring into service and not well suited for intermittent operations to meet the wet weather flow patterns at the plant.

Advantages:

- Reduced secondary clarification footprint
- Very high settling rate
- Short residence time

Disadvantages:

- Process increases the degree of operational complexity
- System requires ballasted media
- System produces more waste sludge
- Needs to be in operation during dry weather.

A summary comparison of advantages and disadvantages of each of the potential high-rate secondary treatment systems is provided in **Table 8-2**.

Table 8-2: Overall Comparison of High-Rate Secondary Treatment Alternatives

High rate Treatment Approach	Advantages	Disadvantages
BioMag®	• Reduced secondary clarification footprint. • Increased treatment capacity compared to the conventional secondary treatment.	• System requires use of magnetite. • Process increases the degree of operational complexity. • Needs to be in operation during dry weather. • System produces more waste sludge.
Bio-Actiflo®	• Reduced secondary clarification footprint. • Very high settling rate. • Short residence time.	• System requires use of ballasted media. • Process increases the degree of operational complexity. • System requires a coagulant and polymer. • System produces more waste sludge.

8.5 High-Rate Disinfection

High-rate disinfection can be achieved by using either higher concentrations of disinfectant or applying higher mixing intensities, which results in a better dispersion and reduces the required contact time. Further, high-rate disinfection can be achieved by using a stronger disinfectant, like ozone (O_3), peracetic acid ($C_2H_4O_3$), or a high-energy irradiation such as E-Beam and Ultraviolet. Ozone and E-Beam have not been widely used for disinfection of the wet weather flows and will not be considered for the CCMUA Wet Weather Upgrades Project.

Wet weather flows are typically associated with highly variable flowrates and water quality, resulting from a mixture of domestic, industrial, and commercial wastewater and polluted stormwater runoff. In addition, the mixture contains the scoured materials that build up in the wastewater collection system between consecutive storm events. Therefore, the disinfection technology will need to deal with the flow and water quality variability.

The evaluated disinfection technologies include chlorine (sodium hypochlorite), ultraviolet irradiation (UV), and peracetic acid.

8.5.1 Sodium Hypochlorite

The Delaware No. 1 Water Pollution Control Facility currently uses sodium hypochlorite solution for disinfection (chlorination) of the secondary effluent prior to discharge. The disinfection system would need to be expanded to handle the peak wet weather flows above 180 mgd to 240 mgd.

Computations indicate that at 240 mgd flows, the chlorine contact time is reduced from 15 minutes to 12 minutes. A minimum of 15 minutes of contact time is required to achieve disinfection goals based on the Recommended Standards for Wastewater Facilities. Therefore, additional Wet Weather Contact tankage is required to provide a minimum of 15 minutes of contact time for wet weather flows up to 55 mgd.

8.5.2 UV Irradiation

For decades ultraviolet irradiation has been a reliable alternative for the disinfection of wastewater. The primary method for utilizing UV disinfection is to expose wastewater to UV irradiation through a series of lamps. UV disinfection process works because the short wavelength UV light penetrates the cell walls of pathogenic organisms structurally altering their DNA, thus preventing cell replication and function.

UV system offers multiple advantages such as:

- Short contact time
- Insensitivity to pH and temperature
- Small footprint
- No formation of disinfection by-products
- No toxic disinfectant residuals
- No on-site chemical storage

- Flexible dosage control
- Quick start-up
- More effective than chlorine in inactivating viruses, spores and cysts
- Improved safety over other disinfectants

The significant disadvantage is that the UV disinfection efficiency is highly impacted by the transmittance, which is in part a function of suspended solids concentration in the wastewater to be treated. For this reason, UV systems are typically used at wastewater treatment plants that have tertiary treatment, which significantly improves the UV transmittance and disinfection efficiency. Also, capital cost of UV systems can be significantly higher compared to chlorine based systems. Therefore, use of UV system for untreated wastewater is limited and it is not considered a good fit for the CCMUA WPCF.

8.5.3 Peracetic Acid

Peracetic acid (PAA) (chemical formula: CH_3COOOH) has been extensively used in the food industry since the 1980s as a disinfectant. It is a stronger oxidant than sodium hypochlorite. Its strong oxidative properties allows disinfection targets to be achieved at reduced contact times, but creates significant handling challenges.

Peracetic acid is highly corrosive. Specialized, relatively expensive storage and dosing facilities are required that would include features such as all stainless steel tankage and piping, and specialized pumps. There are significant environmental health and safety considerations, including training in managing fires with regard to the quantities applicable for a large facility. PAA tends to off-gas oxygen which has to be managed, because tank headspaces with elevated oxygen concentrations represent a fire risk.

PAA in equilibrium is a mixture of peracetic acid, acetic acid, hydrogen peroxide, and water. The chemical equilibrium is shown below.

$$CH_3CO_2H + H_2O_2 \leftrightarrow CH_3\,CO_3H + H_2O$$

Manufacturers that produce PAA in the US for wastewater disinfection are limited to:

- Solvay, Joliet, IL.
- Enviro-Tech, Arkansas

Alternatively, on-site production of peracetic acid can be considered using acetic acid and hydrogen peroxide, and storing the product for an extended period of time to achieve chemical equilibrium.

Some literature suggests that PAA may breakdown rapidly on its own in the disinfection process without quenching. EPA suggests an accepted residual for PAA of 1 mg/L. Some chronic toxicity studies may be needed to determine the appropriate residual. If quenching is necessary to remove PAA from the effluent before discharging to the environment, sodium bisulfate or any other common chemical used for de-chlorination can be used.

Key disadvantages of peracetic acid are:

- Does not tend to deteriorate or breakdown over time like hypochlorite, so long intermittent storage times is less of an issue.
- PAA experience, although limited, indicates that shorter contact times than sodium hypochlorite are required that results in smaller contact basins. This can be advantageous when site constraints are limiting.

The lack of proven experience of using PAA for disinfecting wastewater effluent is a significant disadvantage. PAA would have to be piloted to determine the required CT and effect. On this basis, PAA is not recommended for the CCMUA Wet Weather Upgrade Project.

8.6 Overall Comparison of High-Rate Disinfection Alternatives

A summary comparison of advantages and disadvantages of each of the potential high-rate disinfection systems is provided in **Table 8-3**.

Table 8-3: Overall Comparison of Disinfection Alternatives

High rate Treatment Approach	Advantages	Disadvantages
Sodium Hypochlorite	• Currently in use at WPCF. • Proven and approvable for CSO disinfection.	• Potential toxic byproducts. • Relatively long contact time. • May require dechlorination quenching system.
Ultraviolet	• Requires less space. • No byproducts.	• New Technology for WPCF. • Costly and complex.
Peracetic Acid	• Stronger oxidant than hypochlorite. • Shorter CTs possible. • Does not deteriorate over time. • No byproducts. • Might not need quenching.	• May require approval by NJDEP. • Special handling require due to potential O_2 emissions.

8.7 Discussion of Options

Options C-1, C-2, C-3, and C-4 are considered to be least robust or less robust approaches for conveying flow up to 240 mgd through the plant, because all plant unit processes would need to be in service (no units out of service for inspection, repair, or rehabilitation). The lack of firm capacity in the system leaves the facility vulnerable to non-compliance with the operating permits. If a wet weather water quality standard, which is less stringent than the current permit, could be established with the regulatory agency this would make these options viable. Under a wet weather standard of operation, the capacity of the plant can be expanded for less capital costs.

For example, the four raw wastewater pumps will lift 240 mgd into the plant if all are in service, but if one is out of service for inspection, maintenance or repair, only 180 mgd of flow can be achieved. Another example is that three (3) grit removal tanks can handle 150 mgd, while collecting 95 percent of 100 mesh grit or coarser grit material. At 240 mgd the grit removal performance in the three tanks will suffer slightly but they would be within their hydraulic capacity limits. Options C-1 through C-4 would eliminate the need for additional grit removal capacity, but more grit may accumulate in the downstream process such as digesters, which could create an ongoing maintenance issue.

Also, assuming that the 240 mgd wet weather event is of a short duration, less than a few hours, with low influent characteristic concentrations; the solids handling facilities may not be significantly impacted by additional solids if the enhanced primary treatment can be operated only when flows exceed 180 mgd. Existing sludge storage facilities may handle the additional sludge resulting from a 240 mgd event. If enhanced primary treatment needs to operate at lower plant flows to ensure the system will perform during wet weather events then additional solids handling systems are warranted.

Another cost reduction consideration would be to disinfect primary treatment flows above 180 mgd through a separate outfall by using the existing 72-inch secondary treatment bypass to convey flows above 180 mgd around the primary tanks and out to the river through the twin existing 72-inch outfalls. This flow would be disinfected using the existing underutilized sodium hypochlorite disinfection system in the odor control building. In some options (i.e., C-1 & C-2) the flows above 180 mgd would not receive primary or secondary treatment, the water quality is variable and unlikely to meet the water quality discharge permit without a wet weather permit that would relax the discharge requirements during the storm event period. Further, if there is a separate outfall for the flow this may require additional sampling and reporting by CCMUA to NJDEP.

Options C-5, D-1, and D-2 would increase the firm treatment capacity of the Plant up to 240 MGD with considerably higher capital costs. These options offer appropriate redundancy to the level of the existing treatment processes and ensure the capability of the plant to treat wet weather flows up to 240 MGD.

While there is merit to maximizing the capacity of the infrastructure, there are operation and maintenance needs to be considered. For example, having at least one of each unit process to be out of service for inspection, maintenance or repair at any time. To assume all unit process systems are fully available during any storm event may be over promising performance and under delivering in results.

8.8 Summary of Listed Options

The “All Flow Treatment” option with full secondary treatment is relatively complex to bring into service and not well suited for the intermittent wet weather operations required at the plant. These systems would need to be in service at all times, which is a major change in plant operations and would require a major expense for implementation. Due to the high capital cost requirement, the options for All Flow Treatment was not considered further.

After assessing the available options for expanding the process capabilities at the CCMUA Delaware No. 1 WPCF, the following were selected as practical options for implementation or as probable least-cost options for further detailed evaluation.

Treatment Options:

- Option C-1: Standard-rate or high-rate disinfection with no preliminary treatment or raw sewage pumping upgrades.
- Option C-2: Screening to 0.50 inches and standard-rate or high-rate disinfection with no preliminary treatment or raw sewage pumping upgrades.
- Option C-3: Screening, high-rate primary treatment in the area north of the existing aeration/final tanks, and standard-rate or high-rate disinfection with no preliminary treatment or raw sewage pumping upgrades.
- Option C-4: Screening, high-rate primary treatment in the area north of the existing aeration/final tanks, standard-rate or high-rate disinfection, and contact tanks with no preliminary treatment or raw sewage pumping upgrades.
- Option C-5: Screening, high-rate primary treatment in the area north of the existing aeration/final tanks, standard-rate or high-rate disinfection, and contact tanks with preliminary treatment or raw sewage pumping upgrades designed to provide a firm capacity of 240 MGD.
- Option D-1: Using high-rate primary treatment system in the existing Primary Tanks.
- Option D-2: Using Chemically Enhanced Primary Treatment in the existing Primary Tanks.

Disinfection Upgrade:

- Standard-rate or high-rate chlorination

Section 9 Evaluation of Phase 2 Alternatives

The selected options identified in the previous sections of the Report are further evaluated as systems for potential use for the CCMUA Delaware No. 1 WPCF for Wet Weather Upgrades Project. From an engineering standpoint, all the high-rate primary treatment systems by the different technology manufacturers introduced in the previous section are capable of providing the required high-rate primary treatment. In this section, one high-rate technology (DensaDeg® XRC) is selected to represent the high-rate treatment systems for analysis. If high-rate primary treatment is considered for design and construction, further investigations are recommended to choose the best high-rate primary treatment system among the technologies that were reviewed in the previous section based on the WPCF's needs, preferences, and costs.

9.1 High-Rate Primary Treatment Alternatives

Based upon the CCMUA's process needs and the WPCF's limitations and constraints, multiple alternatives were initially considered for the high-rate primary treatment. Among all the options that were listed in the previous section, four options utilize the application of High-Rate Primary Treatment and one option converts the conventional primary treatment to Chemically Enhanced Primary Treatment. The options for further evaluation are as follows:

Options C-3, C-4, and C-5:

- High-Rate Primary Treatment using DensaDeg® XRC in area north of Aeration Tanks

Options D-1:

- High-Rate Primary Treatment using DensaDeg® XRC in existing Primary Sedimentation Tanks

Options D-2:

- Chemically Enhanced Primary Treatment in existing Primary Sedimentation Tanks

Additional details of these listed options along with advantages and disadvantages of each is provided below.

9.1.1 High-Rate Primary Treatment using DensaDeg® XRC

The basic design and operating principles of the high-rate primary treatment systems are similar in concept. While each system provides significantly enhanced primary treatment performance, after discussion with high-rate primary treatment manufacturers, the DensaDeg® XRC was selected as the system to study in more detail for enhanced primary sedimentation. Conventional primary treatment are based on a surface overflow rates of 0.50 to 0.90 gpm/sq. ft., while DensaDeg® XRC offers 40 to 60 gpm/sq. ft. Consequentially, a smaller footprint is required to treat the wet weather flow with high-rate treatment systems compared to the conventional primary sedimentation tanks.

1) **Options C-3, C-4, and C-5: High-Rate Primary Treatment using DensaDeg® XRC in the Area North of Aeration Tank**

A DensaDeg® XRC system with a capacity of 55 mgd would be constructed on the north side of the Aeration Basins. The existing Primary Sedimentation Tanks would be operated for dry and wet weather flows up to 180 mgd. A new bypass line will be used to direct the flows higher than 180 mgd from the grit removal effluent to the new high-rate treatment units. It should be noted that in Options C-3 and C-4, this scenario can occur only when all four of the raw sewage pumps are in service. Option C-5 provides redundancy due to the upgraded preliminary treatment facility and increases the firm treatment capacity of the Plant up to 240 MGD. Therefore, the aforementioned limitation does not interfere with the treatment of wet weather flows in Option C-5.

2) **Option D-1: High-Rate Primary Treatment using DensaDeg® XRC in the Existing Primary Tanks**

The DensaDeg® XRC alternative requires replacing two Primary Sedimentation Tanks with four DensaDeg® XRC units to treat up to 92 MGD. Each existing primary sedimentation tank is 50 ft. wide, 207 ft. long, and 11ft. deep. Conceptual design shows that four DensaDeg® XRC units would occupy a 50 ft. wide and 68 ft. long space and can fit inside one primary sedimentation tank, however, two tanks were assumed to be required to provide sufficient space for construction and operation of the high rate treatment systems. A minimum depth of 22 feet is required for the high rate treatment units. With the existing tanks at 11 feet deep, special sheeting and shoring construction techniques will need to be employed to place the high-rate system within the tank footprint. The remaining space will be allocated to the ballast media and coagulant storage tanks necessary for the system.

Primary Sedimentation Tanks #1 and #2 were initially considered for the DensaDeg® XRC system, but because of limited space and potential construction issues, Tanks #9 and #10 are preferred for outfitting the new DensaDeg® XRC system. The remaining tanks (#1 to #8) will provide conventional primary treatment for flows up to 148 mgd, for an optimum primary treatment capacity of approximately 240 MGD.

9.1.2 Chemically Enhanced Primary Treatment

The surface overflow rate of a CEPT system is higher than a conventional primary sedimentation system due to high settling characteristics induced with the addition of flocculent forming chemicals with a high specific gravity. The differences between conventional primary treatment and Chemically Enhanced Primary Treatment are summarized in **Table 9-1**. By observation, CEPT offers higher BOD, TSS, and phosphorous removal efficiencies.

Table 9-1: Removal Efficiencies of CEPT vs. Conventional Primary Treatment

Parameters	Removal efficiency of CEPT	Removal efficiency of Conventional Primary Treatment
TSS	60-90%	50-70%
COD or BOD	40-70%	25-40%
Phosphorous	70-90%	5-10%
Bacteria loading	80-90%	50-60%

1) Option D-2: Chemically Enhanced Primary Treatment in the Existing Primary Sedimentation Tanks.

The existing Primary Sedimentation Tanks offer a nominal treatment capacity of 180 mgd with 10 tanks in service. By converting each primary tank to CEPT, the SOR can be increased by two times and each tank can be operated effectively at 37 MGD instead of 18.5 MGD in the existing mode. To increase the primary treatment capacity up to 240 MGD (from 180 MGD), an additional 55 MGD of treatment capacity is required, which can be achieved by converting 3 primary tanks (3 tanks × 18.5 MGD additional capacity/tank = 55.5 MGD) to CEPT. In this scenario, the existing Primary Sedimentation Tanks would be capable of treating 129.5 MGD (7 tanks × 18.5 MGD /tank = 129 MGD) with seven tanks in operation and the other three Chemically Enhanced Primary Sedimentation Tanks would provide treatment capacity for flows up to 111 MGD (3 tanks × 37 MGD /tank = 111 MGD), for a total of 240 mgd.

The existing equipment in the primary tanks needs to be further investigated to evaluate its compatibility with a CEPT environment due to the presence of chemicals. Also, this alternative produces higher primary sludge quantities as compared to the existing conditions. Therefore, further investigation is needed to understand the capability of the existing equipment to handle the higher settled sludge quantities.

Iron salts are the most common chemicals used as coagulant due to low cost and availability. Aluminum salts and lime are also used for this purpose. Lime addition produces more sludge compared to iron and aluminum salts. Further, lime is more difficult to store and handle.

9.2 Chlorination

The NJDEP accepts a chlorine contact time of 15 minutes or higher for wastewater treatment disinfection and the recommended wet weather upgrades can provide a minimum of 15 minutes contact time. However, it is advised to consult with NJDEP to explore the possibility of using high-rate chlorination to reduce capital cost investment. All the options that were presented in the previous sections, can be implemented with standard-rate or high-rate chlorination. In standard-rate chlorination, a separate chlorine contact tank will provide 15 minutes of contact time with the appropriate concentrations of chlorine residual. With high-rate chlorination, a high chlorine dosage, high mixing intensity and less contact time will achieve the same treatment goals as the standard-rate chlorination with 15 minutes of contact. If this approach is accepted, a smaller conduit will be used, which would reduce capital costs.

The calculations presented in **Table 9-2** show that not enough contact time can be provided for 240 mgd in the existing chlorine contact basins, only 12 minutes of contact time can be achieved. Further, the proposed hydraulic modifications in the previous sections will lower the elevation of effluent over the weirs to increase the hydraulic capacity through the plant; however, the lower effluent elevation over the weirs reduces the water elevation at the contact tank and, therefore, reduces the contact time.

Table 9-2: Chlorine Contact Tank Evaluation with Varying Flowrates

General Data	Units	Existing Facilities			Existing Facilities with Hydraulic Modifications			Phase II Flow
WPCF Influent WW Flow	mgd	57	150	185	57	150	185	240
Delaware River HGL	ft.	15.75	16.87	17.45	15.75	16.33	16.65	16.93
Chlorine Contact Tank Height	ft.	20	20	20	20	20	20	20
Is Tank Overflowing?		No	No	No	No	No	No	No
Channel Width	ft.	13.25	13.25	13.25	13.25	13.25	13.25	13.25
Flow Depth	ft.	14.75	14.87	16.45	14.75	15.33	15.65	15.93
Total Head Loss	ft.	0.0006	0.0041	0.005	0.0006	0.0039	0.006	0.009
HGL @ CCT Inlet	ft.	15.75	16.87	17.45	15.75	16.33	16.66	16.94
Volume of CCT	cf	160193	160193	160193	160193	160193	160193	160193
CCT Contact Time	min	46	18	16	46	18	15	12
Required Contact Time	min	15	15	15	15	15	15	15
Does CCT Retention Time Meet Requirement?		Yes	Yes	Yes	Yes	Yes	Yes	No

As mentioned previously, a new 55 mgd bypass conduit will be used to discharge the flow to the Delaware River through a new wet weather outfall. For the standard-rate chlorination, the conduit should be designed to provide a minimum of 15 minutes contact time for the maximum wet weather flow of 55 mgd. It is recommended to start chlorination at the beginning of the conduit to provide the required contact time. The length of the conduit is estimated to be around 700 ft. The preliminary calculations suggest that 15 minute contact time can be achieved by a conduit with 10 ft. width and 12 ft. depth.

For high-rate chlorination, a smaller conduit can be used to bypass the secondary treatment, however, a higher chlorine concentration would be used to achieve the treatment goals.

9.3 Solids Handling Improvement

Preliminary calculations show that the sludge production rates for primary treatment will be increased relatively with both CEPT and the high-rate primary alternatives. Preliminary projections of the sludge production rate for the two alternatives are summarized in **Table 9-3**. According to this table**,** the high-rate primary treatment alternative would produce more sludge compared to CEPT for the same flowrate. Based upon the sludge processing documents received from CCMUA, sludge production rates were projected by increasing the plant wet weather capacity to 240 mgd.

Table 9-3: Projected Sludge Production Rates

Scenario	Max Wet Weather Flowrate, MGD	Primary Sludge		WAS	
		Flow, MGD	Solids Content	Flow, MGD	Solids Content
Existing (maximum month)	150	0.29	2.4%	0.63	0.8%
Projected Maximum Month sludge production with Conventional Primary Treatment	240	0.33	3.0%	0.84	0.8%
Projected sludge production with High-Rate Primary Treatment	240	0.51	3.0%	0.84	0.8%
Projected sludge production with CEPT	240	0.48	3.0%	0.84	0.8%

The primary sludge quantities are increased for both CEPT and high-rate primary treatment alternatives. However, this does not necessarily mean that the Solids Handling Facility would require an upgrade. The duration and frequency of wet weather events are two important parameters in determining whether the Solids Handling Facility requires an upgrade. At the time of preparing this report, the data regarding the duration and frequency of a 240 mgd wet weather flow was not available. Therefore, it is recommended that solids mass balance be performed once the data is available to determine whether solids handling improvements are necessary. Solids from infrequent storm events with wet weather flows up to 240 mgd are expected to be treated with the existing facility. With removal of TSS and BOD with either alternative, higher waste activated sludge (WAS) flowrates are not anticipated, so waste activated sludge flow and solids content were assumed to be constant. The existing gravity belt thickeners should be able to handle the WAS flows. The existing screw thickeners are being used for 16 hrs. /day and should be able to handle a higher flow rate if they are operated for 24 hrs. /day when required. Further studies are needed in the future to determine the condition of the anaerobic digesters and to calculate the new solids retention time in the digesters with a new maximum month flow.

9.4 Cost Evaluation of Options

This section provides a summary of the conceptual level costs for the various options that were listed and discussed in the previous sections of the Report. The costs discussion presented herein include all identifiable components of each unit process such as equipment, electrical systems such as MCCs, HVAC, and the required site work and excavation. A 20% markup for contractor overhead and profit is added to the estimated construction costs. Due to the nature of early phase cost estimation and highly variable or unknown construction items, a contingency factor of 35% was included in the engineer's opinion of probable costs.

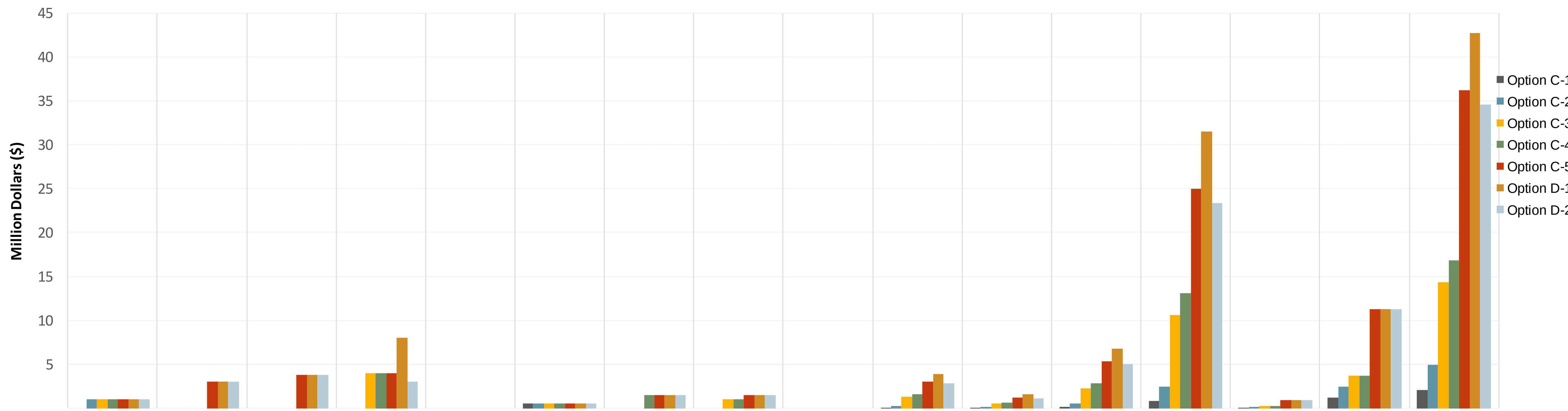
A summary of the conceptual level opinion of probable cost for the options is summarized in **Table 9-4**. In this table, all unit processes in the WPCF are listed with their estimated construction costs for each proposed option. For example in Option C-1, the WPCF will only provide proper disinfection to the wet weather flows for up to 55 mgd in a new wet weather bypass conduit as described previously. Therefore, no estimated construction cost is listed for each one of the listed unit processes except for disinfection. In Option C-2, an additional cost is listed for the screenings compared to Option C-1 since this option proposes an upgrade to the existing screenings units as well.

Raw sewage pumping capacity and grit removal tanks will be upgraded in Options C-5, D-1, and D-2 only. Primary treatment system will be upgraded in Options C-3, C-4, C-5, D-1, and D-2. Standard-rate or high-rate disinfection can be implemented in all the proposed options. In standard-rate chlorination, the new bypass conduit will be designed large enough to provide a minimum contact time of 15 minutes for wet weather flow of 55 mgd. In high-rate chlorination, a smaller conduit can be used with a higher residual chlorine concentration at a reduced contact time to achieve the same treatment goals.

The implementation of high-rate chlorination may require a new permit from NJDEP; however it eliminates the need for a larger bypass conduit or a contact tank, which reduces the construction costs significantly. For instance, Option C-1 only lists the cost for disinfection but not the contact tank, which means that this option proposes the high-rate chlorination to reduce the construction cost. If the NJDEP does not grant a new permit for high-rate chlorination for the wet weather flows, an additional cost should be added to Option C-1 to account for a new contact tank. The same argument can be used for Options C-4, C-5, D-1, and D-2 that have a cost for a contact tank. In other words, the cost listed in these options can be saved or eliminated if the WPCF receives a permit for high-rate chlorination.

The life cycle cost of the options do not include engineering costs or CCMUA's administrative costs.

Figure 9-4: Life Cycle Costs of Conceptual Options



Options	Screenings Upgrades	Pumps Upgrades	Grit Removal Upgrades	Primary Treatment Upgrades	Secondary Treatment Upgrades	Disinfection	Contact Tank	Electrical, Mechanical, and Hydraulic Upgrades	Sludge Handling Facility Upgrades	Overhead & Profit (20%)	Tax (8.1%)	Contingency (35%)	Probable Construction Cost*	Annual O&M	Present Worth of O&M	Total Life Cycle Cost (Total)
Option C-1	-	-	-	-	-	$ 500,000	-	-	-	$ 100,000	$ 41,000	$ 175,000	$ 816,000	$ 100,000	$ 1,250,000	$ 2,066,000
Option C-2	$ 1,000,000	-	-	-	-	$ 500,000	-	-	-	$ 300,000	$ 122,000	$ 525,000	$ 2,447,000	$ 200,000	$ 2,500,000	$ 4,947,000
Option C-3	$ 1,000,000	-	-	$ 4,000,000	-	$ 500,000	-	$ 1,000,000	-	$ 1,300,000	$ 527,000	$ 2,275,000	$ 10,602,000	$ 300,000	$ 3,750,000	$ 14,352,000
Option C-4	$ 1,000,000	-	-	$ 4,000,000	-	$ 500,000	$ 1,500,000	$ 1,000,000	-	$ 1,600,000	$ 648,000	$ 2,800,000	$ 13,048,000	$ 300,000	$ 3,750,000	$ 16,798,000
Option C-5	$ 1,000,000	$ 3,000,000	$ 3,800,000	$ 4,000,000	-	$ 500,000	$ 1,500,000	$ 1,500,000	-	$ 3,060,000	$ 1,239,000	$ 5,355,000	$ 24,954,000	$ 900,000	$ 11,250,000	$ 36,204,000
Option D-1	$ 1,000,000	$ 3,000,000	$ 3,800,000	$ 8,000,000	-	$ 500,000	$ 1,500,000	$ 1,500,000	-	$ 3,860,000	$ 1,563,000	$ 6,755,000	$ 31,478,000	$ 900,000	$ 11,250,000	$ 42,728,000
Option D-2	$ 1,000,000	$ 3,000,000	$ 3,800,000	$ 3,000,000	-	$ 500,000	$ 1,500,000	$ 1,500,000	-	$ 2,860,000	$ 1,158,000	$ 5,005,000	$ 23,323,000	$ 900,000	$ 11,250,000	$ 34,573,000

Section 10 Phase 2 (Plant Capacity Expansion) Recommendations

The wet weather treatment plant expansion options offer different levels of treatment and system reliability at different capital and life cycle costs. Further, the options can be classified as robust, less robust and least robust. Although the least robust options (C-1 and C-2) offer the lowest construction costs and ultimately the lowest present worth, they also come with the highest risk of permit non-compliance during high wet weather flow events unless a wet weather permit is granted by the regulatory agency. Therefore, the capital, annual, and life cycle costs of all the viable options are provided.

With the wet weather system treatment plant expansion information, CCMUA will be able to select the best option by reviewing the information presented in this Report in combination with the collection system improvement options provided under a separate project to develop the overall program needs including costs and corresponding risks.

For Delaware No. 1 WPCF to increase the firm capacity to 240 mgd to treat flow peaks during wet weather storm events and provide a robust treatment system, the capacity of influent pump station, screens, and grit removal systems will need to be increased from 180 mgd to 240 mgd. Options C-5, D-1, and D-2 satisfies the 240 mgd firm capacity (robust) requirement. For Options D-1 and D-2 the hydraulic capacity into and from the primary treatment system will need to be increased and a secondary bypass and outfall will need to be provided.

The less robust options (C-3 and C-4) will use new high-rate treatment systems to treat up to 55 mgd of the wet weather flows and a total of 240 mgd, if all the process units are in service. The significantly lower construction costs for these options comes with a higher level of performance risk. As mentioned in the Report, if one of the four influent pumps is out of service for inspection, maintenance or repair, the 240 mgd flow cannot be achieved. However, it should be noted under these options that the unit processes can be expanded in the future to increase the firm capacity of the plant up to 240 mgd, as required.

Options C-1 and C-2 are considered the least robust options with only partial preliminary treatment and disinfection. These options also have the highest risk of permit non-compliance among all of the options considered.

Options C-1, C-2, C-3, and C-4 would provide treatment for wet weather flows up to 240 mgd with all the unit process systems in service, but are rated at a firm capacity of 180 mgd. New high-rate primary treatment is included in Options C-3 and C-4. Estimated capital costs for the options are $816,000 for Option C-1, $2,447,000 for Option C-2, $10,602,000 for Option C-3, and $13,048,000 for Option C-4, all of which excludes engineering costs and CCMUA's administrative costs. Each of these options should be accompanied with an understanding with NJDEP that there may be permit non-compliance during certain wet weather events, if all the unit processes are not in service. An alternative to non-compliance would be a less stringent permit requirement be made available by NJDEP during and shortly after a wet weather event.

The robust Options C-5, D-1, and D-2 provide a firm treatment capacity of 240 mgd and the estimated capital costs, excluding engineering and CCMUA administrative costs, are $24,954,000, $31,478,000, and $23,323,000, respectively.

With the Wet Weather Plant Bypass Options C-1 to C-5, the wet weather treatment capacity expansion implementation can proceed in steps to achieve higher levels of performance and reliability by beginning with Options C-1 and proceeding to C-5, or beginning from C-2 and proceeding to C-5, or beginning from C-3 and proceeding to C-5, and so forth.

The various viable options to increase the wet weather treatment capacity of the Delaware No. 1 WPCF during wet weather events are summarized in this Concept Study Report, rather than recommending a single option, because collection system improvements will need to be weighed with the treatment plant capacity expansion options to determine a final course of action by CCMUA. Some treatment plant capacity expansion options may provide a better overall fit for CCMUA than others once the overall wet weather improvements program is evaluated. A brief wet weather system description along with the capital, annual and life cycle cost for each option is provided in Table 10-1.

Table 10-1: Wet Weather Options Summary

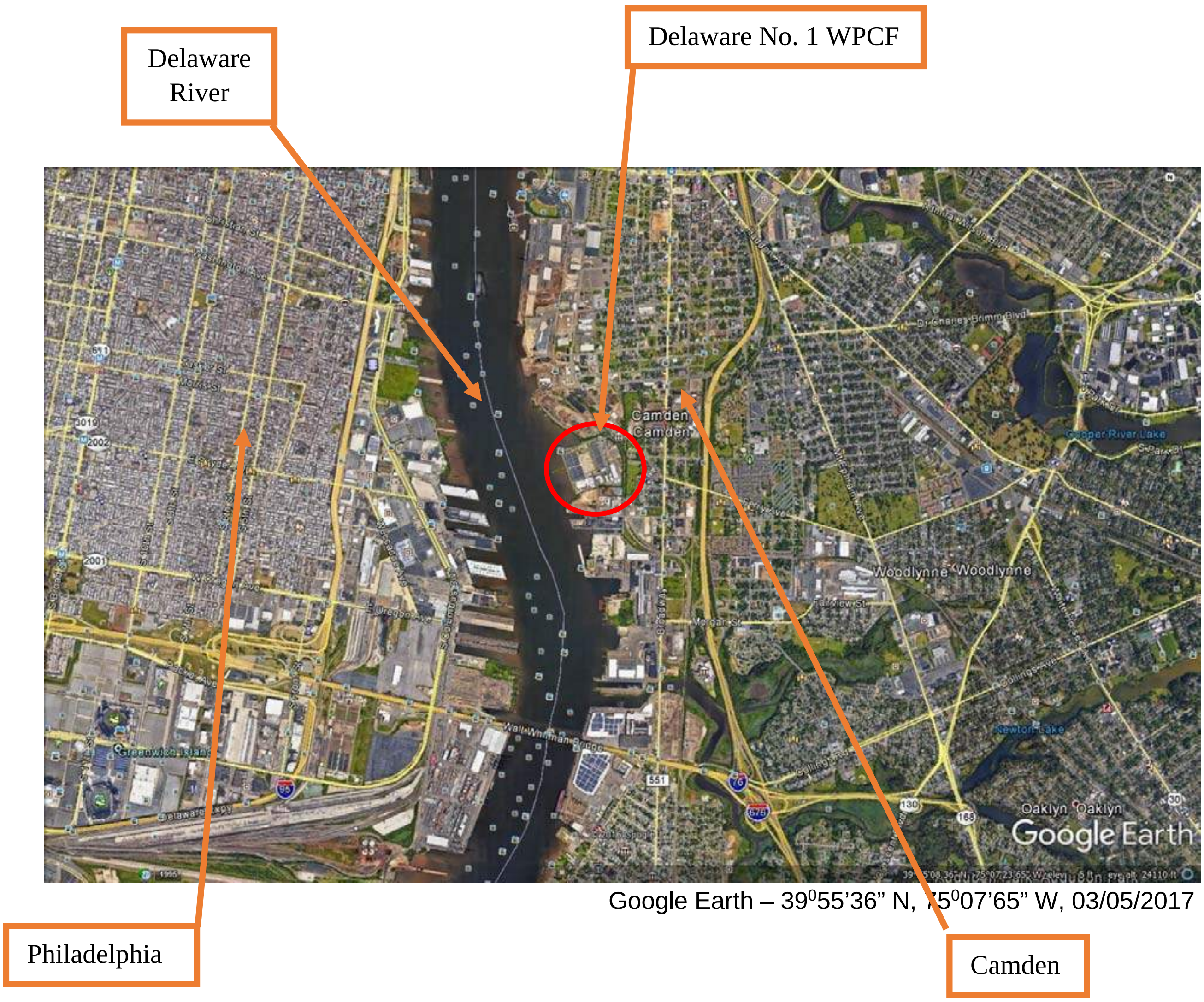
	Firm Capacity 180 MGD*			
	Bypass Primary & Secondary Treatment			
Options	**C-1**	**C-2**	**C-3**	**C-4**
System Classification	**Least Robust**	**Least Robust**	**Less Robust**	**Less Robust**
Wet Weather Treatment Systems Description	• 55 mgd Bypass of Primary and Secondary Treatment through New Wet Weather Outfall • Standard or High-Rate Disinfection	• 55 mgd Bypass of Primary and Secondary Treatment through New Wet Weather Outfall • Upgrade Screenings • Standard or High-Rate Disinfection	• 55 mgd Bypass of Primary and Secondary Treatment through New Wet Weather Outfall • Upgrade Screenings • Add 55 MGD High-Rate Primary Treatment • Standard or High-Rate Disinfection	• 55 mgd Bypass of Primary and Secondary Treatment through New Wet Weather Outfall • Upgrade Screenings • Add 55 MGD High-Rate Primary Treatment • Standard or High-Rate Disinfection • Add Disinfection Capacity
Capital Cost**	$816,000	$2,447,000	$10,602,000	$13,048,000
Annual Cost	$100,000	$200,000	$300,000	$300,000
Life Cycle Cost	$2,066,000	$4,947,000	$14,352,000	$16,798,000

	Firm Capacity 240 MGD		
	Bypass Primary & Secondary Treatment	Bypass Secondary Treatment	
Options	**C-5**	**D-1**	**D-2**
System Classification	**Robust**	**Robust**	**Robust**
Wet Weather Treatment Systems Description	• 55 mgd Bypass of Primary and Secondary Treatment through New Wet Weather Outfall • Expand Preliminary Treatment • Add 55 MGD High-Rate Primary Treatment • Standard or High-Rate Disinfection • Add Disinfection Capacity	• 55 mgd Bypass of Secondary Treatment through New Wet Weather Outfall • Expand Preliminary Treatment • Add 92 MGD High Rate Primary Treatment in Exist. PSTs • Standard or High-Rate Disinfection • Add Disinfection Capacity	• 55 mgd Bypass of Secondary Treatment through New Wet Weather Outfall • Expand Preliminary Treatment • Add Chemically Enhanced Primary Treatment in Exist. PSTs • Standard or High-Rate Disinfection • Add Disinfection Capacity
Capital Cost**	$24,954,000	$31,478,000	$23,323,000
Annual Cost	$900,000	$900,000	$900,000
Life Cycle Cost	$36,204,000	$42,728,000	$34,573,000

Notes: * Plant can treat 240 MGD if all components of each unit process are available for service.
** Excludes engineering and CCMUA's administration fees

APPENDIX A
Vicinity Map

Vicinity Map



Google Earth – 39^{0}55’36” N, 75^{0}07’65” W, 03/05/2017

APPENDIX B
NOAA Tidal Station Data

Philadelphia, PA - Station ID: 8545240

NOAA Data at Time of Field Visit on December 6, 2016:

Date Time	Water Level	
12/5/2016 15:24	2.119	
12/5/2016 15:30	2.031	
12/5/2016 15:36	1.952	
12/5/2016 15:42	1.867	
12/5/2016 15:48	1.788	**Average Water Level**
12/5/2016 15:54	1.709	1.6
12/5/2016 16:00	1.631	**Median Water Level**
12/5/2016 16:06	1.558	1.56
12/5/2016 16:12	1.499	**High Water Level**
12/5/2016 16:18	1.444	2.03
12/5/2016 16:24	1.388	**Low Water Level**
12/5/2016 16:30	1.345	1.296
12/5/2016 16:36	1.316	
12/5/2016 16:42	1.296	
12/5/2016 16:48	1.302	

APPENDIX C
Output Tables from Hydraulic Model at Varying Flow Rates and Tide Levels (without Plant Modifications)

CCMUA
Model Predictions for Delaware River Elevation = 4.0'
Without Modifications

ID #	Location	Top of Tank Elev.	Weir Elev.	Model Prediction at 57 MGD	Model Prediction at 75 MGD	Model Prediction at 150 MGD	Model Prediction at 185 MGD	Model Prediction at 220 MGD
	Grit tanks							
1	Influent Channel	34.08		26.48	26.57	27.24	27.29	28.73
2A	Grit Tank Surface	34.08	26.10	26.47	26.54	27.15	27.15	28.59
2	Effluent Channel	34.08		24.97	25.18	26.45	27.15	27.69
	Primary Tanks							
3	West Influent Channel	26.65		24.87	25.02	25.95	26.48	26.82
3A	East Influent Channel	26.65		24.87	25.02	25.95	26.48	26.82
4	Influent Channel Near Tank #10	26.65	24.58/ 24.62	24.76	24.85	25.40	25.74	25.82
5A	PST Tank #10 Water Surface	26.65		24.65	24.66	24.71	24.73	25.15
5	South Old Effluent Channel	26.65		23.33	23.56	24.12	24.35	24.97
6A	PST Tank #1 Water Surface	26.65		24.65	24.66	24.71	24.73	25.15
6	North Old Effluent Channel	26.65		23.33	23.56	24.12	24.35	24.97
16	North New Effluent Channel	26.65		22.55	22.62	22.93	23.09	24.51
17	South New Effluent Channel	26.65		22.55	22.62	22.93	23.09	24.51
	Aeration Tanks							
7	North Splitter Box Surface	26.00	22.25	22.52	22.57	22.76	22.84	24.20
7A	North Splitter Box Trough	26.00		19.54	19.63	20.44	21.30	24.20
8	South Splitter Box Surface	26.00		22.52	22.57	22.76	22.84	24.20
8A	South Splitter Box Trough	26.00		19.54	19.63	20.44	21.30	24.20
	Aeration Tanks	26.00	18.50	18.69	18.73	18.86	19.18	21.40
10A	North Effluent Channel	20.00		17.65	17.74	18.29	18.76	21.40
9A	South Effluent Channel	20.00		17.65	17.74	18.29	18.76	21.40
	Final Tank Influent Channel							
10	North Influent Channel	20.00		17.61	17.68	18.08	18.49	21.10
9	South Influent Channel	20.00		17.61	17.68	18.08	18.49	21.10
	Final Sedimentation Tanks							
11	North Effluent Trough	20.00		15.96	16.03	16.77	17.59	20.27
11A	North FST Water Surface	20.00	17.50	17.60	17.66	18.00	18.37	21.01
11B	North Effluent Channel	20.00		15.80	16.01	16.77	17.59	20.27
12	South Effluent Trough	20.00		15.96	16.03	16.77	17.59	20.27
12A	South FST Water Surface	20.00	17.50	17.60	17.66	18.00	18.37	21.01
12B	South Effluent Channel	20.00		15.80	16.01	16.77	17.59	20.27
	Chlorine Contact Tanks							
13	North Contact Tank Surface	20.00	14.84	15.75	15.93	16.58	17.37	20.08
14	North CT Effluent Below Weir	20.00		5.08	5.87	11.47	15.37	20.08
15	South Contact Tank Surface	20.00	14.84	15.75	15.93	16.58	17.37	20.08
14A	South CT Effluent Below Weir	20.00		5.08	5.87	11.47	15.37	20.08
	Outfall Well Chamber							
16	Outfall Well Chamber	16.00	-	4.70	5.22	8.88	11.42	14.50

Note:

1. The cells highlighted in red indicate processes with overflow potential.
2. The cells highlighted in blue indicate processes with submerged weir potential.

CCMUA
Model Predictions for Delaware River Elevation = 5.5'
Without Modifications

ID #	Location	Top of Tank Elev.	Weir Elev.	Model Prediction at 57 MGD	Model Prediction at 75 MGD	Model Prediction at 150 MGD	Model Prediction at 185 MGD	Model Prediction at 220 MGD
	Grit tanks							
1	Influent Channel	34.08		26.48	26.57	27.24	28.06	29.14
2A	Grit Tank Surface	34.08	26.10	26.47	26.54	27.15	27.95	29.01
2	Effluent Channel	34.08		24.97	25.18	26.45	27.15	28.11
	Primary Tanks							
3	West Influent Channel	26.65		24.87	25.02	25.95	26.48	27.32
3A	East Influent Channel	26.65		24.87	25.02	25.95	26.48	27.32
4	Influent Channel Near Tank #10	26.65	24.58/ 24.62	24.76	24.85	25.40	25.74	26.41
5A	PST Tank #10 Water Surface	26.65		24.65	24.66	24.71	24.73	26.20
5	South Old Effluent Channel	26.65		23.33	23.56	24.12	24.35	26.16
6A	PST Tank #1 Water Surface	26.65		24.65	24.66	24.71	24.73	26.20
6	North Old Effluent Channel	26.65		23.33	23.56	24.12	24.35	26.16
16	North New Effluent Channel	26.65		22.55	22.62	22.93	23.09	25.91
17	South New Effluent Channel	26.65		22.55	22.62	22.93	23.09	25.91
	Aeration Tanks							
7	North Splitter Box Surface	26.00	22.25	22.52	22.57	22.76	22.84	25.63
7A	North Splitter Box Trough	26.00		19.54	19.63	20.44	21.19	25.63
8	South Splitter Box Surface	26.00		22.52	22.57	22.76	22.84	25.63
8A	South Splitter Box Trough	26.00		19.54	19.63	20.44	21.19	25.63
	Aeration Tanks	26.00	18.50	18.69	18.73	18.86	19.05	22.85
10A	North Effluent Channel	20.00		17.65	17.74	18.29	18.63	22.85
9A	South Effluent Channel	20.00		17.65	17.74	18.29	18.63	22.85
	Final Tank Influent Channel							
10	North Influent Channel	20.00		17.61	17.68	18.08	18.34	22.58
9	South Influent Channel	20.00		17.61	17.68	18.08	18.34	22.58
	Final Sedimentation Tanks							
11	North Effluent Trough	20.00		15.96	16.03	16.77	17.13	21.77
11A	North FST Water Surface	20.00	17.50	17.60	17.66	18.00	18.23	22.51
11B	North Effluent Channel	20.00		15.80	16.01	16.77	17.13	21.77
12	South Effluent Trough	20.00		15.96	16.03	16.77	17.13	21.77
12A	South FST Water Surface	20.00	17.50	17.60	17.66	18.00	18.23	22.51
12B	South Effluent Channel	20.00		15.80	16.01	16.77	17.13	21.77
	Chlorine Contact Tanks							
13	North Contact Tank Surface	20.00	14.84	15.75	15.93	16.58	16.87	21.58
14	North CT Effluent Below Weir	20.00		6.58	7.37	12.97	16.87	21.58
15	South Contact Tank Surface	20.00	14.84	15.75	15.93	16.58	16.87	21.58
14A	South CT Effluent Below Weir	20.00		6.58	7.37	12.97	16.87	21.58
	Outfall Well Chamber							
16	Outfall Well Chamber	16.00	-	6.20	6.72	10.38	12.92	16.00

Note:

1. The cells highlighted in red indicate processes with overflow potential.
2. The cells highlighted in blue indicate processes with submerged weir potential.

CCMUA
Model Predictions for Delaware River Elevation = 10'
Without Modifications

ID #	Location	Top of Tank Elev.	Weir Elev.	Model Prediction at 57 MGD	Model Prediction at 75 MGD	Model Prediction at 150 MGD	Model Prediction at 185 MGD	Model Prediction at 220 MGD
	Grit tanks							
1	Influent Channel	34.08		26.48	26.57	27.24	27.97	31.92
2A	Grit Tank Surface	34.08	26.10	26.47	26.54	27.15	27.85	31.84
2	Effluent Channel	34.08		24.97	25.18	26.45	27.05	31.84
	Primary Tanks							
3	West Influent Channel	26.65		24.87	25.02	25.95	26.37	31.44
3A	East Influent Channel	26.65		24.87	25.02	25.95	26.37	31.44
4	Influent Channel Near Tank #10	26.65	24.58/ 24.62	24.76	24.85	25.40	25.60	30.58
5A	PST Tank #10 Water Surface	26.65		24.65	24.66	24.71	24.98	30.54
5	South Old Effluent Channel	26.65		23.33	23.56	24.12	24.83	30.48
6A	PST Tank #1 Water Surface	26.65		24.65	24.66	24.71	24.98	30.54
6	North Old Effluent Channel	26.65		23.33	23.56	24.12	24.83	30.48
16	North New Effluent Channel	26.65		22.55	22.62	22.93	24.49	30.29
17	South New Effluent Channel	26.65		22.55	22.62	22.93	24.49	30.29
	Aeration Tanks							
7	North Splitter Box Surface	26.00	22.25	22.52	22.57	22.76	24.27	30.03
7A	North Splitter Box Trough	26.00		19.54	19.63	20.44	24.27	30.03
8	South Splitter Box Surface	26.00		22.52	22.57	22.76	24.27	30.03
8A	South Splitter Box Trough	26.00		19.54	19.63	20.44	24.27	30.03
	Aeration Tanks	26.00	18.50	18.69	18.73	18.86	22.29	27.28
10A	North Effluent Channel	20.00		17.65	17.74	18.46	22.29	27.27
9A	South Effluent Channel	20.00		17.65	17.74	18.46	22.29	27.27
	Final Tank Influent Channel							
10	North Influent Channel	20.00		17.61	17.68	18.26	22.09	27.04
9	South Influent Channel	20.00		17.61	17.68	18.26	22.09	27.04
	Final Sedimentation Tanks							
11	North Effluent Trough	20.00		15.96	16.03	17.63	21.51	26.27
11A	North FST Water Surface	20.00	17.50	17.60	17.66	18.19	22.03	27.01
11B	North Effluent Channel	20.00		15.80	16.01	17.63	21.51	26.27
12	South Effluent Trough	20.00		15.96	16.03	17.63	21.51	26.27
12A	South FST Water Surface	20.00	17.50	17.60	17.66	18.19	22.03	27.01
12B	South Effluent Channel	20.00		15.80	16.01	17.63	21.51	26.27
	Chlorine Contact Tanks							
13	North Contact Tank Surface	20.00	14.84	15.75	15.93	17.48	21.37	26.08
14	North CT Effluent Below Weir	20.00		11.08	11.87	17.47	21.37	26.08
15	South Contact Tank Surface	20.00	14.84	15.75	15.93	17.48	21.37	26.08
14A	South CT Effluent Below Weir	20.00		11.08	11.87	17.47	21.37	26.08
	Outfall Well Chamber							
16	Outfall Well Chamber	16.00	-	10.70	11.22	14.88	17.42	20.50

Note:

1. The cells highlighted in red indicate processes with overflow potential.
2. The cells highlighted in blue indicate processes with submerged weir potential.

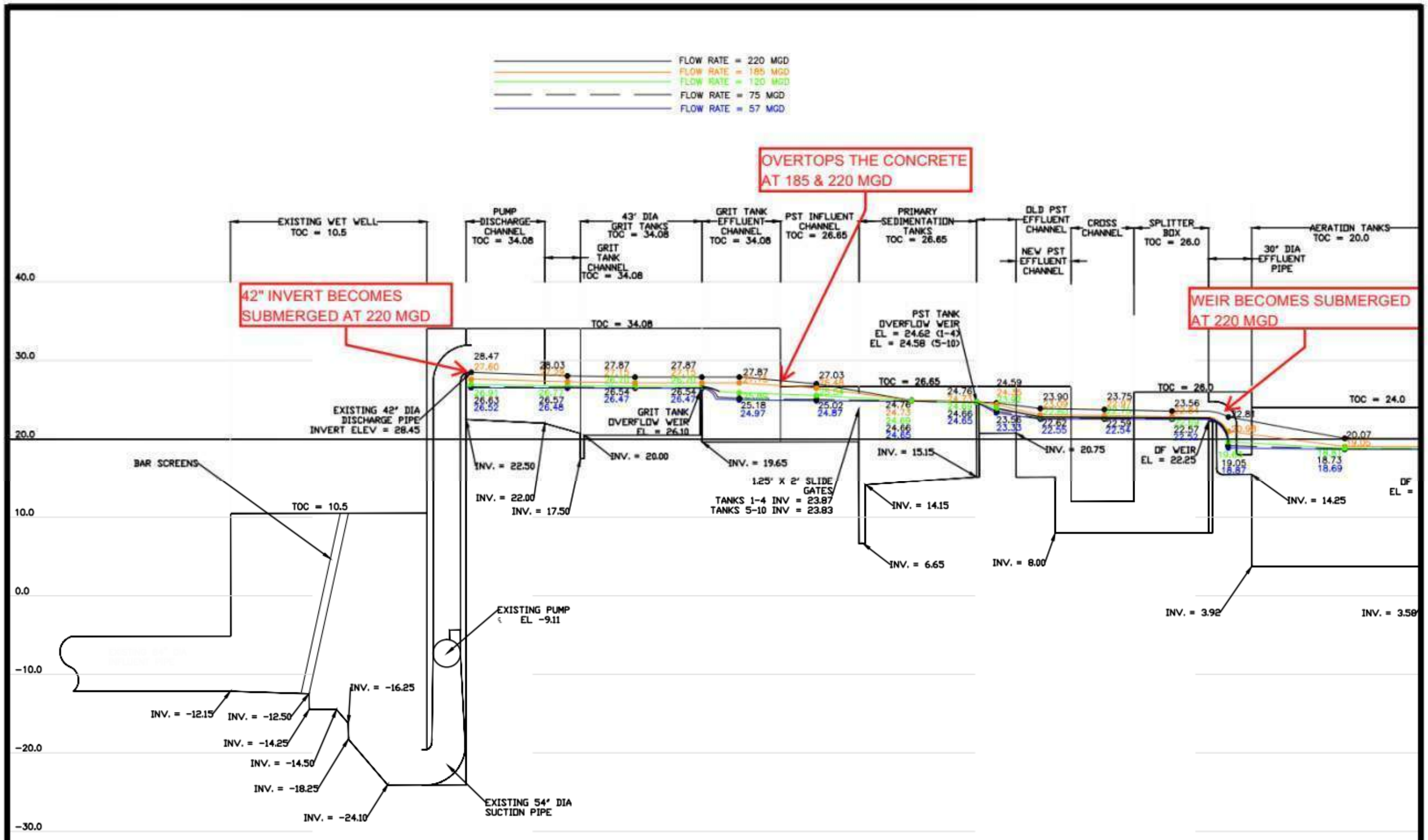
CCMUA
Model Predictions for Delaware River Elevation = 11.5'
Without Modifications

ID #	Location	Top of Tank Elev.	Weir Elev.	Model Prediction at 57 MGD	Model Prediction at 75 MGD	Model Prediction at 150 MGD	Model Prediction at 185 MGD	Model Prediction at 220 MGD
	Grit tanks							
1	Influent Channel	34.08		26.48	26.57	27.24	28.46	33.29
2A	Grit Tank Surface	34.08	26.10	26.47	26.54	27.15	28.35	33.23
2	Effluent Channel	34.08		24.97	25.18	26.45	27.55	33.23
	Primary Tanks							
3	West Influent Channel	26.65		24.87	25.02	25.95	26.95	32.90
3A	East Influent Channel	26.65		24.87	25.02	25.95	26.95	32.90
4	Influent Channel Near Tank #10	26.65	24.58/ 24.62	24.76	24.85	25.40	26.29	32.05
5A	PST Tank #10 Water Surface	26.65		24.65	24.66	24.71	26.13	32.02
5	South Old Effluent Channel	26.65		23.33	23.56	24.12	26.10	31.97
6A	PST Tank #1 Water Surface	26.65		24.65	24.66	24.71	26.13	32.02
6	North Old Effluent Channel	26.65		23.33	23.56	24.12	26.10	31.97
16	North New Effluent Channel	26.65		22.55	22.62	22.93	25.93	31.77
17	South New Effluent Channel	26.65		22.55	22.62	22.93	25.93	31.77
	Aeration Tanks							
7	North Splitter Box Surface	26.00	22.25	22.52	22.57	22.76	25.72	31.51
7A	North Splitter Box Trough	26.00		19.54	19.63	21.41	25.72	31.51
8	South Splitter Box Surface	26.00		22.52	22.57	22.76	25.72	31.51
8A	South Splitter Box Trough	26.00		19.54	19.63	21.41	25.72	31.51
	Aeration Tanks	26.00	18.50	18.69	18.73	20.04	23.76	28.76
10A	North Effluent Channel	20.00		17.65	17.74	19.67	23.76	28.76
9A	South Effluent Channel	20.00		17.65	17.74	19.67	23.76	28.76
	Final Tank Influent Channel							
10	North Influent Channel	20.00		17.61	17.68	19.51	23.58	28.54
9	South Influent Channel	20.00		17.61	17.68	19.51	23.58	28.54
	Final Sedimentation Tanks							
11	North Effluent Trough	20.00		15.96	16.03	19.09	23.01	27.77
11A	North FST Water Surface	20.00	17.50	17.60	17.66	19.46	23.53	28.51
11B	North Effluent Channel	20.00		15.80	16.01	19.09	23.01	27.77
12	South Effluent Trough	20.00		15.96	16.03	19.09	23.01	27.77
12A	South FST Water Surface	20.00	17.50	17.60	17.66	19.46	23.53	28.51
12B	South Effluent Channel	20.00		15.80	16.01	19.09	23.01	27.77
	Chlorine Contact Tanks							
13	North Contact Tank Surface	20.00	14.84	15.75	15.93	18.98	22.87	27.58
14	North CT Effluent Below Weir	20.00		12.58	13.37	18.97	22.87	27.58
15	South Contact Tank Surface	20.00	14.84	15.75	15.93	18.98	22.87	27.58
14A	South CT Effluent Below Weir	20.00		12.58	13.37	18.97	22.87	27.58
	Outfall Well Chamber							
16	Outfall Well Chamber	16.00	-	12.20	12.72	16.38	18.92	22.00

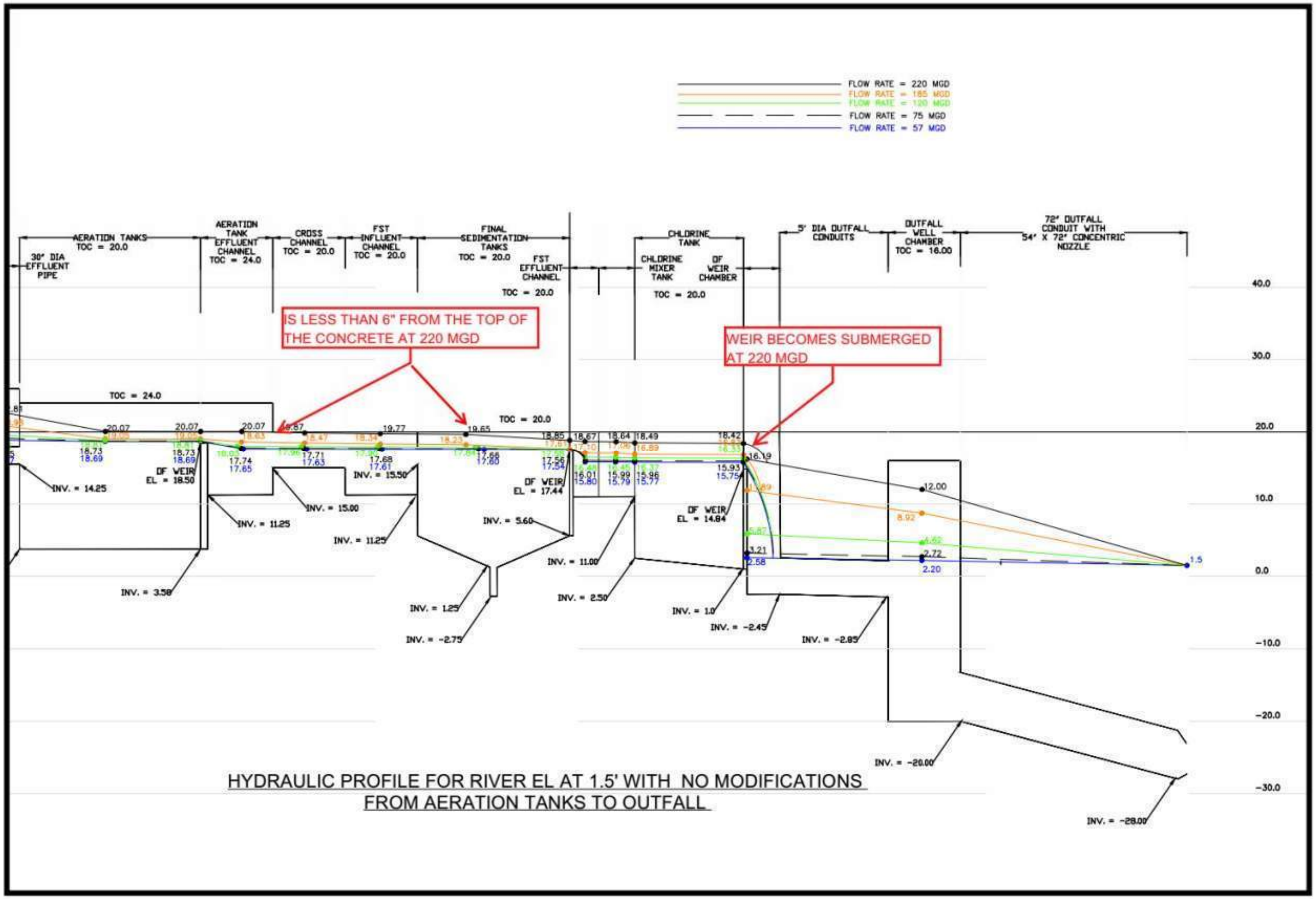
Note:

1. The cells highlighted in red indicate processes with overflow potential.
2. The cells highlighted in blue indicate processes with submerged weir potential.

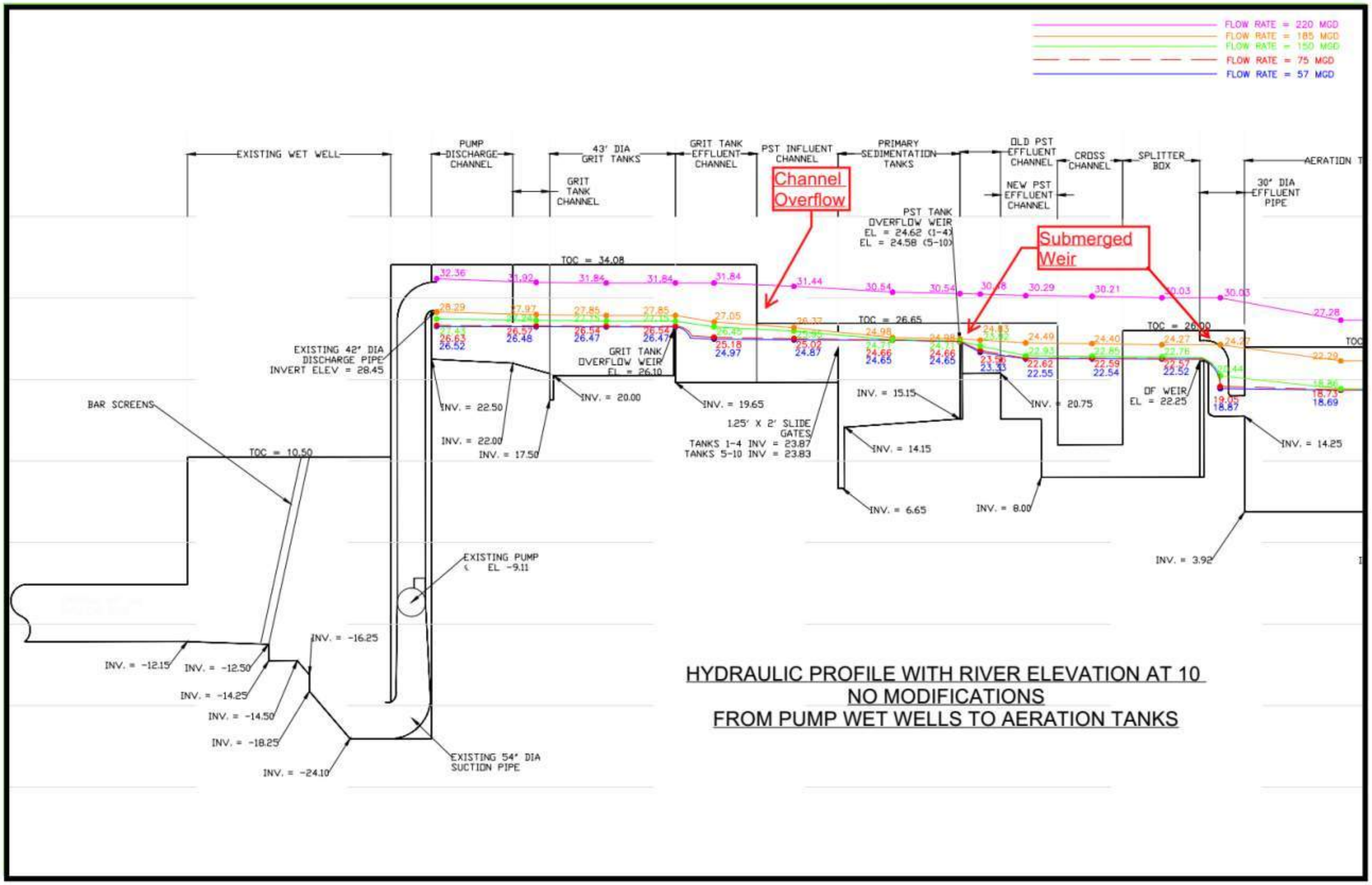
APPENDIX D
Hydraulic Grade Profiles

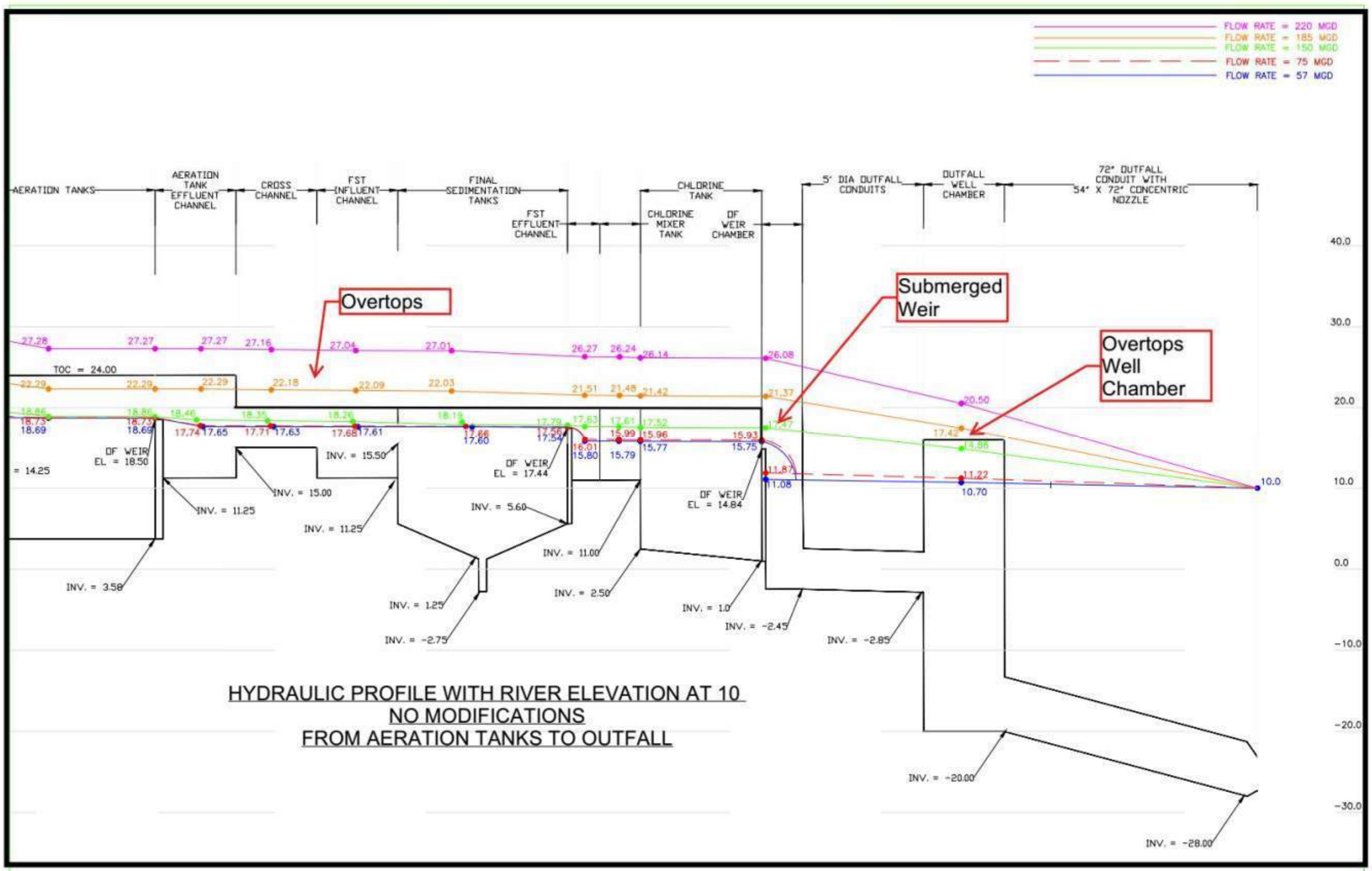


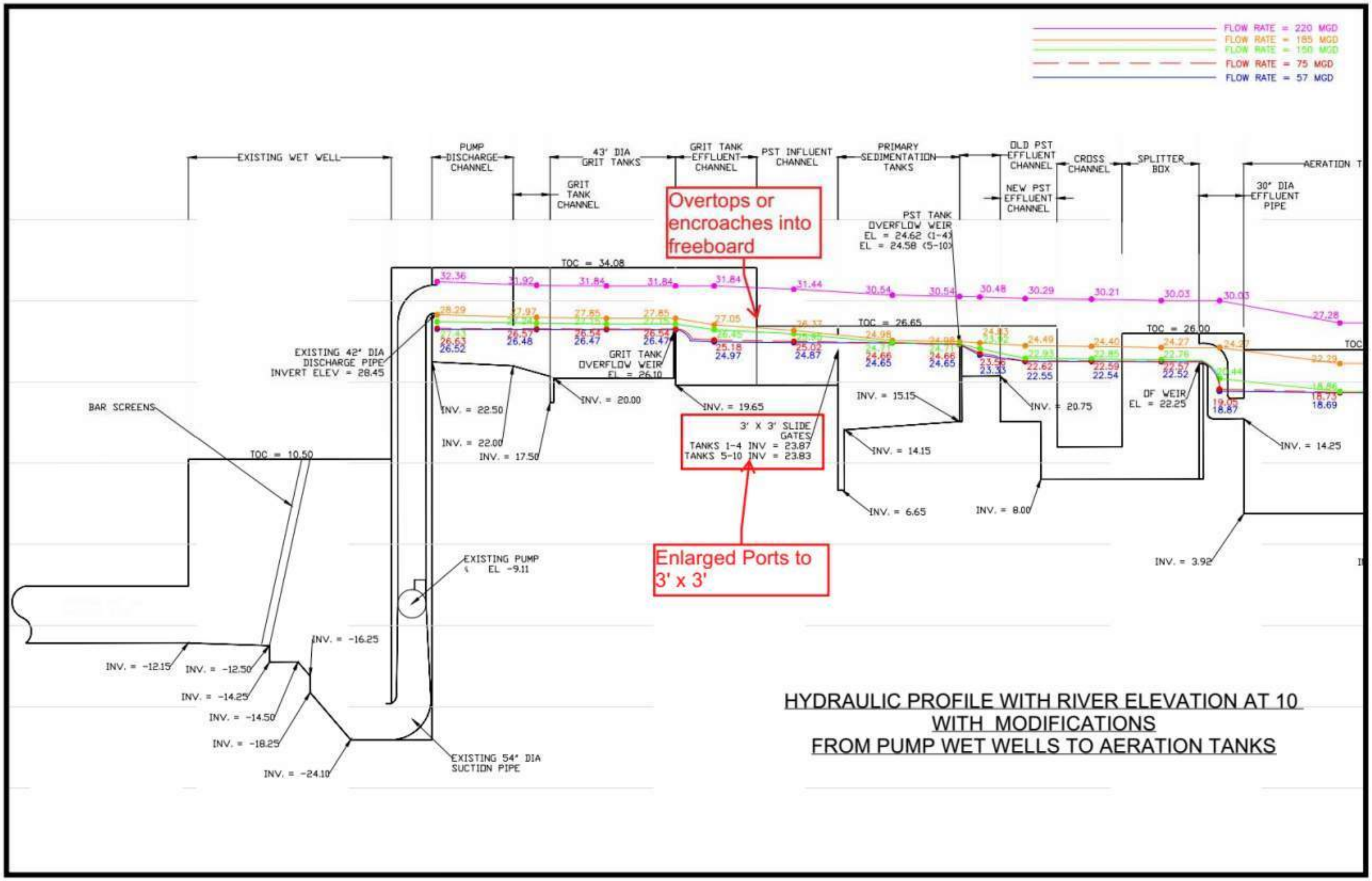
HYDRAULIC PROFILE FOR RIVER EL AT 1.5' WITH NO MODIFICATIONS
FROM PUMP WET WELL TO AERATION TANKS

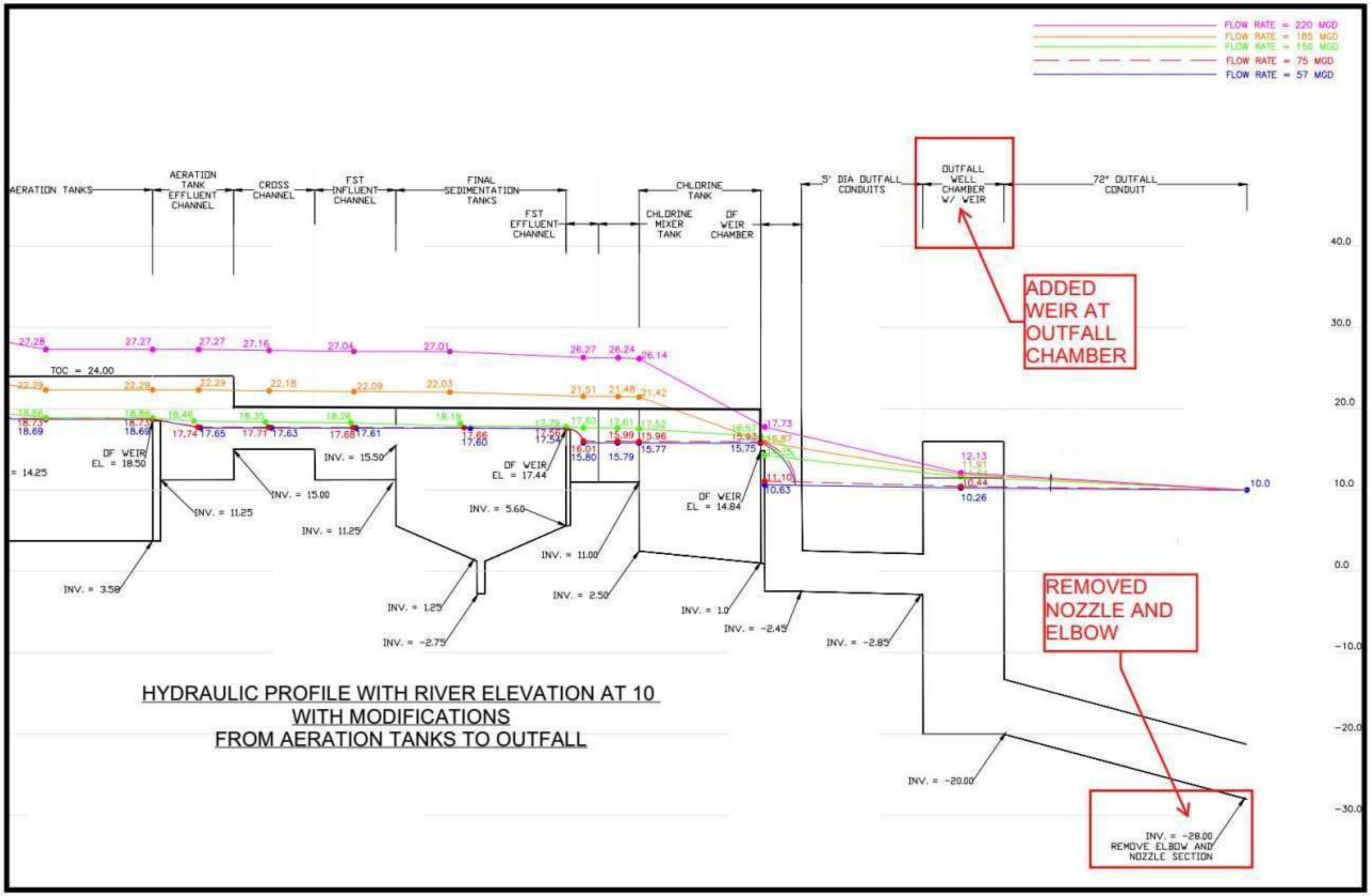


HYDRAULIC PROFILE FOR RIVER EL AT 1.5' WITH NO MODIFICATIONS FROM AERATION TANKS TO OUTFALL

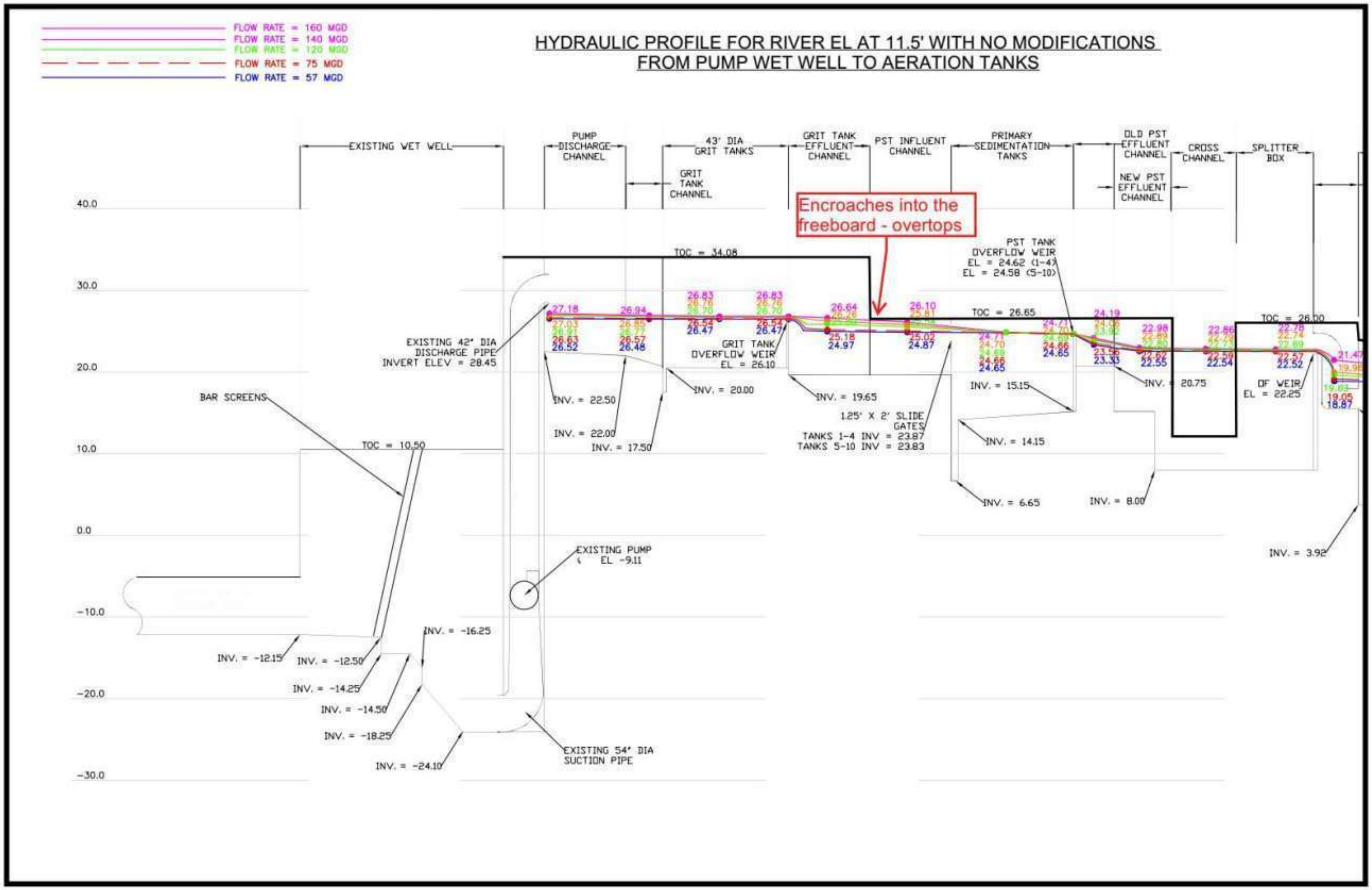

FLOW RATE = 220 MGD
FLOW RATE = 185 MGD
FLOW RATE = 150 MGD
FLOW RATE = 75 MGD
FLOW RATE = 57 MGD
EXISTING WET WELL
PUMP DISCHARGE CHANNEL
43' DIA GRIT TANKS
GRIT TANK CHANNEL
GRIT TANK EFFLUENT CHANNEL
PST INFLUENT CHANNEL
PRIMARY SEDIMENTATION TANKS
OLD PST EFFLUENT CHANNEL
NEW PST EFFLUENT CHANNEL
CROSS CHANNEL
SPLITTER BOX
30" DIA EFFLUENT PIPE
AERATION T
Channel Overflow
Submerged Weir
PST TANK OVERFLOW WEIR EL = 24.62 (1-4) EL = 24.58 (5-10)
TOC = 34.08
TOC = 26.65
TOC = 26.00
EXISTING 42" DIA DISCHARGE PIPE INVERT ELEV = 28.45
GRIT TANK OVERFLOW WEIR EL = 26.10
BAR SCREENS
TOC = 10.50
INV. = 22.50
INV. = 22.00
INV. = 17.50
INV. = 20.00
INV. = 19.65
1.25' X 2' SLIDE GATES
TANKS 1-4 INV = 23.87
TANKS 5-10 INV = 23.83
INV. = 15.15
INV. = 14.15
INV. = 6.65
INV. = 8.00
INV. = 20.75
OF WEIR EL = 22.25
INV. = 14.25
INV. = 3.92
EXISTING PUMP EL -9.11
INV. = -16.25
INV. = -12.15
INV. = -12.50
INV. = -14.25
INV. = -14.50
INV. = -18.25
INV. = -24.10
EXISTING 54" DIA SUCTION PIPE
HYDRAULIC PROFILE WITH RIVER ELEVATION AT 10
NO MODIFICATIONS
FROM PUMP WET WELLS TO AERATION TANKS

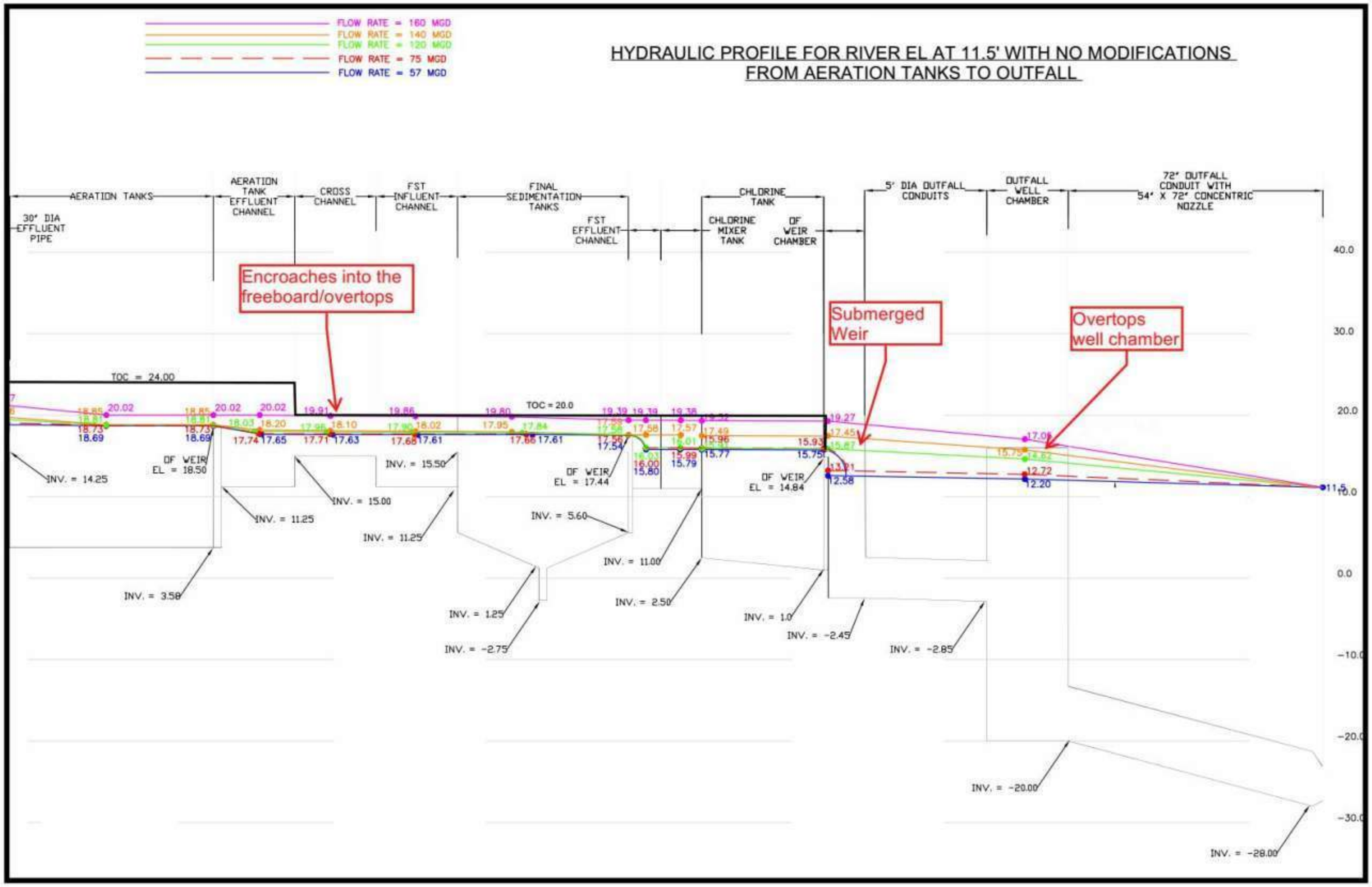

FLOW RATE = 220 MGD
FLOW RATE = 185 MGD
FLOW RATE = 150 MGD
FLOW RATE = 75 MGD
FLOW RATE = 57 MGD
AERATION TANKS
AERATION TANK EFFLUENT CHANNEL
CROSS CHANNEL
FST INFLUENT CHANNEL
FINAL SEDIMENTATION TANKS
FST EFFLUENT CHANNEL
CHLORINE TANK
CHLORINE MIXER TANK
OF WEIR CHAMBER
5' DIA OUTFALL CONDUITS
OUTFALL WELL CHAMBER
72" OUTFALL CONDUIT WITH 54" X 72" CONCENTRIC NOZZLE
Overtops
Submerged Weir
Overtops Well Chamber
TOC = 24.00
OF WEIR EL = 18.50
OF WEIR EL = 17.44
OF WEIR EL = 14.84
INV. = 3.58
INV. = 11.25
INV. = 15.00
INV. = 15.50
INV. = 5.60
INV. = 1.25
INV. = -2.75
INV. = 11.00
INV. = 2.50
INV. = 1.0
INV. = -2.45
INV. = -2.85
INV. = -20.00
INV. = -28.00
40.0
30.0
20.0
10.0
0.0
-10.0
-20.0
-30.0
HYDRAULIC PROFILE WITH RIVER ELEVATION AT 10
NO MODIFICATIONS
FROM AERATION TANKS TO OUTFALL

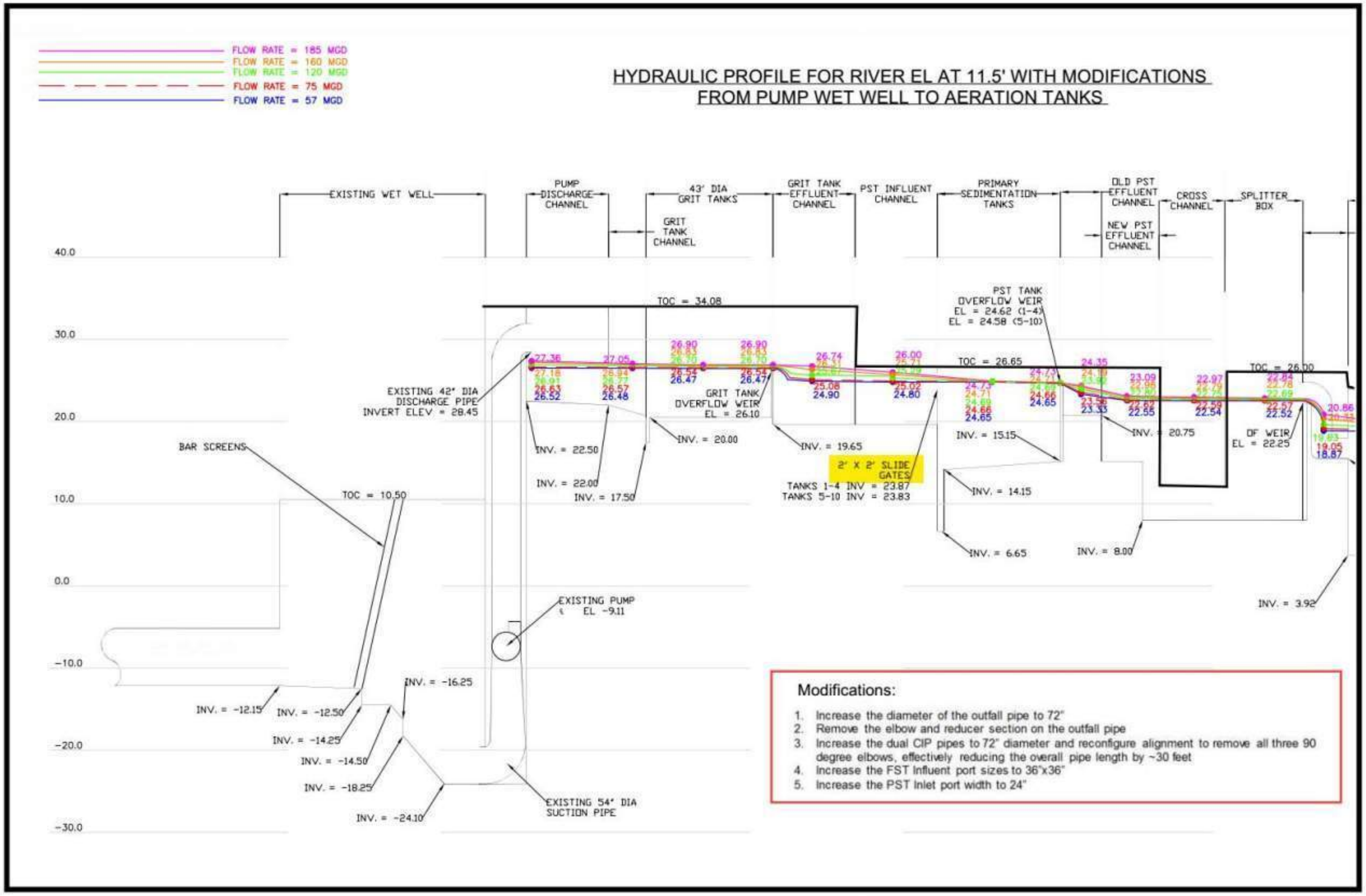

FLOW RATE = 220 MGD
FLOW RATE = 185 MGD
FLOW RATE = 150 MGD
FLOW RATE = 75 MGD
FLOW RATE = 57 MGD
EXISTING WET WELL
PUMP DISCHARGE CHANNEL
43' DIA GRIT TANKS
GRIT TANK CHANNEL
GRIT TANK EFFLUENT CHANNEL
PST INFLUENT CHANNEL
PRIMARY SEDIMENTATION TANKS
OLD PST EFFLUENT CHANNEL
NEW PST EFFLUENT CHANNEL
CROSS CHANNEL
SPLITTER BOX
30" DIA EFFLUENT PIPE
AERATION T
Overtops or encroaches into freeboard
PST TANK OVERFLOW WEIR EL = 24.62 (1-4) EL = 24.58 (5-10)
TOC = 34.08
TOC = 26.65
TOC = 26.00
EXISTING 42" DIA DISCHARGE PIPE INVERT ELEV = 28.45
GRIT TANK OVERFLOW WEIR EL = 26.10
BAR SCREENS
TOC = 10.50
INV. = 22.50
INV. = 22.00
INV. = 17.50
INV. = 20.00
INV. = 19.65
3' X 3' SLIDE GATES
TANKS 1-4 INV = 23.87
TANKS 5-10 INV = 23.83
INV. = 15.15
INV. = 14.15
INV. = 20.75
OF WEIR EL = 22.25
INV. = 14.25
INV. = 6.65
INV. = 8.00
INV. = 3.92
EXISTING PUMP EL -9.11
Enlarged Ports to 3' x 3'
INV. = -16.25
INV. = -12.15
INV. = -12.50
INV. = -14.25
INV. = -14.50
INV. = -18.25
INV. = -24.10
EXISTING 54" DIA SUCTION PIPE
HYDRAULIC PROFILE WITH RIVER ELEVATION AT 10 WITH MODIFICATIONS FROM PUMP WET WELLS TO AERATION TANKS

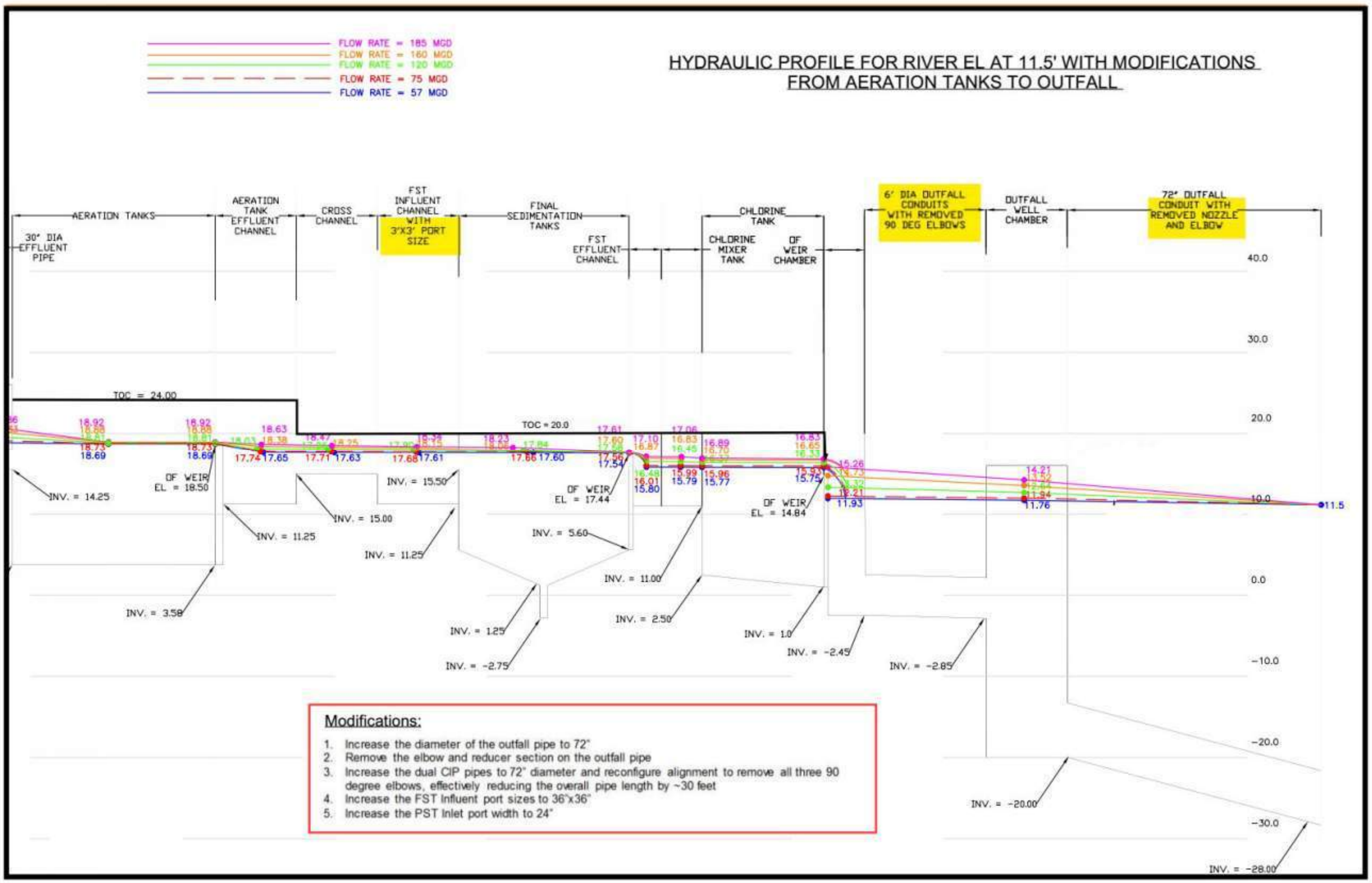


HYDRAULIC PROFILE WITH RIVER ELEVATION AT 10
WITH MODIFICATIONS
FROM AERATION TANKS TO OUTFALL


HYDRAULIC PROFILE FOR RIVER EL AT 11.5' WITH NO MODIFICATIONS
FROM PUMP WET WELL TO AERATION TANKS
FLOW RATE = 160 MGD
FLOW RATE = 140 MGD
FLOW RATE = 120 MGD
FLOW RATE = 75 MGD
FLOW RATE = 57 MGD
EXISTING WET WELL
PUMP DISCHARGE CHANNEL
43' DIA GRIT TANKS
GRIT TANK CHANNEL
GRIT TANK EFFLUENT CHANNEL
PST INFLUENT CHANNEL
PRIMARY SEDIMENTATION TANKS
OLD PST EFFLUENT CHANNEL
NEW PST EFFLUENT CHANNEL
CROSS CHANNEL
SPLITTER BOX
Encroaches into the freeboard - overtops
TOC = 34.08
PST TANK OVERFLOW WEIR EL = 24.62 (1-4) EL = 24.58 (5-10)
TOC = 26.65
TOC = 26.00
EXISTING 42" DIA DISCHARGE PIPE INVERT ELEV = 28.45
GRIT TANK OVERFLOW WEIR EL = 26.10
BAR SCREENS
TOC = 10.50
INV. = 22.50
INV. = 22.00
INV. = 17.50
INV. = 20.00
INV. = 19.65
1.25' X 2' SLIDE GATES TANKS 1-4 INV = 23.87 TANKS 5-10 INV = 23.83
INV. = 15.15
INV. = 14.15
INV. = 6.65
INV. = 8.00
INV. = 20.75
OF WEIR EL = 22.25
INV. = 3.92
EXISTING PUMP EL -9.11
INV. = -12.15
INV. = -12.50
INV. = -14.25
INV. = -14.50
INV. = -18.25
INV. = -24.10
INV. = -16.25
EXISTING 54" DIA SUCTION PIPE
40.0
30.0
20.0
10.0
0.0
-10.0
-20.0
-30.0


FLOW RATE = 160 MGD
FLOW RATE = 140 MGD
FLOW RATE = 120 MGD
FLOW RATE = 75 MGD
FLOW RATE = 57 MGD
HYDRAULIC PROFILE FOR RIVER EL AT 11.5' WITH NO MODIFICATIONS FROM AERATION TANKS TO OUTFALL
AERATION TANKS
AERATION TANK EFFLUENT CHANNEL
CROSS CHANNEL
FST INFLUENT CHANNEL
FINAL SEDIMENTATION TANKS
FST EFFLUENT CHANNEL
CHLORINE TANK
CHLORINE MIXER TANK
OF WEIR CHAMBER
5' DIA OUTFALL CONDUITS
OUTFALL WELL CHAMBER
72" OUTFALL CONDUIT WITH 54" X 72" CONCENTRIC NOZZLE
30" DIA EFFLUENT PIPE
Encroaches into the freeboard/overtops
Submerged Weir
Overtops well chamber
TOC = 24.00
TOC = 20.0
OF WEIR EL = 18.50
OF WEIR EL = 17.44
OF WEIR EL = 14.84
INV. = 14.25
INV. = 3.58
INV. = 11.25
INV. = 15.00
INV. = 15.50
INV. = 11.25
INV. = 5.60
INV. = 1.25
INV. = -2.75
INV. = 11.00
INV. = 2.50
INV. = 1.0
INV. = -2.45
INV. = -2.85
INV. = -20.00
INV. = -28.00
40.0
30.0
20.0
10.0
0.0
-10.0
-20.0
-30.0


FLOW RATE = 185 MGD
FLOW RATE = 160 MGD
FLOW RATE = 120 MGD
FLOW RATE = 75 MGD
FLOW RATE = 57 MGD
HYDRAULIC PROFILE FOR RIVER EL AT 11.5' WITH MODIFICATIONS
FROM PUMP WET WELL TO AERATION TANKS
EXISTING WET WELL
PUMP DISCHARGE CHANNEL
43' DIA GRIT TANKS
GRIT TANK CHANNEL
GRIT TANK EFFLUENT CHANNEL
PST INFLUENT CHANNEL
PRIMARY SEDIMENTATION TANKS
OLD PST EFFLUENT CHANNEL
NEW PST EFFLUENT CHANNEL
CROSS CHANNEL
SPLITTER BOX
40.0
30.0
20.0
10.0
0.0
-10.0
-20.0
-30.0
TOC = 34.08
PST TANK OVERFLOW WEIR EL = 24.62 (1-4) EL = 24.58 (5-10)
TOC = 26.65
TOC = 26.00
EXISTING 42" DIA DISCHARGE PIPE INVERT ELEV = 28.45
GRIT TANK OVERFLOW WEIR EL = 26.10
BAR SCREENS
TOC = 10.50
INV. = 22.50
INV. = 22.00
INV. = 17.50
INV. = 20.00
INV. = 19.65
2' X 2' SLIDE GATES
TANKS 1-4 INV = 23.87
TANKS 5-10 INV = 23.83
INV. = 15.15
INV. = 14.15
INV. = 6.65
INV. = 8.00
INV. = 20.75
OF WEIR EL = 22.25
INV. = 3.92
EXISTING PUMP EL -9.11
INV. = -16.25
INV. = -12.15
INV. = -12.50
INV. = -14.25
INV. = -14.50
INV. = -18.25
INV. = -24.10
EXISTING 54" DIA SUCTION PIPE
Modifications:
1. Increase the diameter of the outfall pipe to 72"
2. Remove the elbow and reducer section on the outfall pipe
3. Increase the dual CIP pipes to 72" diameter and reconfigure alignment to remove all three 90 degree elbows, effectively reducing the overall pipe length by ~30 feet
4. Increase the FST Influent port sizes to 36"x36"
5. Increase the PST Inlet port width to 24"


FLOW RATE = 185 MGD
FLOW RATE = 160 MGD
FLOW RATE = 120 MGD
FLOW RATE = 75 MGD
FLOW RATE = 57 MGD
HYDRAULIC PROFILE FOR RIVER EL AT 11.5' WITH MODIFICATIONS FROM AERATION TANKS TO OUTFALL
AERATION TANKS
30" DIA EFFLUENT PIPE
AERATION TANK EFFLUENT CHANNEL
CROSS CHANNEL
FST INFLUENT CHANNEL WITH 3'X3' PORT SIZE
FINAL SEDIMENTATION TANKS
FST EFFLUENT CHANNEL
CHLORINE TANK
CHLORINE MIXER TANK
OF WEIR CHAMBER
6' DIA OUTFALL CONDUITS WITH REMOVED 90 DEG ELBOWS
OUTFALL WELL CHAMBER
72" OUTFALL CONDUIT WITH REMOVED NOZZLE AND ELBOW
TOC = 24.00
TOC = 20.0
OF WEIR EL = 18.50
OF WEIR EL = 17.44
OF WEIR EL = 14.84
INV. = 14.25
INV. = 3.58
INV. = 11.25
INV. = 15.00
INV. = 11.25
INV. = 15.50
INV. = 1.25
INV. = -2.75
INV. = 5.60
INV. = 11.00
INV. = 2.50
INV. = 1.0
INV. = -2.45
INV. = -2.85
INV. = -20.00
INV. = -28.00
40.0
30.0
20.0
10.0
0.0
-10.0
-20.0
-30.0
11.5
Modifications:
1. Increase the diameter of the outfall pipe to 72"
2. Remove the elbow and reducer section on the outfall pipe
3. Increase the dual CIP pipes to 72" diameter and reconfigure alignment to remove all three 90 degree elbows, effectively reducing the overall pipe length by ~30 feet
4. Increase the FST Influent port sizes to 36"x36"
5. Increase the PST Inlet port width to 24"

APPENDIX E
Process Tables

Grit Tanks

Item Description	Existing Facilities					Modified Existing Facilities				
1. Wastewater Flow, MGD	57	75	150	185	220	57	75	150	185	220
2. No. of Grit Tanks	3	3	3	3	3	3	3	3	3	3
3. Tank Diameter, ft[(1)]	43	43	43	43	43	43	43	43	43	43
4. Tank Area, ft^2	1452	1452	1452	1452	1452	1452	1452	1452	1452	1452
5. Total Tank Area, ft^2	4357	4357	4357	4357	4357	4357	4357	4357	4357	4357
A. Normal Operation										
1. No. of Tanks in Service	3	3	3	3	3	3	3	3	3	3
2. Wastewater Flow, MGD	57	75	150	185	220	57	75	150	185	220
3. Total Operating Tank Area, ft^2	4,357	4,357	4,357	4,357	4,357	4,357	4,357	4,357	4,357	4,357
4. Overflow Rate, GPD/ft^2 [(3)]	13,084	17,215	34,430	42,464	50,498	13,084	17,215	34,430	42,464	50,498
5. Partical Size Removal, mesh	100+	100+	80-100	65-80	60-65	100+	100+	80-100	65-80	60-65

B. One Tank Out of Service										
1. No. of Tanks in Service	2	2	2	2	2	2	2	2	2	2
2. Wastewater Flow, MGD	57	75	150	185	220	57	75	150	185	220
3. Total Operating Area, ft^2 [(4)]	2,904	2,904	2,904	2,904	2,904	2,904	2,904	2,904	2,904	2,904
4. Overflow Rate, GPD/ft^2 [(3)]	19,625	25,823	51,646	63,696	75,747	19,625	25,823	51,646	63,696	75,747
5. Particle Size Removal, mesh	100+	100+	60-65	48-60	35-48	100+	100+	60-65	48-60	35-48

1) For average flow conditions, tank sizes based on MOP-8 maximum recommended overflow rate of 32,300 gpm/ft2 for 95% removal of 100 mesh (0.15mm) particle for a Detritus Grit Tank

2) Calculated overflow rates.

3) Largest Grit Tank Out of Sevice.

Primary Tanks

Item Description	Existing Facilities					Modified Existing Facilities				
A. General										
1. Wastewater Flow, MGD	57	75	150	185	220	57	75	150	185	220
2. No. of Sed. Tanks	10	10	10	10	10	10	10	10	10	10
3. Tank Width, ft	50	50	50	50	50	50	50	50	50	50
4. Tank Length, ft	186	186	186	186	186	186	186	186	186	186
5. Tank Depth, ft	12	12	12	12	12	12	12	12	12	12
6. Flow Depth, ft	10.00	10.01	10.06	11.48	12.00	10.00	10.01	10.04	10.06	10.08
B. Surface Area Evaluation										
1. Tank Surface Area, ft^2	9,300	9,300	9,300	9,300	9,300	9,300	9,300	9,300	9,300	9,300
2. Total Tank Surface Area, ft^2	93,000	93,000	93,000	93,000	93,000	93,000	93,000	93,000	93,000	93,000
3. Surface Overflow Rate, (SOR) gpd/ft^2	613	806	1,613	1,989	2,366	613	806	1,613	1,989	2,366
4. Recommended SOR, gpd/ft^2 (1)	2,000	2,000	2,000	2,000	2,000	2,000	2,000	2,000	2,000	2,000
5. Is Recommended SOR Exceeded?	**NO**	**NO**	**NO**	**NO**	**YES**	**NO**	**NO**	**NO**	**NO**	**YES**
5. Additional Tank Surface Area Required, ft^2	0	0	0	0	17,000	0	0	0	0	17,000
6. Total Tank Surface Area Required, ft^2	28,500	37,500	75,000	92,500	110,000	28,500	37,500	75,000	92,500	110,000

(1) From Recommended Standards for Wastewater Treatment Facilities, Great Lakes Upper Mississippi River Board of State Public Health and Environmental Managers, 1990.

Note:
1) The cells shown in yellow indicate that the Recommended SOR is exceeded.

Aeration Tanks

Item Description	Existing Facilities					Modified Existing Facilities				
A. General										
1. Wastewater Flow, MGD	57	75	150	185	220	57	75	150	185	220
2. No. of Aeration Tanks	8	8	8	8	8	8	8	8	8	8
3. Tank Width, ft	55	55	55	55	55	55	55	55	55	55
4. Tank Length, ft	220	220	220	220	220	220	220	220	220	220
5. Tank Depth, ft	20.42	20.42	20.42	20.42	20.42	20.42	20.42	20.42	20.42	20.42
6. Flow Depth, ft	14.94	14.98	15.11	16.67	16.67	14.94	14.98	15.06	15.13	15.97
B. Detention Time										
1. Tank Volume, MG	1.85	1.85	1.85	1.85	1.85	1.85	1.85	1.85	1.85	1.85
2. Total Tank Volume, MG	14.79	14.79	14.79	14.79	14.79	14.79	14.79	14.79	14.79	14.79
3. Wastewater Flow Per Tank, cfs	11.02	14.51	29.01	35.78	42.55	11.02	14.51	29.01	35.78	42.55
4. Detention Time, hours	4.56	3.47	1.75	1.57	1.32	4.56	3.47	1.75	1.42	1.26

Final Tanks

Item Description	Existing Facilities					Modified Existing Facilities				
A. General										
1. Wastewater Flow, MGD	57	75	150	185	220	57	75	150	185	220
2. No. of Sed. Tanks	8	8	8	8	8	8	8	8	8	8
3. Tank Width, ft	74	74	74	74	74	74	74	74	74	74
4. Tank Length, ft	268	268	268	268	268	268	268	268	268	268
5. Tank Depth, ft	5.05	5.05	5.05	5.05	5.05	5.05	5.05	5.05	5.05	5.05
6. Flow Depth, ft	12.49	12.51	12.74	14.95	14.95	12.49	12.51	12.53	12.55	13.33
B. Surface Area Evaluation										
1. Tank Surface Area, ft^2	19,832	19,832	19,832	19,832	19,832	19,832	19,832	19,832	19,832	19,832
2. Total Tank Surface Area, ft^2	158,656	158,656	158,656	158,656	158,656	158,656	158,656	158,656	158,656	158,656
3. Surface Overflow Rate, (SOR) gpd/ft^2	359	473	945	1,166	1,387	359	473	945	1,166	1,387
4. Recommended SOR, gpd/ft^2 (1)	1,200	1,200	1,200	1,200	1,200	1,200	1,200	1,200	1,200	1,200
5. Is Recommended SOR Exceeded?	**NO**	**NO**	**NO**	**NO**	**YES**	**NO**	**NO**	**NO**	**NO**	**YES**
5. Additional Tank Surface Area Required, ft^2	0	0	0	0	24,677	0	0	0	0	24,677
6. Total Tank Surface Area Required, ft^2	47,500	62,500	125,000	154,167	183,333	47,500	62,500	125,000	154,167	183,333

1) From Recommended Standards for Wastewater Treatment Facilities, Great Lakes Upper Mississippi River Board of State Public Health and Environmental Managers, 1990.

Note:
1) The cells show in yellow indicate that the Recommended SOR has been exceeded

Chlorine Contact Tanks

Item Description	Existing Facilities					Modified Existing Facilities				
A. General										
1. Wastewater Flow, MGD	57	75	150	185	220	57	75	150	185	220
2. No. of Contact Tanks	2	2	2	2	2	2	2	2	2	2
3. Tank Width, ft	15.25	15.25	15.25	15.25	15.25	15.25	15.25	15.25	15.25	15.25
4. Tank Length, ft	620	620	620	620	620	620	620	620	620	620
5. Tank Depth, ft	19	19	19	19	19	19	19	19	19	19
6. Flow Depth, ft	14.75	14.93	16.47	19.00	19.00	14.75	14.93	15.33	15.65	15.83
7. Is Contact Tank Overflowing?	**NO**	**NO**	**NO**	**YES**	**YES**	**NO**	**NO**	**NO**	**NO**	**NO**
B. Contact Time Evaluation										
1. Volume in CCT, ft^3	139,463	141,189	155,755	179,645	179,645	139,461	141,163	144,945	147,971	149,673
2. Required Contact Time, min	20	20	20	20	20	20	20	20	20	20
3. CCT Retention Time, min	53	41	22	21	18	53	41	21	17	15
5. Is Recommended Retention Time Satisfied?	**YES**	**YES**	**YES**	**YES**	**NO**	**YES**	**YES**	**YES**	**NO**	**NO**

Note:

1) Under Part A, the cells shown in yellow indicate that the contact tanks are overflowing.

2) Under Part B, the cells shown in yellow indicate that the Retention Time does not meet the required Contact Time.

APPENDIX F
Opinions of Probable Construction Costs

CCMUA
Delaware No. 1 WPCF
Equipment Rehabilitation and Replacement

Engineer's Opinion of Probable Construction Cost - Preliminary Design Assessment
Cost Summary No. 1

Greeley and Hansen
September 28, 2017

Description	Total Cost
Bid Item #1 - Primary Sedimentation Tank Influent Channel Modifications	$1,323,230
Bid Item #2 - Outfall Modifications and Improvements	$222,224
Subtotal	**$1,545,454**
Contractor's Overhead and Profit (21%)	**$251,546**
Subtotal	**$251,546**
Estimated Probable Cost	**$1,797,000**
The following items are excluded from the Estimated Probable Cost:	
- Permit fees	

CCMUA
Delaware No. 1 WPCF

Engineer's Opinion of Probable Construction Cost - Preliminary Design Assessment
Bid Item #1 - Preliminary Sedimentation Tank Inlet Port Improvements

Greeley and Hansen
September 28, 2017

Description	Quantity	Unit	Unit Price	Total Est Mat'l Cost	Labor % Mat'l	Total Est. Labor Cost	Total Cost
Demolition							
Selective Concrete Demolition - Channel Walls	3	CY	$0	$0	100%	$5,850	$5,850
Selective Concrete Demolition -Top Slab	6	CY	$0	$0	100%	$14,716	$14,716
Subtotal			$0				**$20,566**
Equipment							
1'-3" x 2'-0" Inlet Gates and Actuators	20	EA	$25,000	$500,000	30%	$150,000	$650,000
Actuators for Influent Channel Gates	2	EA	$15,000	$30,000	30%	$9,000	$39,000
Subtotal							**$689,000**
Structural							
Cast-in-Place Concrete Beams	3	CY	$4,883	$14,650	30%	$4,395	$19,045
Reinforcing Steel	1	TN	$4,500	$4,500	30%	$1,350	$5,850
Steel Bulkhead Plate	0.3	TN	$10,667	$3,200	30%	$960	$4,160
Steel L 6 x 4 Angles	0.75	TN	$11,000	$8,250	30%	$2,475	$10,725
Subtotal							**$39,780**
Electrical	1	LS	$127,581	$127,581	30%	$38,274	**$165,855**
I&C	1	LS	$85,054	$85,054	30%	$25,516	**$110,570**
Division 1 Costs (10%)							**$102,600**
Contingency (15%)							**$153,819**
Escalation to Midpoint (4%)							**$41,040**
Total Cost (not including allowance)(Roundoff)							**$1,323,230**

CCMUA
Delaware No. 1 WPCF

Engineer's Opinion of Probable Construction Cost - Preliminary Design Assessment
Bid Item #2 - Outfall Well Modifications and Improvements

Greeley and Hansen
September 28, 2017

Description	Quantity	Unit	Unit Price	Total Est Mat'l Cost	Labor % Mat'l	Total Est. Labor Cost	Total Cost
Demolition							
Cut Existing 60" pipe	1	LS	$0	$0	100%	$7,020	$7,020
Remove Existing L6 x L6 Braces	1	LS	$0	$0	100%	$5,200	$5,200
Subtotal							**$12,220**
Structural							
Steel: HP 14x73	0.25	TN	$12,000	$3,000	30%	$900	$3,900
Steel L 6 x 6 x 1/2" LLB Angles	1.5	TN	$12,000	$18,000	30%	$5,400	$23,400
3/4" Dia. SS Welded Steel Studs	8	EA	$300	$2,400	30%	$720	$3,120
3/4" Dia. SS Adhesive Anchors (ULB)	8	LS	$445	$3,560	30%	$1,068	$4,628
Welding of New 48" Risers	1	LS	$0	$0	100%	$7,800	$7,800
Cover Plate and Assembly	1	LS	$42,000	$42,000	30%	$12,600	$54,600
Subtotal							**$97,448**
Equipment							
48 " Piping	20	LF	$2,400	$48,000	30%	$14,400	$62,400
Subtotal							**$62,400**
Division 1 Costs (10%)							**$17,300**
Contingency (15%)							**$25,936**
Escalation to Midpoint (4%)							**$6,920**
Total Cost (not including allowance)(Roundoff)							**$222,224**

Greeley and Hansen LLC
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City, State Zip
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GREELEY AND HANSEN

Appendix C

Detailed Breakout of Cost Estimates by Subsystem Alternatives

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APPENDIX C

85% Capture Control Alternatives - Conceptual Design Level Cost Elements: Subject to Refinement and Revision

Control Alternative	Regulator Modifications	Flow Restriction Modifications	Source Reduction	Conveyance Upgrades	Pump Stations		WPCF # 1: 130 MGD Additional Wet Weather Capacity (pro-rated)		Satellite Facilities				Total Capital	Annual O&M	Present Worth
									Treatment		Storage				20
					Capital	Annual O&M	Capital	Annual O&M	Capital	Annual O&M	Capital	Annual O&M			Years
Newton Creek Sub-System													$0	$0	$0
Cooper River Sub-System															
Cooper River 1 - Satellite Treatment / Storage + Conveyance Upgrades															
Storage @ C-27 / Thorndyke (3.0 MG)	$300,000				$25,800,000	$131,000					$21,000,000	$463,000	$47,100,000	$594,000	$56,145,000
Treatment @ C-27 / Thorndyke (62.3 MGD)	$300,000				$25,800,000	$131,000			$25,900,000	$571,000			$52,000,000	$702,000	$62,690,000
Cooper River 2 - Conveyance Only	$300,000	$50,000		$42,240,000			$33,461,000	$1,003,830					$76,051,000	$1,004,000	$91,339,000
Cooper River 3 - Satellite Treatment / Storage Only															
Storage (C-22/22A @ 1.2 MG) + (C-27 / Thorndyke @ 3.6 MG)	$300,000										$37,900,000	$836,000	$38,200,000	$836,000	$50,930,000
Treatment (C-22/22A @ 20 MGD) + (C-27 / Thorndyke @ 20.5 MGD)	$300,000								$22,299,000	$492,000			$22,599,000	$492,000	$30,091,000
Delaware River - Camden	$500,000												$500,000	$0	$500,000
Delaware River - Back Channel															
Delaware BC1 - Satellite Treatment / Storage + Conveyance Upgrades @ C-32															
Storage @ C-32 (0.9 MG)	$200,000	$50,000			$19,100,000	$108,000					$12,500,000	$276,000	$31,850,000	$384,000	$37,697,000
Treatment @ C-32 (12.9 MG)	$200,000	$50,000			$19,100,000	$108,000			$12,300,000	$271,000			$31,650,000	$379,000	$37,421,000
Delaware BC2 - Source Reduction + Conveyance Upgrades	$200,000	$50,000	$15,000,000		$19,100,000	$108,000							$34,350,000	$108,000	$35,995,000
Delaware BC3 - Satellite Treatment / Storage Only															
Storage @ C-32 (2.4 MG)	$200,000										$23,800,000	$525,000	$24,000,000	$525,000	$31,994,000
Treatment @ C-32 (32 MGD)	$200,000								$16,300,000	$360,000			$16,500,000	$360,000	$21,982,000
Delaware River - Gloucester City															
Delaware River GC1 - Satellite Treatment / Storage															
Storage Serving G-4 & G-5 (2.4 MG)											$23,800,000	$525,000	$23,800,000	$525,000	$31,794,000
Treatment Serving G4 & G-5 (32 MGD)									$16,300,000	$360,000			$16,300,000	$360,000	$21,782,000
Delaware River GC2 - Conveyance Only	$300,000			$10,560,000			$26,439,000	$793,000					$37,299,000	$793,000	$49,374,000
System-Wide Improvements															
Expansion of Plant to 185 MGD													$0	$0	$0
Expansion of Plant to 220 MGD													$0	$0	$0
Green Stormwater Infrastructure															
Capital Costs													$56,100,000		
O&M Costs														$1,360,000	$76,809,000