NOTICE OF PUBLIC MEETING

Proposed Amendment to Water Quality Management Plan
Lower Dorchester Wastewater Treatment Plant Expansion from 8 to 16 mgd
and
Proposed 4 mgd Central Dorchester Wastewater Treatment Plant

The Berkeley Charleston Dorchester Council of Governments has scheduled a Public Meeting to solicit public comment and input concerning the expansion of Dorchester County's wastewater treatment capacity. The Lower Dorchester Wastewater Treatment Plant (WWTP) is currently permitted to discharge a flow of 8 mgd of treated effluent to Coosaw Creek in the Santee watershed via NPDES Permit No. SC0038822. Dorchester County is seeking an amendment to the BCD Water Quality Management Plan to expand the capacity of the WWTP from 8 mgd to 16 mgd. Additionally, Dorchester County is seeking an amendment to the BCD Water Quality Management Plan for a new 4 mgd Central Dorchester WWTP. The proposed 4 mgd Central WWTP will require a new NPDES discharge permit. Dorchester County proposes to divide the permitted wasteload allocation between the Lower and Central Dorchester WWTPs.

The Public Meeting will be held Tuesday, February 16, 2021 at 6:00 pm.

In an effort to practice social distancing guidelines related to the COVID-19 Pandemic, this meeting will be held virtually and livestreamed using Zoom. The link to access the Public Meeting will be posted on the BCDCOG's website at www.bcdcog.com.

The proposed amendment will be considered for approval by the BCDCOG following public review. Comments made at the Public Meeting or submitted in writing by March 2, 2021 will be in the public record for the Plan amendment proposal.

Written comments should be submitted to the BCD Council of Governments, 5790 Casper Padgett Way, North Charleston, SC 29406. Individuals with questions concerning the proposed amendment may contact the BCDCOG at (843) 529-0400 between the hours of 9 AM and 5 PM.

The Preliminary Engineering Report containing background information and justification for the amendment is available for public review at the Dorchester County Water and Sewer offices located at 235 Deming Way, Summerville, SC 29483 and the BCD Council of Governments' office 5790 Casper Padgett Way, North Charleston, SC 29406.

Date notice will appear in newspaper: February 2, 2021
Name of newspapers: Post and Courier and Charleston Chronicle

Project Summary

Lower Dorchester WWTP Expansion and Proposed Central Dorchester WWTP

Dorchester County is requesting a major amendment to the 208 Regional Water Quality Management Plan of the Berkeley-Charleston-Dorchester (BCD) Region. The requested amendment is required to expand the wastewater treatment capacity to meet the growing needs of the community. The capacity improvements include an expansion of the Lower Dorchester Wastewater Treatment Plant (WWTP) from 8 million gallons per day (mgd) to 16 mgd and a new 4 mgd Central Dorchester WWTP in the Pine Hill Business Campus area west of the Ashley River.

About Dorchester County

Dorchester County owns and operates the Lower Dorchester WWTP in North Charleston and the Upper Dorchester WWTP in St. George. The Lower Dorchester WWTP has a National Pollutant Discharge Elimination System (NPDES) permit for an annual average design flow of 8 mgd with tiered effluent permit limits for 10 and 12 mgd. The receiving stream is Coosaw Creek, which flows into the Ashley River. The Lower Dorchester service area is approximately 100 square miles.

Why is an Increase in Capacity Needed?

The population in Dorchester County has steadily increased since 1960. In 2010, the population was reported at approximately 136,600, representing a 42% increase from the population in 2000. Since 2010, the population has increased by 19% to approximately 162,809 people in 2019. Per the County's Comprehensive Plan, the anticipated population in 2030 population is estimated to be greater than 200,000 people. Approximately 60,000 residents are currently served by the Lower Dorchester WWTP and that number is expected to continue to rise.

When is the Increased Capacity Needed?

In consultation with the County's Planning and Zoning Department, an additional 8 mgd of capacity is needed to support the County's Comprehensive Plan and near-term growth in the area. An increased capacity to 10 mgd at the Lower Dorchester WWTP will be required by 2025 and an additional 5 mgd of capacity by 2030. Therefore, an expansion of the Lower Dorchester WWTP from 8 to 16 mgd is necessary to prepare for the anticipated growth.

A location has also been identified for a proposed Central Dorchester WWTP with an initial capacity of 4 mgd and a future expansion to 8 mgd at the Pine Hill Business Campus located west of the Ashley River. The location of the proposed Central Dorchester WWTP is proximate to the anticipated growth after 2035; however, growth in this area may occur faster and a new treatment facility may be needed prior to 2035. The proposed Central Dorchester WWTP will require a new NPDES permit for wastewater discharge. Speculative limits for the proposed discharge were received from the South Carolina Department of Health and Environmental Control (DHEC) in 2018. The County proposes a redistribution of the County's ultimate oxygen demand (UOD) wasteload allocation between the Lower and proposed Central treatment facilities.

Solution

Wastewater capacity alternatives have been evaluated to address the County's wastewater capacity needs throughout the next planning period. The Lower Dorchester WWTP is currently permitted for an annual average flow of 8 mgd with a tiered permit limit and wasteload allocation for 10 and 12 mgd. Therefore, the alternatives analyzed addressed the additional 4 mgd of capacity required to meet the needs of the service area. The alternatives that were considered in this analysis include:

- No-action.
- Land application.
- Water reuse.
- Use of other surface water discharge locations.
- Connection to other Publicly-Owned Treatment Works (POTWs).
- Infiltration and inflow (I/I) reduction.
- Expansion of the Lower Dorchester WWTP from 8 to 12 mgd and the proposed Central Dorchester WWTP with new NPDES discharge to the Ashley River.
- Expansion of the Lower Dorchester WWTP from 8 to 16 mgd with an increased NPDES discharge to Coosaw Creek.

The Project

A combination of existing and new infrastructure is recommended for the Lower Dorchester WWTP expansion project. The proposed Lower Dorchester WWTP expansion project will provide several improvements to increase the reliability of the treatment process. These improvements include:

- A new preliminary treatment facility with screening and grit removal.
- A conversion of the existing oxidation ditches to a plug-flow five-stage configuration.
- Two new plug-flow five-stage conventional activated sludge basins.
- A new blower building.
- Additional secondary clarification.
- One additional tertiary filter.
- A new ultraviolet (UV) disinfection facility.
- Sludge thickening.
- Electrical distribution system improvements including new generators to serve the facility.
- Flow distribution (e.g., influent, return activated sludge, mixed liquor suspended solids, and secondary effluent).
- No improvements are proposed to the existing dewatering facilities.

The proposed Central Dorchester WWTP will include similar processes as Lower Dorchester. The proposed infrastructure includes:

- Preliminary treatment with screening and grit removal.
- Five-stage conventional treatment activated sludge process with diffused aeration and blowers.
- Secondary clarification.
- Tertiary filtration.
- Ultraviolet disinfection.
- Effluent pumping and post aeration.
- Solids storage and mechanical dewatering.

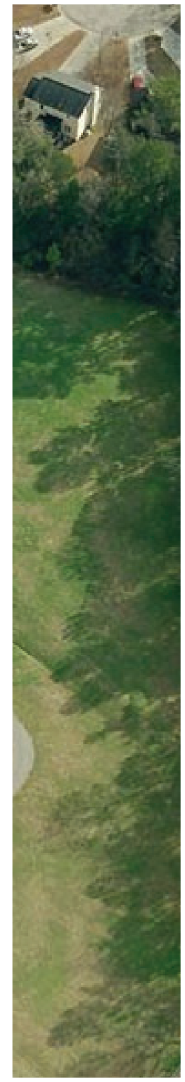
Per discussion with DHEC, the proposed outfall will be located at the Ashley River at the Highway 17 Alt. Bridge.

Get Involved

Dorchester County is following the 208 Water Quality Management Planning guidance for requesting additional wastewater capacity for their service area. A virtual public meeting will be held on February 16, 2021 at 6:00 PM. The public meeting will provide a forum for the County to present the project in more detail. All comments made during the meeting will become part of the public record. Comments may also be submitted in writing by March 2, 2021. The Preliminary Engineering Report containing background information and justification for the amendment is available for public review at 235 Deming Way, Summerville, SC 29483 and the BCD Council of Governments, 5790 Casper Padgett Way, North Charleston, SC 29406.

Preliminary Engineering Report

Lower Dorchester
Wastewater Treatment Plant Expansion



Dorchester County, South Carolina
October 2020

Hazen

DORCHESTER COUNTY
SERVICE • JUSTICE • PROSPERITY
1897

Preliminary Engineering Report

Lower Dorchester Wastewater Treatment Plant Expansion

Dorchester County, South Carolina
October 2020

SOUTH CAROLINA
LICENSED PROFESSIONAL ENGINEER
No. 24069
ALAN STONE
10/20/20

Client Contact:

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SOUTH CAROLINA
CERTIFICATE OF AUTHORIZATION
HAZEN& SAWYER
No. C00638

EXECUTIVE SUMMARY

Dorchester County owns and operates the Lower Dorchester Wastewater Treatment Plant (WWTP). The County has a National Pollutant Discharge Elimination System (NPDES) permit for an annual average design flow of 8 million gallon per day (mgd) with permit limits for expansion to 10 and 12 mgd. The WWTP receiving stream is Coosaw Creek, a tributary to the Ashely River. The WWTP discharge is located approximately 0.78 miles from the confluence with the Ashley River near the Charleston County line. The Lower Dorchester WWTP serves approximately 60,000 people. The majority of the influent flow is pumped to the WWTP.

The population in Dorchester County has demonstrated steady increases since 1960. In 2010, the County's population was reported at approximately 136,600 people, representing a 42 percent increase from the County's population in 2000. Since 2010, the growth in the County increased by 19 percent to approximately 162,809 people in 2019. Per Dorchester County's Comprehensive Plan (November 2018), the 2030 population in the County is anticipated to be slightly greater than 200,000 people.

Dorchester County continues to experience growth in the Lower Dorchester service area. The County's Planning Director was consulted in the fall of 2019 for information on the timing and location of the County's near and long term growth. An additional 8 mgd of capacity is needed over the planning period to satisfy the anticipated growth, which is commensurate with the County's Comprehensive Plan. Included in the anticipated planned growth are firm commitments for approximately 10 mgd of capacity at the Lower Dorchester WWTP by the year 2025 and firm requests from the development community for an additional 5 mgd of capacity by the year 2030. Therefore, an expansion of the Lower Dorchester WWTP from 8 to 16 mgd is necessary to prepare for the anticipated near-term growth. The Lower Dorchester WWTP site is constrained and will only support an ultimate capacity of 16 mgd.

The County identified a location for a proposed Central Dorchester WWTP with an initial capacity of 4 mgd and a future expansion to 8 mgd in the Pine Hill Business Campus area west of the Ashley River. The location of the proposed Central Dorchester WWTP is proximate to the anticipated growth after 2035. The proposed Central Dorchester WWTP is discussed in this Lower Dorchester Preliminary Engineering Report (PER) for two reasons. First, the County proposes to re-distribute the County's ultimate oxygen demand (UOD) wasteload allocation between the Lower and proposed Central treatment facilities. Second, the County wishes to pursue the NPDES permit for the proposed Central Dorchester WWTP in the event that growth in the service area occurs faster than the growth currently anticipated in the planning period.

Current and Future NPDES Permit Requirements

The Lower Dorchester WWTP is permitted to discharge 8 mgd of treated effluent into Coosaw Creek in the Santee watershed via NPDES permit SC0038822. The NPDES permit includes tiered flow limits for 10 and 12 mgd. The NPDES permit includes effluent limits requirements for UOD at annual average design flows of 10 and 12 mgd. The UOD was developed by the South Carolina Department of Health and Environmental Control (DHEC) as a result of the Charleston area total maximum daily load (TMDL) to address dissolved oxygen impairment in the Charleston Harbor and Cooper, Wando, and Ashley Rivers.

The County requested a wasteload allocation from DHEC for a discharge for the proposed Central Dorchester WWTP in February 2018. Per a speculative limit response letter in June 2018, DHEC provided a stringent speculative limit ammonia concentration of 0.5 milligrams per liter (mg/L) for the proposed Central Dorchester WWTP in the summer months. Additionally, DHEC stated that a pound per pound allocation of UOD load from another treatment facility on the Ashley River would be required to maintain the TMDL. In response to the speculative limit letter, the County proposes to re-distribute the Lower Dorchester UOD wasteload allocation between the Lower Dorchester WWTP expansion from 8 to 16 mgd and the proposed Central Dorchester WWTP. For a 16 mgd Lower Dorchester WWTP and a 4 mgd proposed Central Dorchester WWTP, the concentration limits would be 5.0 mg/L of five-day carbonaceous biochemical oxygen demand ($cBOD_5$) and 0.8 mg/L of ammonia for the Lower Dorchester WWTP and 3.0 mg/L $cBOD_5$ and 0.5 mg/L of ammonia for the proposed Central Dorchester WWTP.

During the speculative limit and wasteload allocation discussions, DHEC indicated that future nutrient limits are a possibility in the Charleston Harbor and Cooper, Wando, and Ashley Rivers. The timing of the proposed nutrient limits was unclear; however, these nutrient limits are not anticipated in the next two NPDES permit cycles. DHEC is in the process of collecting data in the watershed for model development, calibration, and validation. In response, the County has agreed to construct a five-stage biological process for the Lower Dorchester WWTP expansion and the proposed Central Dorchester WWTP in anticipation of future nutrient limits. The recommended five-stage configuration will allow flexibility for the process to be optimized for future nutrient limits. The process modeling indicates that the recommended process volume and configuration may be able to achieve an effluent total nitrogen between 6 and 10 mg/L.

Summary of Alternatives Analysis

Eight wastewater capacity alternatives were evaluated to address the Dorchester County's wastewater capacity needs for an additional 8 mgd over the next planning period. The Lower Dorchester WWTP is currently permitted for an annual average flow of 8 mgd with a tiered permit limit and wasteload allocation of 12 mgd. The alternatives analysis addressed the additional 4 mgd of capacity required to meet the needs of the service area over the planning period. The alternatives that were considered in this analysis include no-action, land application, water reuse, use of other surface water discharge locations, connection to other Publicly-Owned Treatment Works (POTWs), infiltration and inflow (I/I) reduction, expansion of the Lower Dorchester WWTP from 8 to 12 mgd and the proposed Central Dorchester WWTP with new NPDES discharge to the Ashley River, and expansion of the Lower Dorchester WWTP from 8 to 16 mgd with an increased NPDES discharge to Coosaw Creek.

An expansion of the Lower Dorchester WWTP from 8 to 16 mgd is the preferred alternative. This alternative is the most economical for the County's rate-payers. The second preferred alternative includes the option of constructing a new Central Dorchester WWTP during the next planning period. However, the County may use the Central Dorchester WWTP site for equalization during this planning period. The County also wishes to pursue the NPDES permit for the proposed Central Dorchester WWTP in the event that growth in the service area occurs faster than the growth currently anticipated.

Facility Infrastructure Recommendations

A combination of existing and new infrastructure is recommended for the Lower Dorchester WWTP expansion project. Table ES-1 provides a summary of the recommended infrastructure necessary for a WWTP expansion to 16 mgd. The proposed Lower Dorchester WWTP expansion project will provide several improvements to increase the reliability of the treatment process. A new preliminary treatment facility and additional secondary clarification will be required. In lieu of mechanical aeration, the existing oxidation ditches will be converted to a plug-flow five-stage configuration with diffused aeration and blowers. Additionally, two new plug-flow conventional activated sludge basins with a plug flow five-stage configuration are required for the expansion from 8 to 16 mgd. The ultraviolet (UV) disinfection facility will be replaced with newer and more reliable technology to meet the stringent enterococci limits. The electrical distribution system will be also be re-designed to address the increase in electrical load for the expansion and improve the reliability and redundancy of the plant electrical system. The electrical improvements include new generators to serve the entire facility. Additionally, new flow distribution (e.g., influent, return activated sludge, mixed liquor suspended solids, and secondary effluent) is necessary to meet the stringent effluent permit limits.

Table ES-1: Summary of Existing and New Infrastructure Required for a 16 mgd Lower Dorchester WWTP Expansion

Unit Process	Unit Process Type	Infrastructure Needed for 16 mgd WWTP Capacity
Influent pumping	On-site influent pump station 2	• New 30-inch force main to replace existing parallel 16-inch force mains • Two new influent jockey pumps to capture lower range of WWTP flow
Preliminary treatment facility	Screens and compactors	• New structure • Two mechanically cleaned screens • One manually cleaned screen
	Grit removal	• Two grit units, 12 stacked trays per unit
	Grit cyclones	• Two units
	Grit classifiers	• Two units
Influent flow measurement	Parshall flume	• One flume at 48-inch throat width • Integral with new preliminary treatment facility
Influent / RAS distribution	Splitter box	• New distribution box • Four distribution weirs • Integral with new preliminary treatment facility
Waste activated sludge pumping	WAS pump station	• Two pumps and magnetic flow meter • Integral with new preliminary treatment facility
Secondary treatment	Aeration basins	• Two new aeration basins in a plug flow five-stage configuration • Retrofit of existing oxidation ditches 3 and 4 to a plug flow five-stage configuration

Table ES-1: Summary of Existing and New Infrastructure Required for a 16 mgd Lower Dorchester WWTP Expansion

Unit Process	Unit Process Type	Infrastructure Needed for 16 mgd WWTP Capacity
	NRCY Pumps	• One pump per basin, four total
	Vertical shaft mixers	• Five mixers per basin, 20 total
Aeration System	Multi-stage centrifugal blowers	• New blower building with electrical room housing main WWTP switchgear • Three 4,000 scfm blowers • Two 6,200 scfm blowers • 2,500 diffusers per basin, 10,000 diffusers total
MLSS distribution	Splitter box	• New distribution box • Six distribution weirs
Secondary clarification and RAS pumping	Secondary clarifiers	• Four existing clarifiers and two new clarifiers • Six total clarifiers
	RAS pump station	• New RAS pump station • Three RAS pumps
Tertiary filtration	Disk filters	• Six existing filters and one new filter • Seven filters total • Filter relocation to common point • New tertiary effluent box
Disinfection	UV disinfection	• New UV structure and electrical room • Two channels
Effluent flow measurement	Parshall flume	• One flume at 48-inch throat width • Integral with new UV structure
Reclaimed water system	Pumping	• Two new reclaimed water transfer pumps • No modifications to existing reclaimed water pump station
	Storage	• No modifications to existing ground storage tank
Solids Handling	Decant basins	• Three days of storage at 0.7% solids • No modifications to existing basins or aeration system
	Aerated sludge holding	• Two new aerated rectangular sludge storage tanks • 5 to 8 storage days at 2% to 3% solids • Two new positive displacement blowers
	Thickening	• New thickening building with electrical room • Two new RDTs, pumps and appurtenances with space for a third RDT
	Dewatering	• No modifications to existing dewatering building with two DCENs

Summary of Opinion of Probable Cost

The opinion of probable construction cost for the expansion of the Lower Dorchester WWTP to 16 mgd was prepared in accordance with the guidelines of the Association for the Advancement of Cost Engineering (AACE) International for a Class 3 level of estimation. A Class 3 estimate is prepared based on information developed during a preliminary design. The expected accuracy range for a Class 3 level of estimation is +30% to –20%.

The opinion of probable construction cost is summarized in ES-2 and expressed in 2019 dollars. The cost opinion is based on the facility infrastructure recommendations for the liquid and solids infrastructure improvements. Construction costs include a 30 percent contingency, 3 percent bonds and insurance, 7 percent County tax on materials, and 20 percent contractor overhead and profit. The cost opinion also includes 15 percent for general conditions to include mobilization, contract administration, trailer, field supervisor, shop drawings, and start-up and training. Labor was escalated to the mid-point of construction at 3.5 percent over a 36 month construction duration. Materials and equipment was escalated to the mid-point of construction at 5 percent. Construction costs were estimated using quotes from equipment vendors and quantity take-offs for concrete, excavation, stone, metal appurtenances, and piping. For smaller ancillary equipment, costs were estimated from similarly sized Hazen and Sawyer projects.

Table ES-2: Opinion of Probable Construction and Project Costs for Lower Dorchester WWTP Expansion to 16 mgd

Project Component [1, 2, 3, 4, 5, 6]	Opinion of Capital Construction Cost for 8 to 16 mgd Expansion of Lower Dorchester WWTP
Demolition	$759,000
Site work	$4,690,000
Yard piping	$8,365,000
Preliminary treatment facility, influent/RAS distribution, WAS pumping	$7,544,000
New aeration basins 1 and 2	$13,245,000
Retrofit of aeration basins 3 and 4	$9,343,000
Mixed liquor suspended solids distribution box	$1,578,000
Secondary clarifiers	$4,595,000
Return activated sludge pump station 5	$1,424,000
Blower building	$4,987,000
Tertiary disk filter and tertiary effluent box	$2,021,000
UV disinfection and building	$5,209,000
Thickening building	$5,770,000

Table ES-2: Opinion of Probable Construction and Project Costs for Lower Dorchester WWTP Expansion to 16 mgd

Project Component [1, 2, 3, 4, 5, 6]	Opinion of Capital Construction Cost for 8 to 16 mgd Expansion of Lower Dorchester WWTP
Aerated sludge holding and blowers	$2,942,000
Electrical work and generator	$6,684,000
General conditions	$11,872,000
Total opinion of probable construction cost	**$91,000,000**
Construction cost opinion range at Class 3 AACE level:	
Low (-20%)	$72,800,000
High (30%)	$118,300,000

[1] Cost opinion includes 3% for bonds and insurance.
[2] Cost opinion includes 20% contractor overhead and profit and 7% County taxes on materials.
[3] Site assumes that shallow foundations will be adequate in lieu of auger cast piles. A geotechnical evaluation is required to confirm this assumption.
[4] Cost opinion includes 30% contingency.
[5] General conditions assumes 15% for mobilization, contract administration, field staff and trailer, shop drawings, and start-up and training.
[6] Labor and materials / equipment were escalated to the mid-point of construction at 3.5% and 5%, respectively.

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List of Acronyms

AACE	Association for the Advancement of Cost Engineering
AC	Absence of Controls
ATS	Automatic transfer switch
BOD_5	Biochemical oxygen demand, 5-day
cBOD5	Carbonaceous biochemical oxygen demand, 5-day
CF	Cubic feet
Cl_2	Chlorine
COD	Chemical oxygen demand
CWA	Clean Water Act
DCEN	Decanting centrifuge
DHEC	South Carolina Department of Health and Environmental Control
DO	Dissolved oxygen
EPA	United States Environmental Protection Agency
EPDM	Ethylene propylene diene methylene
ERU	Equivalent residential unit
FEMA	Federal Emergency Management Agency
GBT	Gravity belt thickener
GIS	Geographic information system
gpd	Gallon per day
gpd/SF	Gallon per day per square foot
gpm	Gallon per minute
gpm/SF	Gallon per minute per square foot
HDPE	High density polyethylene
HP	Horsepower
HRT	Hydraulic retention time
HUC	Hydrologic unit code
lb	Pound
lb/d	Pounds per day
lb/hr	Pound per hour
lb/MG	Pound per million gallon
icfm	Inlet cubic feet per minute
I/I	Infiltration and inflow
in/wk	Inch per week
IUP	Industrial use permit

List of Acronyms

kVA	Kilo-volt-ampere
kW	Kilowatts
MCC	Motor control center
MG	Million gallons
mgd	Million gallons per day
mg/L	Milligrams per liter
mJ/cm^2	Milli-joules per square centimeter
MLSS	Mixed liquor suspended solids
MLVSS	Mixed liquor volatile suspended solids
MOPO	Maintenance of plant operations
MPN	Most probable number
MSA	Metropolitan Statistical Area
MSL	Mean sea level
NATS	Non-automatic transfer switch
NCSD	North Charleston Sanitary District
NH_3-N	Ammonia
NPDES	National Pollutant Elimination System
NRCY	Nitrified recycle flow
PER	Preliminary Engineering Report
POTW	Publicly-owned treatment works
psig	Pounds per square inch gage
RAS	Return activated sludge
RDT	Rotary drum thickener
SCADA	Supervisory control and data acquisition
SCE&G	South Carolina Electric & Gas
scfm	Standard cubic feet per minute
SF	Square feet
SIU	Significant industrial user
SOTE	Standard oxygen transfer efficiency
SRT	Solids retention time
TKN	Total Kjeldahl nitrogen
TMDL	Total maximum daily load
TSS	Total suspended solids

List of Acronyms

UOD	Ultimate oxygen demand
USGS	United States Geological Survey
UV	Ultraviolet
V	Voltage
VFD	Variable frequency drive
WAS	Waste activated sludge
WWTP	Wastewater treatment plant

1. Need for Improvements

1.1 Introduction

Dorchester County owns and operates the Lower Dorchester Wastewater Treatment Plant (WWTP). The County has a National Pollutant Discharge Elimination System (NPDES) permit for an annual average design flow of 8 million gallon per day (mgd) with permit limits for expansion to 10 and 12 mgd. The WWTP receiving stream is Coosaw Creek, a tributary to the Ashely River. The WWTP discharge is located approximately 0.78 miles from the confluence with the Ashley River near the Charleston County line. The Lower Dorchester WWTP serves approximately 60,000 people. The majority of the influent flow is pumped to the WWTP.

The Lower Dorchester WWTP was first constructed in 1984 as a 1.85 mgd lagoon treatment facility. The facility was expanded to 4 mgd in 1994 to conventional activated sludge via an extended aeration process. A significant plant expansion occurred in 2006 with an increase in capacity from 4 to 8 mgd. The 2006 expansion project included the addition of a new sludge dewatering facility. The Lower Dorchester WWTP was designed as a conventional activated sludge treatment plant with ammonia oxidation (e.g., nitrification) via oxidation ditch technology. Other unit processes include secondary clarification, tertiary filtration via cloth media disk filters, and ultraviolet (UV) disinfection.

This Preliminary Engineering Report (PER) has been prepared in accordance with the requirements of R.61-67, *Standards for Wastewater Facility Construction*, for the proposed upgrades and capacity expansion from 8 to 16 mgd. Per the requirements of R.61-67, this PER includes a description of the proposed improvements, a wastewater characterization, a watershed characterization, a discussion of wastewater shutdown and bypass, and an alternatives analysis. Seven alternatives were evaluated in addition to the selected alternative.

1.2 Contact Information

<u>Owner Contact Information</u>

Larry Harper
Director, Water and Sewer Department
235 Deming Way
Summerville, SC 29483
(843) 832-0061

<u>Engineer Contact Information</u>

Mary E. Sadler, PE
Senior Associate, Hazen and Sawyer
4011 West Chase Blvd, Suite 500
Raleigh, NC 27607

1.3 Service Area

Dorchester County is encompassed entirely within the Greater Charleston Metropolitan Statistical Area (MSA). Dorchester County is adjacent to and northwest of Charleston County. The County is also bounded by Colleton County to the west, Berkeley County to the east, and Orangeburg County to the northwest. The Lower Dorchester service area consists of approximately 100 square miles. The Lower Dorchester WWTP is located in the southeastern portion of the service area. Figure 1-1 provides an illustration of the Lower Dorchester service area with respect to municipal limits, major roadways, rivers, and county boundaries.

1.4 Need for Improvements

1.4.1 Wastewater Flow Projections

The population in Dorchester County has demonstrated steady increases since 1960. In 2010, the County's population was reported at approximately 136,600 people, representing a 42 percent increase from the County's population in 2000. Since 2010, the growth in the County increased by 19 percent to approximately 162,809 people in 2019. Per Dorchester County's Comprehensive Plan (November 2018), the 2030 population in the County is anticipated to be slightly greater than 200,000 people.

Dorchester County continues to experience growth in the Lower Dorchester service area. The projected wastewater flow in the Lower Dorchester service area is anticipated to exceed the Lower Dorchester WWTP permitted capacity of 8 mgd over the planning period. The County's Planning Director was consulted in the fall of 2019 for information on the timing and location of the County's near and long term growth. Table 1-1 provides a summary of the anticipated near and long-term growth and associated average and peak flow delineated by planning year. Figure 1-2 provides an illustration of the service area growth by planning year. An additional 8 mgd of capacity is needed over the planning period to satisfy the anticipated growth, which is commensurate with the County's Comprehensive Plan. Included in the anticipated planned growth are firm commitments for approximately 10 mgd of capacity at the Lower Dorchester WWTP by the year 2025 and firm requests from the development community for an additional 5 mgd of capacity by the year 2030.

The County's Planning Department provided projected equivalent residential units (ERUs) based on available information from the development community. The near-term 3-year development is located primarily in previously developed areas. The near-term 5-year development (e.g., between 2020 and 2025) is anticipated to be located on the west side of the Ashley River near the County's existing collection system infrastructure. The 2025 to 2035 development is located along Highway 27 towards Ridgeville. The 2040 to 2050 growth is anticipated to be located farther west toward the Edisto River.

The County identified a location for a proposed Central Dorchester WWTP with an initial capacity of 4 mgd and a future expansion to 8 mgd in the Pine Hill Business Campus area west of the Ashley River. The location of the proposed Central Dorchester WWTP is proximate to the anticipated growth after 2035. The PER for the proposed Central Dorchester WWTP is located in Appendix A (W.K. Dickson, 2019). The proposed Central WWTP is discussed in this Lower Dorchester PER for two reasons. First, the County proposes to re-distribute the County's ultimate oxygen demand (UOD) wasteload allocation between the Lower and proposed Central treatment facilities. Second, the County wishes to pursue the NPDES permit for the proposed Central Dorchester WWTP in the event that growth in the service area occurs faster than the growth currently anticipated in the planning period. The Lower Dorchester WWTP site is constrained and will only support an ultimate capacity of 16 mgd.

1.4.2 Proposed Lower Dorchester WWTP Improvements

The proposed Lower Dorchester WWTP expansion project will provide several improvements to increase the reliability of the treatment process. In lieu of mechanical aeration, the existing oxidation ditches will be converted to a plug-flow five-stage configuration with diffused aeration and blowers. Additionally, two new plug-flow conventional activated sludge basins with a plug flow five-stage configuration are required for the expansion from 8 to 16 mgd. A new preliminary treatment facility and additional secondary clarification will be required. The UV disinfection facility will be replaced with newer and more reliable technology to meet the stringent enterococci limits. The electrical distribution system will be also be re-designed to address the increase in electrical load for the expansion and improve the reliability and redundancy of the plant electrical system. The electrical improvements include a new generator to serve the entire facility. Additionally, an expansion on the Lower Dorchester WWTP site requires that flow distribution (e.g., influent, return activated sludge, mixed liquor suspended solids, and secondary effluent) be carefully considered due to the stringent effluent permit limits.

Table 1-1: Summary of Anticipated Residential Units by Planning Year with Associate Average and Peak Flow

Planning Year	Planning Window	Number of Planned Residential Units [2]	Flow Anticipated in Planning Year Window		Cumulative Flow Anticipated in Planning Year Window		Anticipated Flow to Lower Dorchester WWTP	
			Average Flow, mgd [3]	Peak Flow, mgd	Average Flow, mgd [4]	Peak Flow, mgd	Total Average Cumulative Flow, mgd	Total Peak Cumulative Flow, mgd
2019	Average annual flow [1]	21,644	----	----	----	----	6.8	15.62
2023	0 to 3 years	2,100	0.53	1.31	0.53	1.31	7.33	16.93
2025	3 to 5 years	10,099	2.52	6.31	1.02	7.62	9.85	23.24
2030	5 to 10 years	7,620	1.91	4.76	4.95	12.39	11.75	28.01
2040	10 to 20 years	6,700	1.68	4.19	6.63	16.57	13.43	32.19
2050	20 to 30 years	10,750	2.69	6.72	9.32	23.29	16.12	38.91

[1] *Annual average flow for 2019.*
[2] *Data provided by Dorchester County's Director of Planning in the fall of 2019.*
[3] *The approved unit contributory loading for the Lower Dorchester WWTP is 250 gallons per day per ERU.*
[4] *A peaking factor of 2.5 was assumed per historical County peaking factors.*

1.5 NPDES Permit Requirements

1.5.1 Current NPDES Permit Requirements

The Lower Dorchester WWTP is permitted to discharge 8 mgd of treated effluent into Coosaw Creek in the Santee watershed via NPDES permit SC0038822. The NPDES permit includes tiered flow limits for 10 and 12 mgd. The current NPDES permit expired in January 2019. The permit renewal application has been submitted to the South Carolina Department of Health and Environmental Control (DHEC) in accordance with the required regulatory timeline. The 2018 permit renewal application did not include a request to expand the WWTP from 8 to 16 mgd. However, an update to the 2018 renewal application to include a major modification for the expansion to 16 mgd was submitted with this PER in 2020.

A summary of the Lower Dorchester WWTP monthly and weekly average effluent permit limits are provided in Table 1-2. Flow is not a regulated parameter in the permit, although the average design flows are referenced as 8, 10, and 12 mgd. The current annual average flow to the facility is approximately 6.5 mgd. The NPDES permit contains effluent permit limits for five-day carbonaceous biochemical oxygen demand ($cBOD_5$), total suspended solids (TSS), ammonia (NH_3-N), enterococci, dissolved oxygen (DO), and pH. Monitoring and reporting is required for total nitrogen, total copper, and total mercury.

The NPDES permit for the Lower Dorchester WWTP includes effluent limits requirements for UOD at annual average design flows of 10 and 12 mgd. The permit does not contain UOD limits at a flow of 8 mgd. The UOD was developed as a result of the Charleston area total maximum daily load (TMDL) to address dissolved oxygen impairment in the Charleston Harbor and Cooper, Wando, and Ashley Rivers (DHEC, 2013). The TMDL was implemented with a revision supported by a new hydrodynamic model in 2013.

Table 1-2: Summary of Lower Dorchester NPDES Permit Limits for 8, 10, and 12 mgd

		8 mgd		10 mgd		12 mgd	
Parameter	**Units**	**Monthly Average**	**Weekly Average**	**Monthly Average**	**Weekly Average**	**Monthly Average**	**Weekly Average**
Flow [1]	mgd	----	----	----	----	----	----
Carbonaceous biochemical oxygen demand, five day ($cBOD_5$)	mg/L	7.0	10.5	9.6	14.4	9.0	13.5
Ammonia (Mar – Oct)	mg/L	0.8	1.2	1.92 [2]	2.88 [2]	1.92 [2]	2.88 [2]
Ammonia (Nov – Feb)	mg/L	1.6	2.4	4.65 [2]	6.98 [2]	4.65 [2]	6.98 [2]
Total suspended solids (TSS)	mg/L	22.5	33.75	30	45	30	45
Dissolved oxygen	mg/L	> 5		> 5		> 5	
pH	s.u.	6.5 – 8.5		6.5 – 8.5		6.5 – 8.5	

Table 1-2: Summary of Lower Dorchester NPDES Permit Limits for 8, 10, and 12 mgd

		8 mgd		10 mgd		12 mgd	
Parameter	**Units**	**Monthly Average**	**Weekly Average**	**Monthly Average**	**Weekly Average**	**Monthly Average**	**Weekly Average**
Total nitrogen	----	Monitor and report		Monitor and report		Monitor and report	
Total copper	----	Monitor and report		Monitor and report		Monitor and report	
Total mercury	----	Monitor and report		Monitor and report		Monitor and report	
Enterococci	MPN / 100 mL	35	104 (daily max)	35	104 (daily max)	35	104 (daily max)
Whole effluent toxicity, chronic	----	25%	40% (daily max)	25%	40% (daily max)	25%	40% (daily max)
Ultimate oxygen demand, lb/d, Mar – Oct				2,082 lb/d		2,365 lb/d	
Ultimate oxygen demand, lb/d, Nov – Feb	----			3,550 lb/d		4,126 lb/d	

[1] Flow is not a regulated parameter in the Lower Dorchester WWTP NPDES permit, although the average design flows are referenced in the permit as 8, 10, and 12 mgd.
[2] Concentration limits are whole effluent toxicity based.

Table 1-2 also provides a summary of the UOD limits, the UOD-based ammonia concentration limit, the UOD-based $cBOD_5$ concentration limit, and the ammonia toxicity limit. The UOD-based concentration limits for ammonia and $cBOD_5$ are calculated by DHEC to be the maximum concentration that may be discharged at any time. With ammonia and $cBOD_5$ evaluated simultaneously, the permitted UOD equals 5 milligrams per liter (mg/L) of $cBOD_5$ and 0.8 mg/L of ammonia that may be discharged at a design flow of 12 mgd. It should be noted that the UOD may vary with fluctuations in either $cBOD_5$ or ammonia. Per the NPDES permit, the UOD is calculated as follows:

$$\text{UOD (lb/d)} = (2.22 \times cBOD_5 \times \text{Design Flow} \times 8.34) + (4.57 \times NH_3\text{-N} \times \text{Design Flow} \times 8.34)$$

1.5.2 Future NPDES Permit Requirements

The County requested a wasteload allocation from DHEC for a discharge for the proposed Central Dorchester WWTP in February 2018. Wasteload allocation for three discharge locations were requested to facilitate planning and land acquisition for the proposed treatment facility. DHEC stated in a response letter (June 2018) that the proposed Central Dorchester WWTP discharge would be incorporated into the Charleston area TMDL for the Charleston Harbor and Cooper, Wando, and Ashley Rivers. Location #3 for the Central Dorchester WWTP, Ashley River at Highway 17 Alt. Bridge, was determined to be a feasible discharge location per DHEC water quality modeling. However, DHEC stated that a pound per pound allocation of UOD load from another treatment facility on the Ashley River would be required to maintain the TMDL. The speculative limits for discharge location #3 also included a stringent monthly average ammonia concentration limit of 0.5 mg/L due to uncertainty in DHEC's water quality model results.

The County proposes to re-distribute the Lower Dorchester UOD wasteload allocation between the Lower Dorchester WWTP expansion from 8 to 16 mgd and the proposed Central Dorchester WWTP. Table 1-3 provides a summary of the Lower Dorchester and proposed Central WWTP capacity combinations at a UOD of 2,365 pound per day (lb/d) in the summer months and 4,126 lb/d in the winter months per the Lower Dorchester NPDES permit. For each capacity combination, a maximum $cBOD_5$ and ammonia concentration threshold was calculated. The ammonia and $cBOD_5$ concentration matrix reflects a stringent speculative limit ammonia concentration of 0.5 mg/L at discharge location #3 for the proposed Central Dorchester WWTP. DHEC indicated in a meeting in October 2018 that flexibility may be granted for the nitrification limit in the winter months November through February, as these months are not applicable to the TMDL. For a 16 mgd Lower Dorchester WWTP and a 4 mgd proposed Central Dorchester WWTP, the concentration limits would be 5.0 mg/L $cBOD_5$ and 0.8 mg/L of ammonia for the Lower Dorchester WWTP and 3.0 mg/L $cBOD_5$ and 0.5 mg/L of ammonia for the proposed Central Dorchester WWTP.

During the speculative limit and wasteload allocation discussions, DHEC indicated that future nutrient limits are a possibility in the Charleston Harbor and Cooper, Wando, and Ashley Rivers. The timing of the proposed nutrient limits was unclear; however, nutrient limits are not anticipated in the next few NPDES permit cycles. The County has agreed to construct a five-stage biological process for the Lower Dorchester WWTP expansion and the Central Dorchester WWTP in anticipation of future nutrient limits.

Table 1-3: Summary of Flow, Ultimate Oxygen Demand Combinations, and Concentration Thresholds for Lower Dorchester WWTP Expansion and Proposed Central WWTP

WWTP Capacity Combinations		Lower Dorchester WWTP		Proposed Central Dorchester WWTP		
Lower WWTP	Central WWTP	$cBOD_5$ Limit, mg/L [2]	Ammonia Limit, mg/L	$cBOD_5$ Limit, mg/L [2]	Ammonia Limit, mg/L [3, 4]	Total UOD, lb/d
Summer UOD of 2,365 lb/d [1]						
12 mgd	8 mgd	5.0	0.8	5.0	0.5	2,363
16 mgd	4 mgd	5.0	0.8	3.0	0.5	2,265
16 mgd	8 mgd	3.0	0.8	5.0	0.5	2,263
Winter UOD of 4,126 lb/d [1]						
12 mgd	8 mgd	5.0	< 2.5	5.0	0.8	3,232
16 mgd	4 mgd	5.0	< 2.5	5.0	0.8	3,495
16 mgd	8 mgd	5.0	< 2.5	5.0	0.8	3,984

[1] *Per the NPDES Permit and Fact Sheet for Lower Dorchester WWTP issued by SCDHEC, 2013.*
[2] *Assumed f-ratio of 2.22 from Lower Dorchester WWTP.*
[3] *Speculative ammonia concentration limit per DHEC wasteload allocation correspondence for the proposed Central WWTP at discharge location #3, Ashley River at Highway 17 Alt. Bridge (June 2018) at an assumed f-ratio of 2.2.*
[4] *DHEC indicated that the 0.5 mg/L concentration threshold would be applicable for the summer months March through October. DHEC indicated that the winter month (November through February) ammonia threshold could be increased to at least 0.8 mg/L, and possibly greater than 0.8 mg/L, to provide flexibility for cold-temperature nitrification.*

1.6 Receiving Water Characterization

The Cooper River subbasin, 8-digit hydrologic unit code (HUC) 03050201 and 03050202, is located within the Santee River Basin. The subbasin encompasses an area of approximately 1,545 square miles (DHEC, 2013). Cooper River is formed at the confluence of the East Branch Cooper River and West Branch Cooper River. Cooper River receives drainage from Back River, Goose Creek, Wando River, and Ashley River. Cooper River drains into Charleston Harbor.

The nearest Ashley River United States Geological Survey (USGS) gage station upstream of the Lower Dorchester WWTP discharge is Station 021720825 – Ashley River below Summerville, South Carolina. This station is located approximately 4 miles upstream of the WWTP discharge point. The station has been in operation since January 2017 and is affected by tide, which results in a twice-daily fluctuation of approximately 6 feet.

Per Section 303(d) of the Clean Water Act (CWA), if a surface water quality standard is exceeded and the impaired waters do not have a United States Environmental Protection Agency (EPA) approved TMDL, then an integrated reporting category of “5” is assigned to those waters and the waters are incorporated into the 303(d) list. The 303(d) list is sent to EPA and incorporated into the *National Water Quality Inventory Report*, which is provided to Congress every two years. The South Carolina 303(d) impairment list, published in 2016, indicates two locations on the Ashley River in Dorchester County upstream of the Lower Dorchester WWTP and two locations on the Ashley River in Charleston County immediately downstream of the WWTP as an impaired water. These impairments include recreational uses due to enterococcus at one Dorchester County monitoring station and both identified Charleston County stations, fish community due to mercury at one Dorchester County station, and aquatic life use due to pH and turbidity at one Charleston County station. Additionally, the Lower Dorchester WWTP is located within a TMDL watershed for dissolved oxygen. Figure 1-3 illustrates the location of the 303(d) listed surface water impairment stations by category.

Figure 1-4 provides an illustration of the NPDES dischargers in the portion of the Santee Basin within Dorchester County. In the described area, there are three industrial dischargers, one municipal discharger, and one domestic discharger upstream of the Lower Dorchester WWTP. There are six industrial discharge permits for a single industrial entity discharging into the Ashley River within five miles downstream of the Dorchester County WWTP.

1.7 Summary of Industrial Contribution

The Lower Dorchester WWTP influent wastewater is comprised of primarily domestic (e.g., residential and commercial) sources. As of December 2019, nine significant industrial users (SIUs) are permitted to discharge to the Lower Dorchester WWTP via Industrial Use Permits (IUPs) issued by the County. Non-categorical SIUs are subject to local limits via the County’s Pretreatment Program. Three of the nine industries are designated as categorical per 40 CFR 403 and are subject to local and categorical limits. The County does not receive liquid hauled waste at the Lower Dorchester WWTP.

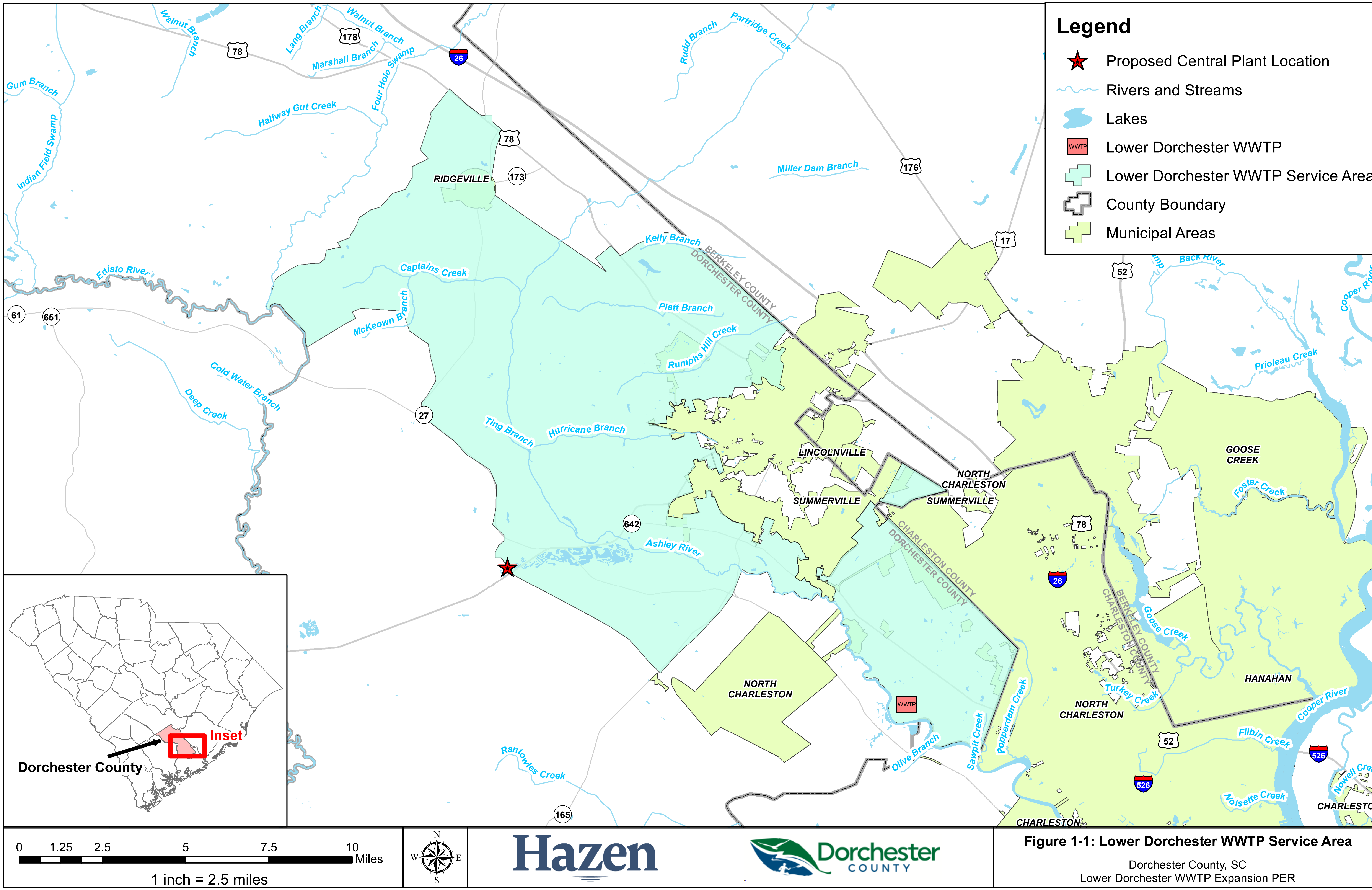


Figure 1-1: Lower Dorchester WWTP Service Area

Dorchester County, SC

Lower Dorchester WWTP Expansion PER

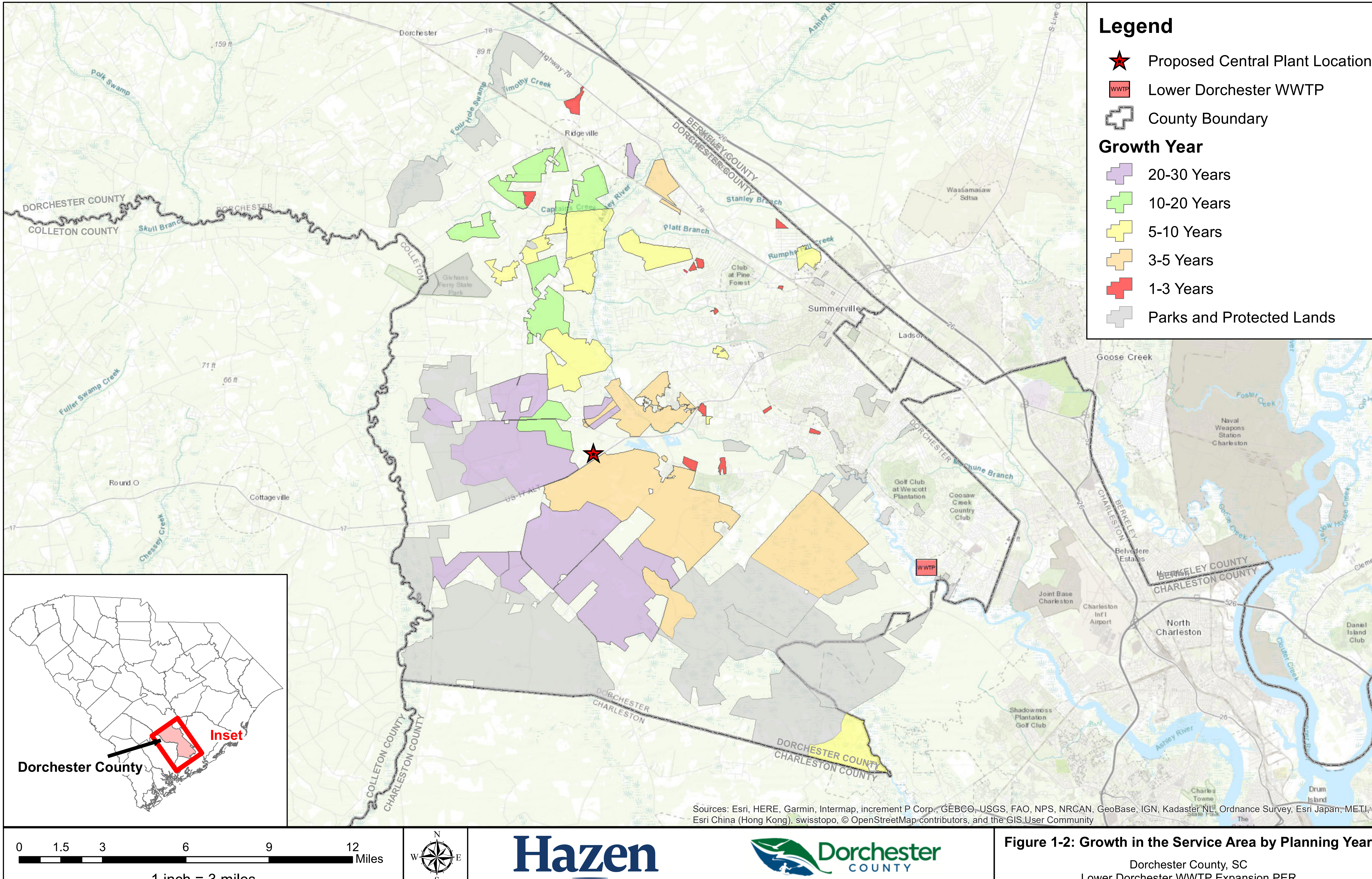


Figure 1-2: Growth in the Service Area by Planning Year

Dorchester County, SC
Lower Dorchester WWTP Expansion PER

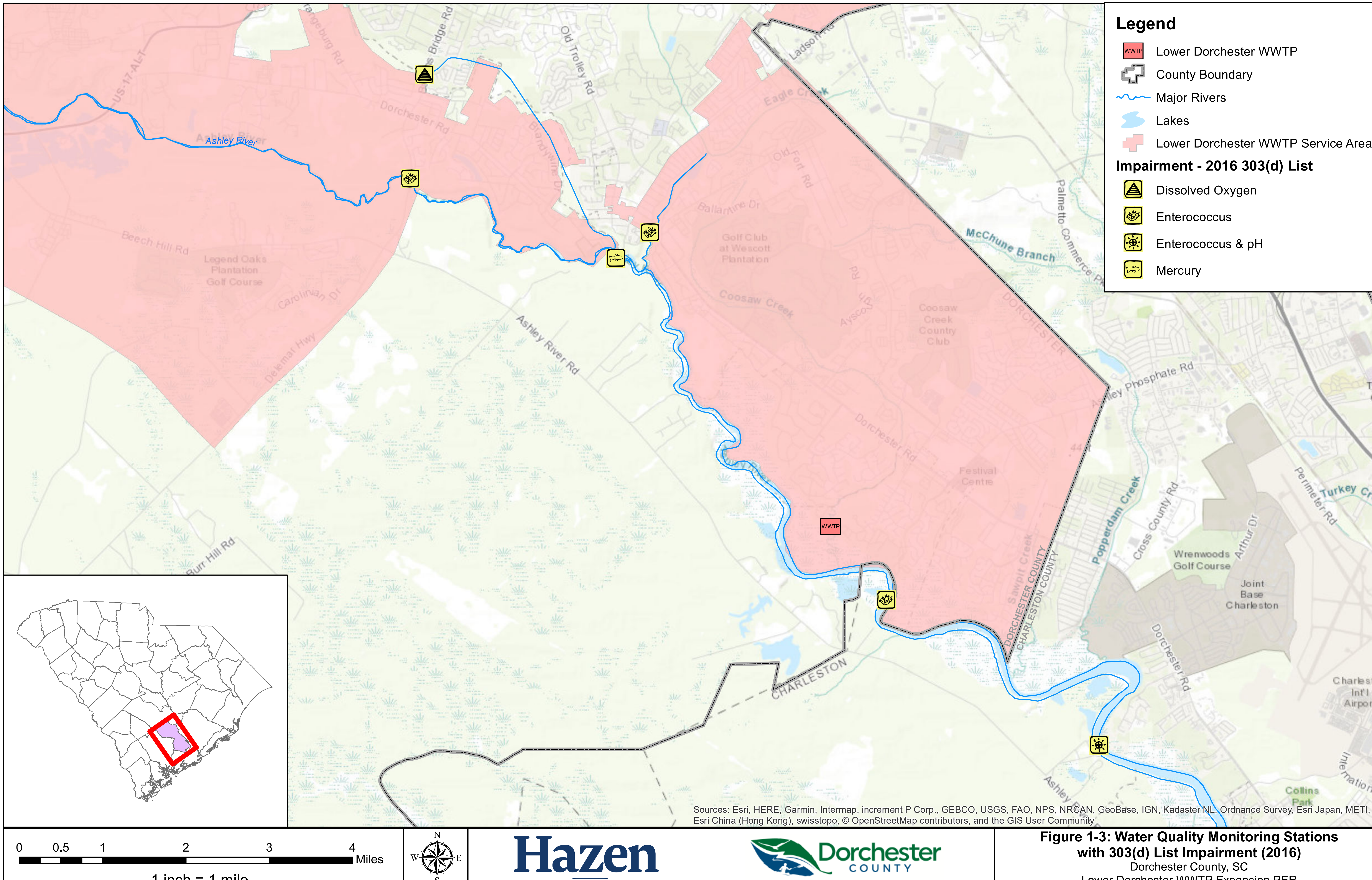
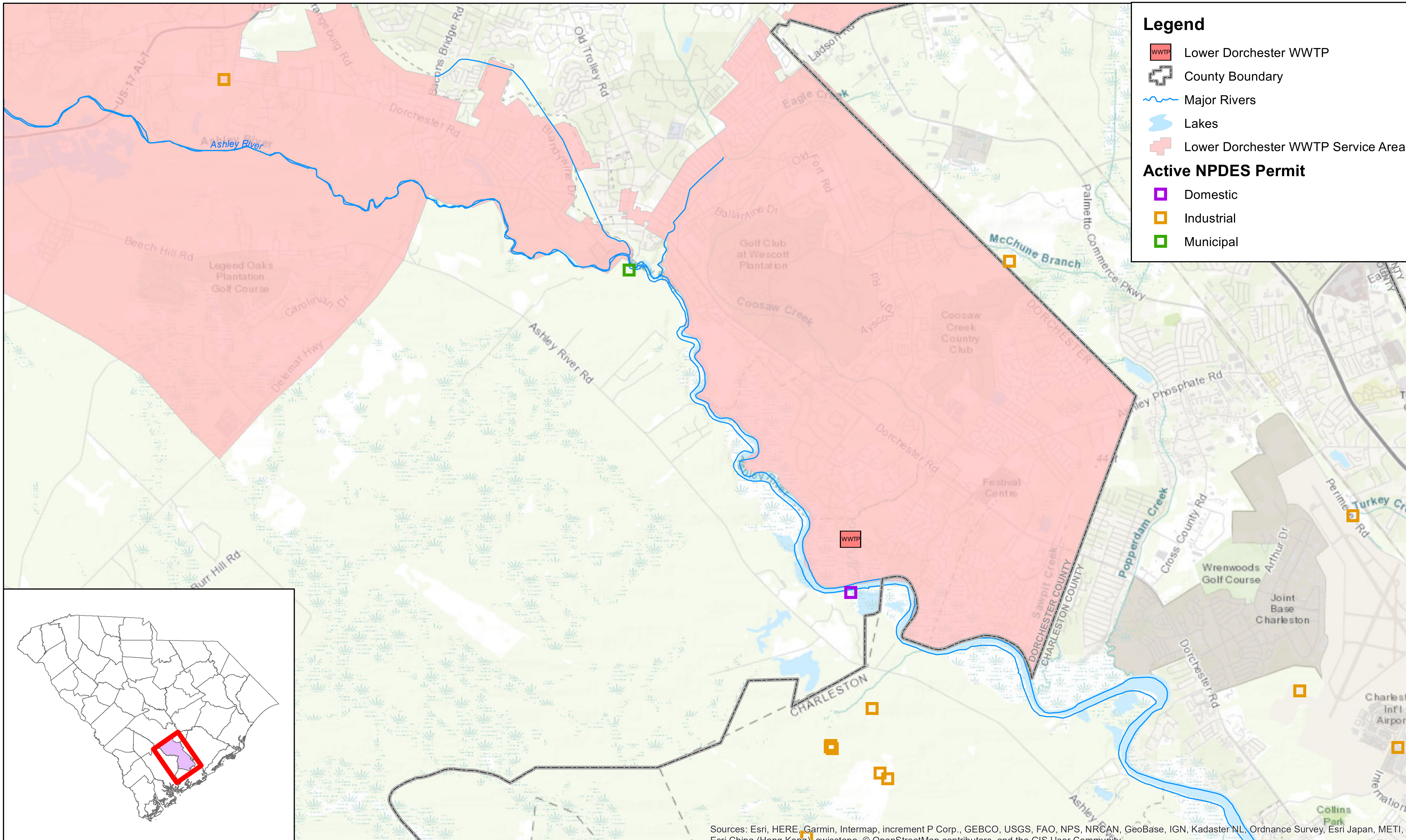


Figure 1-3: Water Quality Monitoring Stations with 303(d) List Impairment (2016)

Dorchester County, SC

Lower Dorchester WWTP Expansion PER



0 0.5 1 2 3 4 Miles

1 inch = 1 mile

Hazen

Dorchester COUNTY

Figure 1-4: Active NPDES Permitted Dischargers

Dorchester County, SC

Lower Dorchester WWTP Expansion PER

2. Summary of Existing Treatment Facilities

The Lower Dorchester WWTP has an average design capacity of 8 mgd and a rated hydraulic peak hour flow capacity of 20 mgd. The existing WWTP is designed for conventional activated sludge treatment using oxidation ditch technology for cBOD removal and ammonia oxidation followed by tertiary filtration. Filtered effluent is disinfected by UV disinfection followed by cascade aeration. Waste activated sludge (WAS) is stored in aerobic sludge holding tanks followed by dewatering via centrifuge. Solids are conveyed to a landfill for final disposal. A summary of the existing Lower Dorchester WWTP unit processes is as follows:

- Influent pump stations
- Fine screening and grit removal facility
- Oxidation ditches with mechanical aeration
- Secondary clarifiers
- Return activated sludge pump stations
- Cloth media disk filters
- UV disinfection
- Aerated sludge holding
- Dewatering via centrifuge

The following sections provide a brief summary of the liquid and solids unit processes at the Lower Dorchester WWTP. Table 2-1 provides a summary of design criteria for the existing liquid unit processes. Table 2-2 provides a summary of design criteria for the existing solids unit processes

2.1 Liquid Treatment Facilities

Raw influent wastewater is pumped to the preliminary treatment facility via two main influent pump stations. One influent pump station is located at the Lower Dorchester WWTP site (e.g., pump station) and the second pump station is less than a mile from the WWTP (pump station). The WWTP's on-site pump station 2 is equipped with three submersible pumps at a firm capacity of 17.3 mgd. Off-site influent pump station 1 is equipped with three submersible pumps and one standby pump with a firm capacity of 16 mgd. This pump station has the ability to bypass pump station 2 by pumping influent directly to the preliminary treatment facility.

The preliminary treatment facility consists of three mechanical fine screens followed by four vortex grit removal units. The screens are the band screen type with 3 millimeter (mm) openings. Screenings are conveyed to a screenings compactor. Grit is pumped to classifiers for dewatering and hauled off-site for disposal. After grit removal, flow is conveyed to a splitter box to distribute flow to the aeration basins 3 and 4 via two 36-inch pipes. Aeration basins 1 and 2 have been removed from service.

Secondary treatment is provided in an activated sludge process via two EIMCO Carousel® oxidation ditches with bridge-mounted mechanical surface aerators. The facility operates in an extended aeration/complete mix mode. The oxidation ditch equipment includes submersible mixers, gates, dissolved oxygen monitors and automatic control equipment. The aeration basin volume is approximately 6 million gallons (MG). Mixed liquor from aeration basins 3 and 4 is conveyed to the clarifier division box where it splits to secondary clarifiers 3 through 8.

There are eight secondary clarifiers. Clarifiers 1 and 2 are not operational and are not needed for compliance. Clarifiers 3 and 4 are 65 feet in diameter and clarifiers 5 through 8 are 100 feet in diameter. All clarifiers are center-feed clarifiers of the riser-pipe type. As part of the activated sludge process, all return activated sludge (RAS) from the secondary clarifiers is recycled to the first zone of aeration basins 3 and 4. RAS pump station 2 conveys RAS from clarifiers 3 and 4. RAS pump station 3 conveys RAS from clarifiers 5 and 6. RAS pump station 4 conveys sludge from clarifiers 7 and 8.

Secondary effluent is conveyed to tertiary cloth media disk filters followed by UV disinfection. Clarified effluent from clarifiers 3 and 4 is conveyed to the filter influent pump station to lift to the tertiary filters. Secondary effluent from clarifiers 5, 6, 7, and 8 flows by gravity to the disk filters. The Lower Dorchester WWTP has six disk filters in operation. After filtration, flow is disinfected using UV light prior to flow measurement and discharge. Four banks of UV lamps are installed in two separate channels. Flow is conveyed through an effluent Parshall flume. Treated effluent is discharged to Coosaw Creek via a 48-inch gravity sewer and two 24-inch gravity sewers.

Table 2-1: Summary of Design Criteria for Existing Liquid Treatment Infrastructure

Unit Process	Parameter	Unit	Design Criteria	To Be Demolished
Fine Bar Screens				
	Number	----	3	Yes
	Number of channels	----	1	
	Capacity, each	mgd	15	
	Capacity, firm (N+1)	mgd	30	
	Screen size	mm	3	
	Angle of inclination	degree	45	
	Type	----	Mechanical	
	Motor	HP	2	
Screenings Compactor				
	Number of units	----	1	Yes
	Diameter	inches	10	
	Motor	HP	5	
On-site Influent Pump Station No. 2				
	Number of pumps	----	3	
	Type	----	Submersible	
	Model	----	XFP 351M-CH3	

Table 2-1: Summary of Design Criteria for Existing Liquid Treatment Infrastructure

Unit Process	Parameter	Unit	Design Criteria	To Be Demolished
	Total dynamic head	feet	54	
	Unit flow capacity	gpm	6,020	
	Unit head capacity	feet	54.0	
	Design efficiency	%	81.2	
	Unit brake horse power	HP	101.1	
	Drive type	----	Variable speed	
Off-site Influent Pump Station No. 1				
	Number of pumps	----	4	
	Type	----	Submersible	
	Capacity, each	gpm	5,600	
	Total dynamic head	feet	68	
	Capacity, each	mgd	8.06	
	Firm capacity	mgd	16	
	Motor	HP	140	
	Drive type	----	Variable speed	
Influent Flow Measurement (Influent Pump Station No. 2)				
	Number, total	----	1	
	Type (pump No. 1-3)	----	Magnetic	
	Capacity range	mgd	0 – 30	
Grit Removal				
	Number of units	----	4	Yes
	Number of equipment installed	----	1	
	Capacity, per unit	mgd	7	
	Diameter	feet	10	
	Type	----	Vortex	
	Headloss	inch	1.2	
Grit Pumps				
	Number of units	----	4	Yes
	Type	----	Vertical, closed-coupled, vacuum primed	
	Capacity	gpm	250	
	Motor	HP	10	
Grit Concentrator				
	Number	----	4	Yes
	Speed	RPM	250	
	Dewatering (min)	%	93 – 94	

Table 2-1: Summary of Design Criteria for Existing Liquid Treatment Infrastructure

Unit Process	Parameter	Unit	Design Criteria	To Be Demolished
Aeration Basins No. 3 and 4				
	Number	----	2	
	Type	----	EIMCO Carousel®	
	Length	feet	348	
	Width	feet	66	
	Depth	feet	16.75	
	Total volume	MG	5.3	
	HRT	hours	16	
Aeration System				
	Aeration system type	----	Mechanical surface	Yes
	Number of aerators, per basin	----	4	
	Motor	HP	125	
Mixing System				
	Type	----	Submersible	Yes
	Number of anoxic mixers, per basin	----	2	
	Motor, anoxic	HP	7.7	
	Number of anaerobic mixers, per basin	----	6	
	Motor, anaerobic	HP	2.3	
Secondary Clarifiers 1 and 2 (not in use)				
	Diameter, each	feet	65	Yes
	Sidewater depth, each	feet	12	
	Surface area, each	SF	3,318	
Secondary Clarifiers 3 and 4				
	Diameter, each	feet	65	Yes
	Sidewater depth, each	feet	12	
	Surface area, each	SF	3,318	
Secondary Clarifiers 5, 6, 7, and 8				
	Diameter, each	feet	100	
	Sidewater depth, each	feet	16.2	
	Surface area, each	SF	7,854	
RAS Pump Station No. 1 (not in use)				
	Secondary clarifiers served	----	1 and 2	Yes
	Number of pumps	----	3	
	Type	----	Non-clog centrifugal	
	Capacity, each	gpm	700	
	Firm capacity	mgd	2	

Table 2-1: Summary of Design Criteria for Existing Liquid Treatment Infrastructure

Unit Process	Parameter	Unit	Design Criteria	To Be Demolished
RAS Pump Station No. 2				
	Secondary clarifiers served	----	3 and 4	Yes
	Number of pumps per station	----	3	
	Type	----	Non-clog centrifugal	
	Capacity, each	gpm	700	
	Firm capacity	mgd	2	
RAS Pump Station No. 3				
	Secondary clarifiers served	----	5 and 6	
	Number of pumps, total	----	3	
	Number of pumps, duty	----	2	
	Number of pumps, spare	----	1	
	Type	----	Self-priming centrifugal	
	Capacity, each	gpm	1,660	
	Capacity, each	mgd	2.39	
	Firm capacity	mgd	4.78	
RAS Pump Station No. 4				
	Secondary clarifiers served	----	7 and 8	
	Number of pumps, total	----	3	
	Number of pumps, duty	----	2	
	Number of pumps, spare	----	1	
	Type	----	Self-priming centrifugal	
	Capacity, each	gpm	1,660	
	Capacity, each	mgd	2.39	
	Firm capacity	mgd	4.78	
Total RAS Pump Station Capacity				
	Total firm capacity	mgd	13.6	
Filter Influent Pump Station				
	Number of pumps	----	4	Yes
	Type	----	Submersible	
	Capacity, each	gpm	3,170	
	Firm capacity	mgd	13.69	
Tertiary Filters				
	Number	----	6	
	Type	----	Disk	
	Disks per unit	----	12	
	Filtration rating	um	10	
	Active filter depth	mm	3 – 5	

Table 2-1: Summary of Design Criteria for Existing Liquid Treatment Infrastructure

Unit Process	Parameter	Unit	Design Criteria	To Be Demolished
	Submerged disk surface area, each	SF	53.8	
	Total surface area	SF	269	
	Hydraulic loading rate at design flow	gpm/SF	3.25	
Backwash Storage Tank (Old Chlorine Contact Chamber)				
	Number	----	1	Yes
	Volume	gal	7,500	
Backwash Return Pumps				
	Number	----	2	Yes
	Type	----	Centrifugal	
	Capacity, each	gpm	130	
Ultraviolet Disinfection Facilities				
	Total number of banks	----	4	Yes
	Number modules per bank	----	3	
	Number lamps / module	----	14	
	Total number of lamps	----	168	
	Watts per lamp	W	401.2	
	Total kW per channel	kW	33.7	
	Peak flow capacity	mgd	20.0	
	UV dose at 65% UVT	mJ/cm^2	31	
	Level control	----	Actuated downward opening gate	
	Inlet baffle plate	----	Yes	
	Channel dimensions	----	31 feet x 50.79 inches x 62.04 inches	
	Water level at lamps	inches	35.43	
Effluent Flow Measurement				
	Type	----	Parshall Flume	Yes
	Throat width	inches	24 nested with a 36	
	Capacity range	mgd	0 to 30	
Reuse Water Transfer System				
	Tank volume	gal	7,500	Relocated
	Number of pumps	----	3	
	Type	----	Vertical turbine	
	Capacity, each	gpm	380	
	Motor	HP	40	
	Air compressor motor	HP	5	

Table 2-1: Summary of Design Criteria for Existing Liquid Treatment Infrastructure

Unit Process	Parameter	Unit	Design Criteria	To Be Demolished
Reuse Storage Tank				
	Number	----	1	
	Volume	MG	1	
Reuse Pump Station				
	Number of pumps	----	2	
	Capacity, each	gpm	1,800	
	Firm capacity	gpm	1,800	
	Firm capacity	mgd	2.6	
Standby Generators				
	Number	----	3	
	Capacity, generator 1	kW	1,100	Yes
	Capacity, generator 2	kW	300	Yes
	Capacity, generator 3	kW	150	

2.2 Solids Handling Facilities

WAS is discharged from each of the RAS lines through a meter vault and conveyed to a 1.6 million gallon (MG) sludge holding (decant) basin. Solids are thickened by decanting in the sludge holding basin. Downstream of the sludge holding basin, sludge is pumped via the sludge transfer station to one of two aerobic digesters. Oxygen is supplied by blowers via coarse bubble diffused aeration. Thickened sludge is then conveyed to the sludge dewatering facility. Sludge feed pumps convey sludge to two decanting centrifuges (DCENs) for dewatering. Upstream of the centrifuges, a polymer solution is fed and mixed with the thickened sludge for the dewatering process. The second centrifuge, feed pump, and polymer system were added in 2020. After dewatering, sludge is transported to the Oak Ridge Landfill in Dorchester County for final disposal. The vacuum assisted sludge drying beds are not operational and are not needed for compliance.

Table 2-2: Summary of Design Criteria for Existing Solid Treatment Infrastructure

Unit Process	Parameter	Unit	Design Criteria	To Be Demolished
Sludge Holding (Decant) Basin				
	Number	----	1	
	Min volume	MG	0.230	
	Max volume	MG	1.25	
	Aeration system type	----	Positive displacement blowers with fine bubble diffused aeration	

Table 2-2: Summary of Design Criteria for Existing Solid Treatment Infrastructure

Unit Process	Parameter	Unit	Design Criteria	To Be Demolished
	Number of blower	---	3 (2 duty, one spare)	
	Blower motor	HP	125	
Aerobic Sludge Digesters				
	Number	----	2	Yes
	Digester No. 1 depth	feet	30	
	Digester No. 1 diameter	feet	30	
	Digester No. 1 volume	MG	0.075	
	Digester No. 2 diameter	feet	50	
	Digester No. 2 volume	MG	0.264	
	Total volume	MG	0.34	
	Aeration system	----	Coarse bubble diffused air	
	Blower type	----	Positive displacement	
	Digester No. 1 blower motor	HP	75	
	Digester No. 2 blower motor	HP	30	
Solids Transfer Pump Station				
	Number of pumps	----	2	
	Type	----	Self-priming centrifugal	
	Capacity, each	gpm	680	
	Horsepower	HP	10	
Sludge Feed Pumps, Dewatering Building				
	Number of pumps	----	2	
	Type	----	Rotary lobe	
	Capacity, each	gpm	200	
	Horsepower	HP	10	
Polymer Feed Pumps				
	Number	----	2 (one spare)	
	Type	----	Positive displacement, progressing cavity	
	Neat polymer metering capacity	gph	3.3 – 13	
	Dilution water capacity	gpm	5 – 50	
Centrifuge				
	Number	----	2	
	Percent solids	%	21	
	Solid capture efficiency	%	95	
	Polymer dosage	lb/ton active	20	
	Solids loading rate (dry)	lb/hr-meter	2,000	

Table 2-2: Summary of Design Criteria for Existing Solid Treatment Infrastructure

Unit Process	Parameter	Unit	Design Criteria	To Be Demolished
	Hydraulic loading rate	gpm	200	
Ultimate Solids Disposal				
	Method of disposal	----	Sanitary Landfill	----

3. Alternatives Analysis

Eight wastewater capacity alternatives were evaluated to address the Dorchester County's wastewater capacity needs for 16 mgd over the next planning period. Per Section 1.3, the Lower Dorchester WWTP is currently permitted for an annual average flow of 8 mgd with a tiered permit limit and wasteload allocation of 12 mgd. This alternatives analysis addresses the additional 4 mgd of capacity that will be required to meet the needs of the service area over the planning period. The alternatives that were considered in this analysis include the following:

- No-action
- Land application
- Water reuse
- Use of other surface water discharge locations
- Connection to other Publicly-Owned Treatment Works (POTWs)
- Infiltration and inflow (I/I) reduction
- Expansion of the Lower Dorchester WWTP from 8 to 12 mgd and the proposed Central Dorchester WWTP with new NPDES discharge to Ashley River (second preferred alternative)
- Expansion of the Lower Dorchester WWTP from 8 to 16 mgd with an increased NPDES discharge to Coosaw Creek (preferred alternative)

3.1 No-action

Under the no-action alternative, the County would not construct the necessary wastewater treatment capacity to meet the growth needs over the planning period. Privately owned wastewater treatment package systems and septic systems would proliferate in the service area in the absence of adequate centralized treatment capacity. The EPA estimates that between 10 and 70 percent of all septic systems in the United States are failing and are a common source of water contamination (EPA, 2013). By dramatically increasing the number of septic systems in the area, the risk of failing septic systems causing a decline in water quality also increases.

The no-action alternative does not address growth needs in the service area. Growth will continue in the Lower Dorchester WWTP service area leading to an increased probability of a decline in water quality from underperforming septic systems and the proliferation of privately owned wastewater treatment package plants. Therefore, this alternative was eliminated from further consideration.

3.2 Land Application

Land application of treated wastewater effluent was identified as an alternative to an increase in the surface water discharge to Coosaw Creek. Land application systems include individual or community onsite subsurface systems, drip irrigation, and spray irrigation. Land application systems do not require advanced secondary treatment processes prior to irrigation and also do not facilitate other options for effluent disposal, such as reuse. A land application treatment system would be designed to reduce biochemical oxygen demand (BOD_5) to 10 mg/L, ammonia to 2 mg/L, nitrate to 10 mg/L, and TSS concentrations to less than 30 mg/L. The Lower Dorchester WWTP effluent quality currently meets land application standards. This alternative would require the secondary treatment and effluent disposal of the additional 4 mgd required for the WWTP expansion. Screening and grit removal, activated sludge, clarification, and pumping facilities would be located at the Lower Dorchester WWTP site and secondary effluent would be pumped to a storage pond on the land application site(s) for distribution via spray irrigation.

In accordance with South Carolina R.61-9, *Water Pollution Control Permits*, the design loading rates for land application systems range between 0.5 and 2 inches per week (in/wk) based on the depth to the seasonal high groundwater level. The median loading rate is 1.0 in/wk and is associated with a groundwater level four feet below the land application surface. For this analysis, a conservative loading rate of 1.0 in/wk was used. Given a wastewater flow of 4 mgd and a land application loading rate of 1.0 in/wk, the minimum area required to be dedicated for land application is approximately 1,030 acres. However, the actual land that would be required must account for stream buffers, occupied building buffers, property boundary buffers, storage ponds, and areas that have greater than 10 percent slope or inadequate soil conditions. When all of these additional considerations are included, it is estimated that approximately 1,900 acres would be required for a 4 mgd land application system. A summary of the estimated land requirements and calculation methods is provided in Table 3-1.

Table 3-1: Summary of Estimated Land Requirements for Land Application Alternative

Parameter	Value	Calculation Method
Design flow rate	4 mgd	Projected capacity increase
Maximum land application rate	1.0 in/wk	Median rate from R.61-9
Area required for average day flow (ADF)	1,030 acres	Design flow rate / maximum land application rate
Storage pond capacity	60 days	Accommodate variations in flow
Storage pond depth [1]	6 feet	Average depth not including freeboard and precipitation allowance
Area required for storage pond	184 acres	$\frac{\text{(Design Flow × Storage Pond Capacity)}}{\text{Storage Pond Depth}}$
Area for building and property boundary buffer and unsuitable soil/slope conditions	304 acres	25% of ADF area and storage pond area
Area for stream and wetland buffers	365 acres	30% of ADF area and storage pond area
Total area required	1,900 acres	ADF area + storage pond area + buffer and unsuitable conditions areas

[1] *Assumed a groundwater table depth of four feet.*

A geographic information system (GIS) analysis was performed using Dorchester County parcel data in order to determine if any tracts of land would be suitable for land applying 4 mgd of treated wastewater. This evaluation did not consider parcels located in adjacent counties. Parcels greater than 500 acres were evaluated as potential land application sites to limit the number of properties that would require condemnation. Additionally, only rural parcels currently zoned as AC (Absence of Controls) were identified as potential land application sites. Sites that are currently zoned for heavy industrial use or are owned by the state or federal government for parkland were removed from consideration. Parcels with large percentages of area in the Federal Emergency Management Agency (FEMA) 100-year floodplain were also eliminated as potential land application locations. The remaining rural parcels were then grouped by contiguous adjacent land areas. Only one group of parcels, located to the west of the Ashley River and the Lower Dorchester WWTP, was identified as a potential land application location after the application of screening criteria. Table 3-2 summarizes the total land area available and approximate distance from the Lower Dorchester WWTP for this identified group of contiguous parcels.

The two parcels that make up the identified potential land application location are owned by large timber and forest operations. The condemnation of these lands is anticipated to be extremely difficult. The costs associated with the infrastructure required to build a land application system in either of these areas is also expected to be prohibitive. A preliminary cost evaluation for this alternative results in a total present worth of approximately $413 million. Therefore, the land application alternative was removed from consideration as a viable project alternative.

Table 3-2: Summary of Potential Land Application Location in Dorchester County

Location of Parcel Group	Number of Parcels	Total Land Area [1]	Estimated Distance from Lower Dorchester WWTP
West of Ashley River between State Highways 165 and 61	2	6,660 acres	5.5 miles

[1] *Includes land that may not be suitable for land application due to buffer requirements, slope, or soil condition.*

3.3 Water Reuse

Water reuse systems were evaluated as a discharge alternative for the proposed treatment capacity expansion of the Lower Dorchester WWTP. Water reuse, also referred to as reclaimed water, is the beneficial use of highly treated wastewater effluent for opportunities other than direct discharge to surface waters. Per R.61-9, DHEC defines reclaimed water as a method of advanced wastewater treatment designed to produce an effluent of high quality suitable for irrigation in areas with public contact, such as yard irrigation and public open spaces. Non-conjunctive reuse is defined as a wastewater treatment system that relies on reclaimed water uses to account for all of the generated wastewater (i.e., zero direct discharge to surface water). To increase the capacity of the Lower Dorchester WWTP to meet the anticipated wastewater flow of 16 mgd without an increasing to the surface water discharge, a 4 mgd non-conjunctive reuse system would be required.

Design criteria for reclaimed water systems are more stringent than for land application systems, and therefore require a higher level of treatment prior to effluent disposal. The treatment system would be designed for a monthly average BOD_5 and TSS of 5 mg/L with a weekly average of 7.5 mg/L. Effluent nitrate would be monitored and reported. A turbidity limit would be applied in specific circumstances. Disinfection requirements would be applied per R.61-9.122. The Lower Dorchester WWTP currently meets reuse standards for treatment.

If a dedicated land application system were implemented as a non-conjunctive reuse option, the land requirements would be similar to a traditional land application system. If the level of wastewater treatment meets the requirements of R.61-9.505.45(i), property buffers and extensive storage is not required. Approximately 1,500 acres would be required for a dedicated reuse land application system using identical application rates of a traditional land application system. Per the conclusions in Section 3.2, the potential acquisition of identified properties and the high infrastructure costs of a dedicated land application system also remove this option from consideration at a total present worth of approximately $385 million.

Another non-conjunctive reuse alternative is conveying reuse water for industry use. The Lower Dorchester WWTP currently does not have any industrial contributors with an average daily flow in excess of 1 mgd. For dedicated reuse to be effective, high water use industrial customers are needed to ensure that an average daily flow 4 mgd of reuse water can be accepted on a year-round basis. The current permitted flows of all the significant industrial dischargers to the Lower Dorchester WWTP combined is less than 1 mgd. The use of existing industrial users is not effective or practical based on the lack of high-volume industrial water use.

In addition to industries that discharge wastewater directly to the Lower Dorchester WWTP collection system, industries that have individual NPDES permits were identified as potential reuse partners. A GIS analysis was performed to identify all individual industrial NPDES permitted discharges located within Dorchester County that have a permitted discharge greater than 1 mgd are within a 25-mile radius of the Lower Dorchester WWTP. Only one industry, Showa Denko Carbon, was located within the search area and it had a 2018 average facility flow of 0.159 mgd (EPA ECHO Database, 2018), which is not adequate for a dedicated reuse partnership for the acceptance of 4 mgd of reuse water. Therefore, a non-conjunctive recycling and reuse alternative was eliminated from further consideration.

3.4 Use of Other Surface Water Discharge Locations

The option of re-locating the Lower Dorchester WWTP outfall was discussed in a meeting with DHEC, Dorchester County staff, and Hazen staff in October 2018. The advantages and disadvantages of the relocating the outfall were discussed. The outfall is located within a mile of the confluence with the Ashley River. The County would not be able to obtain additional UOD capacity if the outfall were to be relocated. However, the County would gain dilution capacity for other pollutants (e.g., metals), which would provide the potential for an increase in the maximum allowable headworks load for the County’s Pretreatment Program. It was concluded that the outfall should remain in the current location, as property acquisition and environmental permitting issues would be a significant concern.

3.5 Connection to other Publicly-Owned Treatment Works

An alternative for wastewater capacity is through partnerships with neighboring wastewater service providers. For regionalization to be a viable alternative, a regional partner or partners must be available that have sufficient excess wastewater capacity to serve both the anticipated wastewater flow of the Lower Dorchester WWTP service area in addition to their own anticipated wastewater flow. Potential wastewater treatment facilities were identified within a 25-mile radius of the Lower Dorchester WWTP.

Table 3-3 provides a summary of potential regional wastewater service partners and associated permitted discharge capacity, discharge location, and estimated distance from the Lower Dorchester WWTP. To be a viable alternative as a regional partner, neighboring WWTPs must have adequate capacity to accept the anticipated wastewater flow of 4 mgd from the Lower Dorchester WWTP service area in addition to meeting the anticipated flow in their respective service areas. The potential identified partners have NPDES discharge permits ranging from 0.3 mgd to 36 mgd. When an additional 4 mgd is added to the current average daily facility flows, four POTW treatment facilities are in excess of 100 percent available capacity. The remaining POTW treatment facility capacities range between 63 and 94 percent available capacity. A few treatment facilities would be close to the 80 percent threshold to start planning an expansion via a submittal of a PER DHEC requirements.

Two POTWs were contacted for the possibility of leasing 4 mgd of treatment capacity to the County. The Town of Summerville currently has a 10 mgd treatment facility with an average day flow of 5.4 mgd. The treatment facility is expandable to 14 mgd. The Town has available UOD to sustain a WWTP expansion and meet effluent discharge limits. The Town offered a contract cost of $3.00 per 1,000 gallons treated at a 2 percent annual inflation rate (W.K. Dickson, 2019). The total present worth for the County to lease capacity from Summerville is approximately $319 million.

The North Charleston Sanitary District (NCSD) was also contacted regarding a long-term lease of 4 mgd capacity. The NCSD treatment facility is rated at 34 mgd with an average day flow of 17 mgd. NCSD provided a contract cost of $5.615 per 1,000 gallons treated at a 2 percent annual inflation rate (W.K. Dickson, 2019). The total present worth for the County to lease capacity from NCSD is approximately $371 million.

Regionalization is not a viable wastewater capacity alternative. An expansion of the Lower Dorchester WWTP from 8 to 12 mgd and an additional 4 mgd of capacity leased from a neighboring community's wastewater treatment facility is required to meet the growth needs in the County for the planning period. There is a high probability that the County will not be able to purchase the 4 mgd of capacity in a neighboring POTW treatment facility. Regionalization with leased capacity in lieu of purchased capacity does not represent a long term wastewater solution for Dorchester County.

Table 3-3: Summary of Potential Regional Wastewater Service Partners for the Dorchester County Water and Sewer Department

Municipality / Wastewater Service Provider	Treatment Facility	Current NPDES Permitted Discharge [1]	Current Average Day Flow [2]	Percent of Capacity with 4 mgd Dorchester Flow	Estimated Distance from Lower Dorchester WWTP
Town of Summerville	Summerville WWTF	10 mgd	5.4 mgd	94%	8.2 miles
Berkeley County Water and Sanitation	Lower Berkeley WWTF	22.5 mgd	12.6 mgd	74%	14.2 miles
North Charleston Sewer District	Felix C Davis WWTP	34 mgd	17.3 mgd	63%	15.5 miles
Charleston Water	Plum Island WWTP	36 mgd	23.9 mgd	78%	17.5 miles
Town of Mt. Pleasant	Rifle Range Road and Center Street WWTPs	9.7 mgd	8.6 mgd	> 100%	21.3 miles
Berkeley County Water and Sanitation	Central Berkeley WWTF	6 mgd	0.4 mgd	74%	22 miles
Town of Sullivan's Island	Sullivan's Island WWTF	0.57 mgd	0.5 mgd	> 100%	23 miles
Town of Moncks Corner	Moncks Corner WWTF	3.2 mgd	1.1 mgd	> 100%	24.7 miles
Isle of Palms Water and Sewer Commission	Forest Trail WWTP	0.3 mgd	0.18 mgd	> 100%	25 miles

[1] *Data from BCDCOG 208 Water Quality Plan Update, 2011, Volume II.*
[2] *Data from EPA Enforcement and Compliance History Online (ECHO) Database, https://echo.epa.gov/. Facility flow average represents data between January 2019 through March 2020.*

3.6 Infiltration and Inflow Reduction

Dorchester County is actively addressing I/I issues in the collection system. The County conducts regular inspections and maintenance on the collection system conveyance to limit the impact of I/I into the wastewater system. The 4 mgd of capacity needed to address growth in the planning areas cannot be accounted for in I/I reduction efforts in the collection system. Therefore, the I/I reduction alternative has been eliminated from further consideration as a stand-alone alternative to the proposed project. However, the County will continue on-going efforts to promote I/I reduction.

3.7 Expansion of Lower Dorchester WWTP from 8 to 12 mgd and Proposed Central Dorchester WWTP and new NPDES Discharge to Ashely River (Second Preferred Alternative)

Dorchester County has explored the option of constructing a new Central Dorchester WWTP in the Pine Hill Business Campus area to meet the growth needs of the County. The proposed Central WWTP would be constructed as a 4 mgd conventional treatment facility expandable to 8 mgd. Section 1.3.2 provides a summary of the County's wasteload allocation request from DHEC for a new discharge for the proposed Central WWTP. The Ashley River at Highway 17 Alt. Bridge was determined to be a feasible discharge location per DHEC water quality modeling. However, DHEC stated in the speculative limits letter that a pound per pound allocation of UOD load from another treatment facility on the Ashley River would be required to maintain the Charleston area TMDL for UOD. A discussion of the UOD distribution between the Lower and Central Dorchester WWTPs is provided in Section 1.3.2.

The County purchased property in the Pine Hill Business Campus for the location of the proposed Central Dorchester WWTP. The location of the proposed Central Dorchester WWTP is proximate to the anticipated growth in the service area after 2035. The PER for the proposed Central WWTP was prepared as a separate document from this PER for the Lower Dorchester WWTP expansion project and is provided in Appendix A. The Central Dorchester WWTP PER also includes the DHEC response documentation for the wasteload allocation request.

The County proposes to pursue an NPDES permit for a discharge to the Ashley River for future construction of the Central Dorchester WWTP. Build-out capacity on the Lower Dorchester WWTP site is 16 mgd. The County wishes to pursue the NPDES permit for the proposed Central Dorchester WWTP in the event that growth in the service area occurs faster than the growth currently anticipated in the planning period. Additionally, the County may consider using the Central WWTP property to construct future influent equalization prior to construction of the Central Dorchester WWTP, should equalization be necessary to control peak flow to either the Lower Dorchester WWTP or the proposed Central WWTP. The total present worth for this alternative is $314 million.

3.8 Expansion of Lower Dorchester WWTP from 8 to 16 mgd with an Increased NPDES Discharge to Coosaw Creek (Preferred Alternative)

The proposed capacity expansion of the Lower Dorchester WWTP from 8 to 16 mgd was evaluated as a viable alternative to meet the growth needs in the service area. The Lower Dorchester NPDES permit includes tiered flow limits for 10 and 12 mgd. The proposed expansion to 16 mgd will require an expanded surface water discharge to Coosaw Creek. Increasing the Lower Dorchester WWTP effluent discharge to 16 mgd is not anticipated to cause an impact to Coosaw Creek. Per Section 1.3.2, the County proposes to maintain the regulated UOD at the limiting threshold of 2,365 lb/d in the summer months. For a 16 mgd Lower Dorchester WWTP and a 4 mgd proposed Central WWTP, the concentration limits would be 5.0 mg/L $cBOD_5$ and 0.8 mg/L of ammonia for the Lower Dorchester WWTP and 3.0 mg/L $cBOD_5$ and 0.5 mg/L of ammonia for the proposed Central WWTP.

An expansion of the Lower Dorchester WWTP with a surface water discharge to Coosaw Creek was identified as the most viable alternative for the proposed 8 mgd wastewater treatment capacity increase to meet the growth needs in the service area. This alternative capitalizes on the County's existing investment in wastewater treatment infrastructure. Furthermore, this alternative is more economical than the other identified alternatives. In addition to the expansion of the Lower Dorchester WWTP, existing facility improvements are required for improved reliability and rehabilitation and/or replacement of facilities with limited remaining life or hydraulic restrictions. Section 4 provides a discussion of the basis of design for the Lower Dorchester WWTP expansion. The total present worth of this alternative is approximately $242 million.

3.9 Summary of Total Present Worth of Alternatives

Table 3-4 summarizes the total present worth for each of the evaluated alternatives. The preferred alternative is the most economical for the County's rate-payers. The second preferred alternative includes the option of constructing a new Central Dorchester WWTP during the next planning period. However, the County may use the Central Dorchester WWTP site for equalization during this planning period. The County also wishes to pursue the NPDES permit for the proposed Central Dorchester WWTP in the event that growth in the service area occurs faster than the growth currently anticipated. The total present worth calculations for the alternatives are provided in Appendix B.

Table 3-4: Summary of Total Present Worth of Alternatives

Alternative	Capital Cost Opinion [1]	Total Present Worth of O&M [2, 3]	Present Worth of Salvage [4]	Total Present Worth	Comment
No Action	NA	NA	NA	NA	Alternative does not address growth needs in the service area
Land Application Alternatives:					
Lower Dorchester WWTP expansion from 8 to 16 mgd and land application of 4 mgd partially treated effluent	$261,000,000	$176,000,000	$24,000,000	$413,000,000	High cost alternative, low probability of timely land acquisition [5, 6]
Lower Dorchester WWTP expansion from 8 to 16 mgd and land application of 4 mgd reclaimed water	$234,000,000	$176,000,000	$25,000,000	$385,000,000	High cost alternative, low probability of timely land acquisition [5, 6]
Use of other surface water discharge locations	NA	NA	NA	NA	Property acquisition and environmental issues a significant concern
Connection to other POTWs:					
Lower Dorchester WWTP expansion from 8 to 12 mgd and Summerville contract to Lease 4 mgd of treatment capacity	$122,000,000	$218,000,000	$21,000,000	$319,000,000	Alternative does not represent a long term wastewater solution [7]
Lower Dorchester WWTP expansion from 8 to 12 mgd and North Charleston Sewer District Contract to lease 4 mgd of treatment capacity	$123,000,000	$278,000,000	$30,000,000	$371,000,000	Alternative does not represent a long term wastewater solution [8]

Table 3-4: Summary of Total Present Worth of Alternatives

Alternative	Capital Cost Opinion [1]	Total Present Worth of O&M [2, 3]	Present Worth of Salvage [4]	Total Present Worth	Comment
Inflow and Infiltration Reduction	NA	NA	NA	NA	Alternative does not address growth needs in the service area
Expanded or New Surface Water Discharges:					
Lower Dorchester WWTP expansion from 8 to 12 mgd and Central Dorchester WWTP at 4 mgd	$153,000,000	$190,000,000	$29,000,000	$314,000,000	Alternative provides a long term wastewater solution for County [9]
Lower Dorchester WWTP expansion from 8 to 16 mgd	$103,000,000	$162,000,000	$23,000,000	$242,000,000	Alternative provides a long term wastewater solution for County at lower total present worth than any of the alternatives [10]

[1] All costs in 2019 dollars with a contingency of 30%.
[2] Time period 20 years and a discount rate of 0.4% per 2020 Discount Rates for OMB Circular No. A-94, M-20-7.
[3] O&M costs from 2020 Water and Sewer Rate Study (Hazen and Sawyer). O&M costs include contractual services, maintenance and repairs, utilities, office and truck expenses, miscellaneous charges, supplies, collection expenses, sewer line rehabilitation, building improvements, and vehicle maintenance. O&M costs for the proposed Central WWTP were based on a proportional cost per million gallons treated with a start-up flow of 2 mgd.
[4] Aggregate structural / mechanical / electrical life assumed to be 40 years.
[5] Assumes $30,000 per acre to acquire land from established timber company.
[6] Assumes $30,000 per acre for spray irrigation system.
[7] Summerville CPW estimate of $3.00 per 1,000 gallons treated inflated at 2% per year. Start-up flow of 2 mgd.
[8] North Charleston Sewer District estimate of $5.615 per 1,000 gallons treated inflated 2% per year. Start-up flow of 2 mgd.
[9] Capital costs for proposed Central WWTP from W.K. Dickson Preliminary Engineering Report Appendix A (2019).
[10] Capital costs includes 3% for bonds and insurance, 20% contractor overhead and profit, and 7% for taxes.

4. Basis of Design for Selected Alternative

The existing Lower Dorchester WWTP infrastructure was evaluated for an expansion to 16 mgd. An expansion of the Lower Dorchester WWTP is necessary to prepare for the anticipated growth service area. The following sections provide a summary of the process design criteria, the design criteria for the liquid treatment facilities (e.g., influent pumping through disinfection), the design criteria for the solids handling facilities, and a discussion of the necessary upgrades to the electrical infrastructure.

4.1 Process Design Criteria

A basis of design was developed to establish the appropriate design criteria for new or expanding wastewater treatment facilities per standard engineering practice. Hazen and Sawyer developed a biological process model for the Lower Dorchester WWTP to establish process volumes, aeration requirements, and solids production for the plant expansion to 16 mgd. Influent data, effluent data, and process operating records were provided by County staff.

4.1.1 Flow and Load Characterization

Limited influent flow data have been collected at the Lower Dorchester WWTP. Data for $cBOD_5$ and TSS were provided from January 2014 through April 2018. Sampling frequency was initially limited to approximately once a month and increased to weekly sampling in July 2016. For the basis of design, the $cBOD_5$ and TSS loads were based on historical influent data from July 2016 through April 2018. Chemical oxygen demand (COD) and ammonia loads were based on historical data from August 2016 through April 2018. Influent TKN concentration data were collected from October 2018 through December 2018 and averaged approximately 33 mg/L.

Table 4-1 summarizes the reported average influent flow, concentrations, and loads for $cBOD_5$, COD, TSS, and ammonia. Recycle streams are returned upstream of the influent sample point. The influent flow to the WWTP averages approximately 6.2 mgd. Influent flow has ranged from less than 5 mgd to 12 mgd. The Lower Dorchester WWTP experienced an extreme peak flow of approximately 23 mgd during a storm event in October 2015. Influent $cBOD_5$, COD, TSS, and ammonia concentrations are within a typical range for a municipal treatment facility treating primarily domestic strength wastewater. Table 4-2 summarizes the historical influent flow and load peaking factors.

4.1.2 Design Wastewater Temperature

Plant operating data was compiled to evaluate the minimum (e.g., cold weather) temperature for process simulations. County staff collects effluent temperature in lieu of activated sludge temperature. The historical effluent temperature data was reviewed to establish a minimum temperature for process modeling and aeration system design. Table 4-3 summarizes the temperature statistics for the Lower Dorchester WWTP. A minimum 7-day temperature of 16°C was selected for secondary process design.

Table 4-1: Summary of Influent Flow, Concentration, and Load

Parameter	Flow, mgd	Concentration, mg/L	Load, lb/d
Flow [1]	6.2	----	----
$cBOD_5$ [2]	----	182	9,410
COD [2]	----	379	19,600
TSS [2]	----	215	11,100
Ammonia [2]	----	25	1,300

[1] Average of influent data from January 2014 to April 2018.
[2] Average of influent data from July 2016 to April 2018.

Table 4-2: Summary of Historic Influent Flow and Load Peaking Factors

Flow Criteria	Flow, mgd [1]	Peaking Factor				
		Flow [1]	$cBOD_5$ [2]	COD [2]	TSS [2]	NH_3-N [2]
Minimum day	2.0	0.32	0.35	0.09	0.35	0.15
Average annual	6.3	----	----	----	----	----
Maximum month (30-day)	7.4	1.18	1.50	1.34	1.62	1.30
Maximum 7-day	9.7	1.55	2.31	1.70	2.06	1.81
Maximum day	13.2	2.11	3.54	2.26	2.30	2.59

[1] Average of influent data from January 2014 through April 2018.
[2] Average of influent data from July 2016 through April 2018.

Table 4-3: Historical Effluent Temperature

Flow Criteria	Temperature, °C
Minimum day	14
Minimum 7-day	16
Average annual	23
Maximum month (30-day)	29
Maximum 7-day	29

4.1.3 Proposed Influent Loads and Peaking Factors

The majority of the influent flow is pumped to the WWTP. The County's pump stations are designed for a peak hour factor of 2.5. A peak hour flow factor of 2.5 was assumed for design of the WWTP expansion, which is consistent with the previous expansion project. Based on the County staff's operational experience, peak hour flow events are sustained for long periods of time (e.g., over 6 to 8 hours) during wet weather events.

Design influent loads were developed based on the Lower Dorchester WWTP's historical data and best professional judgment. Due to the limited amount of influent data, the design influent load peaking factors were selected based on typical ratios for domestic wastewater treatment plants. The selected design influent flow and load peaking factors for five-day biochemical oxygen demand (BOD_5), TSS, ammonia, and total Kjeldahl nitrogen (TKN) are summarized in Table 4-4. Table 4-5 summarizes the design influent concentration and load at the design maximum month, maximum week, and maximum day flows.

Table 4-4: Summary of Design Peaking Factors for Influent Flow and Loads

Parameter	Flow	$cBOD_5$	TSS	Ammonia	TKN
Maximum month peaking factor	1.25	1.5	1.5	1.5	1.5
Maximum 7-day peaking factor	1.5	2.0	2.0	2.0	2.0
Maximum day peaking factor	2.5	2.5	2.5	2.5	2.5
Maximum hour peaking factor	2.5	----	----	----	----

Table 4-5: Summary of Design Influent Loads

Parameter	Flow, mgd	$cBOD_5$ mg/L	$cBOD_5$ lb/d	TSS mg/L	TSS lb/d	Ammonia mg/L	Ammonia lb/d	TKN mg/L	TKN lb/d
Annual average	16	200	26,690	190	25,350	26	3,470	40	5,340
Maximum month	20	240	40,030	228	38,030	31	5,200	48	8,000
Maximum 7-day	24	267	53,380	253	50,710	35	6,940	53	10,680
Maximum day	40	200	66,720	190	63,380	26	8,670	40	13,340

4.1.4 Process Evaluation

BioWin® Version 5.3 (Envirosim, LTD) was used to develop an uncalibrated process model to assist in evaluating the conversion of the current oxidation ditches to diffused air plug flow reactors and the installation of two new plug flow aeration basins. Influent wastewater characterization sampling was not performed, so default BioWin® influent fractions were used. Dynamic daily simulations were performed for the period of August 19, 2016 through May 1, 2018. The process model influent and effluent concentration results correlated well to the reported historical data. The process model oxidation ditch mixed liquor suspended solids (MLSS), mixed liquor volatile suspended solids (MLVSS), and solids production also correlated well.

The BioWin® model simulations demonstrated that the existing oxidation ditches are treatment-limited due to the insufficient aeration capacity of the existing mechanical surface aerators. With the existing mechanical surface aeration system, treatment capacity is limited to 3 mgd per basin to meet the stringent ammonia permit limit of 0.8 mg/L. Therefore, an average day flow greater than approximately 6 mgd will cause non-compliance with the stringent ammonia limit unless the oxidation ditch basin volume is increased or additional aeration capacity is provided. BioWin® model simulations verified that a conversion to a plug flow configuration with a diffused aeration system will increase the capacity of the existing basins to 4 mgd per basin within the existing basin volume.

Steady state simulations were used to evaluate a five-stage process configuration (e.g., plug flow) in addition to estimated nitrified recycle flow (NRCY) and WAS production. In a five-stage process, influent is conveyed to an anaerobic zone followed by anoxic, aerobic, post-anoxic, and aerobic process zones with a 3 to 4Q internal recycle (e.g., recycle from aerobic zone to anoxic zone). The post-anoxic zone in the five-stage process (e.g., the fourth zone) will be constructed as a swing zone, whereby mixing equipment and diffused aeration grids will be installed in this zone. The swing zone provides flexibility to convert to a three-stage process to manage seasonal variability, such as extreme cold temperatures or wet weather. In a three-stage process (e.g., A^2O), influent is conveyed to an anaerobic zone followed by anoxic and aerobic zones with a 3 to 4Q internal recycle (e.g., recycle from aerobic zone to anoxic zone).

Table 4-6 summarizes the predicted secondary process parameters, cake production, and final effluent quality for annual average load conditions at 16 mgd and 23°C, 16 mgd at 16°C, and maximum month conditions at 16 mgd and 16°C. The steady state process model simulations were conducted using the following assumptions:

- Influent split between basins of 50:50.
- All basins in service.
- Aerobic solids retention time (aSRT) of 8 days.
- Dissolved oxygen of 2.0 mg/L in the first aerobic zone and 1.0 mg/L in the second aerobic zone.
- Dissolved oxygen of 1.0 mg/L in the swing zone for the three-stage configuration simulations.
- RAS flow rate of 80 percent of plant influent flow.

It is recommended that the County proceed with conversion of the oxidation ditches to a plug flow five-stage configuration and construction of two new aeration basins for a total capacity of 16 mgd. The five-stage configuration will allow flexibility for the process to be optimized for future nutrient limits per the discussion in Section 1.3.2. The nutrient limit target is currently unknown pending DHEC water quality model development. The process modeling indicates that the recommended process volume and configuration may be able to achieve an effluent total nitrogen between 6 and 10 mg/L. A detailed influent and effluent wastewater characterization to validate model assumptions, pilot testing of carbon addition depending on the total nitrogen limit, and a process optimization study will be required to assess the capability of the Lower Dorchester WWTP to reliably achieve a specific effluent nitrogen target.

Table 4-6: Summary of Process Simulation Results with a Five-stage Process for 16 mgd Expansion

		16 mgd Annual Average Load and 23°C		16 mgd Annual Average Load and 16°C		16 mgd Maximum Month Load and 16°C	
Parameter	**Unit**	**NRCY Flow 32 mgd**	**NRCY Flow 48 mgd**	**NRCY Flow 32 mgd**	**NRCY Flow 48 mgd**	**NRCY Flow 32 mgd**	**NRCY Flow 48 mgd**
Influent flow	mgd	16	16	16	16	16	16
NRCY, % of influent	%	200	300	200	300	200	300
Anoxic effluent nitrate	mg/L	0.5	1.3	0.4	1.1	0.9	2.3
MLSS	mg/L	3,000	3,000	3,10	3,100	4,670	4,670
MLVSS	mg/L	2,200	2,200	2,270	2,260	3,430	3,440
RAS MLSS	mg/L	6,600	6,600	6,900	6,870	10,330	10,300
WAS flow	mgd	0.3	0.3	0.3	0.3	0.30	0.30
WAS solids	lb/d	16,600	16,500	17,260	17,210	25,970	25,930
Effluent COD	mg/L	24	24	24	24	35	35
Effluent BOD_5	mg/L	< 5	< 5	< 5	< 5	< 5	< 5
Effluent TSS	mg/L	< 5	< 5	< 5	< 5	< 5	< 5
Effluent ammonia	mg/L	< 0.3	< 0.3	< 0.5	< 0.5	< 0.5	< 0.5
Effluent nitrate	mg/L	4.5	3.9	4.4	3.7	6.7	6.2
Effluent TKN	mg/L	1.5	1.5	1.8	1.8	2.2	2.2
Effluent total nitrogen	mg/L	6.0	5.4	6.2	5.6	9.2	8.6
Effluent total phosphorus	mg/L	3.5	3.5	3.3	3.3	1.2	1.3

4.1.5 Summary of Aeration Basin Design Criteria

Table 4-7 provides a summary of the aeration basin design criteria for the five-stage process configuration. The tabulated volumes assume all basins were in service. The hydraulic retention time (HRT) assumes an annual average flow of 16 mgd with each basin receiving 4 mgd of flow (e.g., the converted oxidation ditches receive 8 mgd and the new aeration basins receive 8 mgd).

Table 4-7: Summary of Aeration Basin Design Criteria for the Five-Stage Process Configuration at 16 mgd

Parameter	Units	Converted Oxidation Ditches	New Aeration Basins
Number of basins	----	2	2
Anaerobic volume	MG	0.4	0.4
Anoxic volume	MG	1.3	1.2
Aerobic volume	MG	2.5	2.8
Post anoxic volume	MG	0.9	1
Re-aeration volume	MG	0.2	0.2
Total volume	MG	5.3	5.5
Anaerobic HRT	hour	1.2	1.1
Anoxic HRT	hour	3.8	3.5
Aeration HRT	hour	7.6	8.3
Post anoxic HRT	hour	2.7	3
Re-aeration HRT	hour	0.7	0.6
Total HRT	hour	16	16.6

4.1.6 Estimated Solids Quantities for Dewatering and Disposal

BioWin® simulations were used to estimate solids quantities for the proposed expansion. The solids quantities are provided in Table 4-8 for the five-stage process configuration. Solids quantities were estimated on a dry weight basis at 16°C for the annual average condition. The BioWin® model predicted an annual average sludge production of 1,100 pound per million gallon (lb/MG) over the 16-month period evaluated; however, four months out of 16 months indicated a sludge production of 1,400 lb/MG. Therefore, a sludge production of 1,400 lb/MG was selected as the basis for design. Peaking factors for maximum month, maximum two-week, and maximum week solids production are based Hazen’s experience with municipal wastewater treatment facilities. A net yield of 0.67 pounds of solids per pound of BOD_5 removed and a secondary effluent solids concentration of 10 mg/L were used for estimated solids quantities.

Table 4-8: Summary of Estimated Solids Quantities

Annual Average Flow	Condition	Total Suspended Solids Production	Volatile Suspended Solids Production [2, 3]
16 mgd	Annual average	22,720	18,400
	Maximum month [1]	32,150	26,040
	Maximum two-week [1]	36,350	29,440
	Maximum week [1]	39,760	32,200
12 mgd	Annual average	17,040	13,800
	Maximum month [1]	24,110	19,530
	Maximum two-week [1]	27,260	22,100
	Maximum week [1]	29,820	24,150
8 mgd	Annual average	11,360	9,200
	Maximum month [1]	16,070	13,020
	Maximum two-week [1]	18,180	14,730
	Maximum week [1]	19,880	16,100

[1] *Maximum month, two week, and one week peaking factors based on Hazen and Sawyer's experience with municipal wastewater treatment facilities.*
[2] *Estimated fraction of volatile solids at 0.81 mg VSS/mg TSS.*
[3] *Estimated yield 0.65 mg TSS/mg BOD removed.*

4.2 Liquid Treatment Facility Design Criteria

This section provides a discussion of the expansion requirements for the Lower Dorchester WWTP new and existing necessary for the expanded flow of 16 mgd. All of the equipment selections discussed herein are preliminary and will be finalized during the final design phase. Figure 4-1 illustrates the process flow schematic for the liquid and solids facilities. The new and proposed facilities are indicated on the figure. Existing and proposed site plan figures are provided in Section 5.

4.2.1 Reliability Classification

DHEC establishes reliability classification requirements for wastewater treatment plants for the purpose of protecting surface waters. There are three levels of reliability classification. Classification I is the most restrictive. The Lower Dorchester WWTP has been designated as a Classification III facility. The proposed expansion of the Lower Dorchester WWTP will meet SCDHEC's Reliability Classification III as defined in Section 67.400 of the Standards for Wastewater Facility Construction, regulation R.61-67. The Class III reliability requirements mandate that backup components and auxiliary power be included in the facility design.

4.2.2 Proposed Demolition of Existing Infrastructure

Several unit processes on the Lower Dorchester WWTP site are permanently out of service or only operational in extreme circumstances, such as an extended peak wet weather event. These unit processes have been identified for demolition to reclaim site footprint for the new infrastructure required for the facility expansion. The structures proposed for demolition are showing signs of age-related corrosion, are not designed to current best engineering practices and standards, or their use interferes with the treatment process. All of the identified structures for demolition were constructed in the 1984 or 1994 expansion projects. A summary of the existing infrastructure proposed for demolition is provided in Tables 2-1 and 2-2 of this PER.

4.2.3 Hydraulic Profile

The recommended infrastructure will be designed to fit within the existing hydraulic profile. There are several changes to the process that will result in new hydraulic control points. An influent Parshall flume will be added to the process in addition to a new influent distribution box to mix preliminary treated influent and RAS and then split flow evenly between the four aeration basins. A new MLSS distribution box will be constructed to split flow evenly between the four existing and two new 100-foot diameter secondary clarifiers. The new MLSS distribution box will replace the existing hexagon-shaped distribution box that is undersized for the expanded flow. The new hydraulic profile also reflects the increases in process header line sizes that are requires to pass peak flow. A new tertiary effluent distribution box will be constructed to combine effluent flow from the filters for conveyance to disinfection.

The 100-year flood elevations were revised by the USGS and effective in Dorchester County in April 2017. The existing flood elevation of 8 feet mean sea level (MSL) was raised to a new flood elevation of

10 feet MSL at the outfall location. The new flood elevation results in the effluent outfall manholes being overtopped at peak flow for the expansion. The manholes must be raised to accommodate the new flood elevation.

4.2.4 Influent Pump Station

Raw influent wastewater is pumped to the preliminary treatment facility via two main influent pump stations. The off-site pump station 1 conveys 16 mgd to the WWTP via an existing 36-inch force main to the existing preliminary treatment facility. The second source of influent flow is the on-site influent pump station 2. The on-site influent pump station consists of three 100 horsepower (HP) submersible pumps each sized to convey 6,020 gallon per minute (gpm) at 54 feet of total dynamic head. The off-site pump station is equipped for a fourth pump slot to increase station capacity. These pumps convey flow through parallel 16-inch force mains to the existing preliminary treatment facility. Table 2-1 provides a summary of the existing design conditions for the two influent pump stations.

A new 30-inch force main is proposed to route flow from the on-site influent pump station to the new preliminary treatment facility. The existing submersible pumps were evaluated with the proposed 30-inch force main to determine impacts on operating capacity. The larger single 30-inch force main does not have a negative impact on the pump station capacity. Due to the larger equivalent pipe area of the proposed 30-inch force main, a small amount of additional capacity will be provided. The total system will have an anticipated operating range between 3,500 gpm (~5 mgd) and 13,000 gpm (~19 mgd).

Pump station 2 typically operates five or six minutes per hour due to the maximum turn-down of 3,500 gpm. The infrequent pump run times cause the influent flow to be pulsed into the WWTP instead of a gradual ramping up and down. The influent flow pulsing is causing significant issues with control of the activated sludge process and UV disinfection. Two Gorman-Rupp suction lift-type jockey pumps are recommended to capture the lower range of influent flow. The jockey pumps will be equipped with variable frequency drives (VFDs) for turn-down to approximately 1.8 mgd. The jockey pumps will be installed on the top slab of the existing influent pump station. A new influent pump station electrical building will house the VFDs and control panels for the new jockey pumps. The VFDs and control panels for the existing influent pumps will also be relocated to the new influent pump station electrical building.

4.2.5 Preliminary Treatment Facility

Fine screening, grit removal, influent flow measurement, combined influent and RAS distribution, and WAS pumping will be located in a new multi-purpose preliminary treatment facility. The new preliminary treatment facility will hydraulically pass a peak flow of 40 mgd. The existing preliminary treatment facility at the Lower Dorchester WWTP is hydraulically undersized to pass flow greater than 20 mgd. The new multi-level pretreatment facility will consist of elevated screens, compactors, and grit classifiers with grit pumps and dumpsters located at grade. Screened influent flow will be conveyed to one of two grit removal units followed by a Parshall flume for influent flow measurement. The new preliminary treatment facility will also house screenings consolidation and compaction equipment along with grit washing and classification units. Compacted screenings and classified grit will discharge by gravity to a series of dumpsters at grade level.

4.2.5.1 Screening

The screening recommended for the Lower Dorchester WWTP expansion are two 0.25-inch fine screens located upstream of grit removal. The fine screens will further protect downstream treatment units from unnecessary wear or process impairments. A high level screening evaluation was conducted during a site visit with County staff. The results of the discussion were used to determine which fine screening method would be most applicable for the Lower Dorchester WWTP. The results of this screening analysis determined that multi-rake screens would be the optimal screen selection and were carried forward for further evaluation.

Each screen will be dedicated to a single screenings compactor to reduce screenings disposal volume. Compacted screenings will be discharged into a chute, delivering the compacted screenings from the upper level of the preliminary treatment facility to collection dumpsters located on the ground level of the facility. Dumpsters will then be emptied by trucks at a ground level truck loading facility for off-site disposal. Table 4-9 provides a summary of the design criteria for the fine screens and compactors.

Table 4-9: Summary of Fine Screen and Compactor Design Criteria

Parameter	**Unit**	**Design Criteria**
Screen Channels		
Number of channels	----	3
Channel width	feet	4.5
Channel depth	feet	8.0
Screen channel flow minimum, per channel	mgd	6
Screen channel flow maximum, per channel	mgd	20
Maximum headloss, per screen	inch	11 [1]
Screenings removal rate, average day	CF/MG	7 [2]
Mechanically Cleaned Screen		
Number of units	----	2
Type	----	Multi-rake bar
Design capacity, per unit	mgd	20 mgd
Screen opening	mm	6
Angle of inclination	°	80
Channel width	feet	4.5
Motor	HP	1
Motor enclosure	----	Explosion proof
Manually Cleaned Screen		
Number of units	----	1

Table 4-9: Summary of Fine Screen and Compactor Design Criteria

Parameter	Unit	Design Criteria
Type	----	Manually cleaned
Bypass capacity	mgd	40
Width	feet	6
Opening between bars	inch	0.5
Fine Screenings Wet Quantities		
Minimum	CF/day	42
Average	CF/day	112
Maximum (instantaneous)	CF/day	280
Screenings Washer / Press		
Number of units	----	2
Input capacity, wet screenings	CF/hr	106
Motor	HP	7.5

[1] At 40% blind of screen.
[2] Average limit per Water Environment Federation Manual of Practice 8, Figure 11.2.

4.2.5.2 Grit Removal

The final treatment component of the preliminary treatment facility will be grit removal. Removing grit from the liquid treatment train in the preliminary treatment facility prevents excess deposition of grit in downstream aeration basins and clarifiers. Two grit removal technologies were evaluated to include the induced vortex and stacked tray types. Induced vortex grit facilities have a lower associated capital cost than stacked tray facilities; however, stacked tray systems have smaller overall footprint and have greater grit capture rate. The total present worth for both grit removal technologies is within 5 percent based on similarly sized equipment. The stacked plate grit system is newer to the wastewater industry and is a proprietary design supplied by Hydro International.

There are differences in capture rates between the induced vortex and stacked tray grit systems based on grit gradation typical of the southeastern U.S. An induced vortex grit system has a 95 percent removal of grit particles greater than 150 microns, which is equivalent to a capture rate of approximately 65 to 75 percent of total grit. Stacked tray systems have a 95 percent removal of grit particles greater than 75 microns, which is equivalent to an overall capture rate of 95 to 98 percent of total grit. The difference between the 75 and 150 micron particle size is the fraction of grit classified as sugar sand. The removal of sugar sand grit particles will benefit downstream unit processes, including wear on aeration diffusers, sludge pumping, and less overall plant maintenance costs.

The stacked tray grit system is recommended for the Lower Dorchester WWTP expansion to include grit pumping, grit cyclones, and grit classifiers. Table 4-10 provides a summary of the design criteria for the grit removal system. Two grit units are recommended for the plant expansion to 16 mgd. Grit will be pumped to a cyclone and classifier to remove grit from the liquid treatment stream. Classified grit will be directed to a collection chute with a gravity discharge into a dumpster at grade elevation for collection and off-site disposal.

Table 4-10: Summary of Grit Removal System Design Criteria

Parameter	Unit	Design Criteria
Screen Channels		
Average grit loading	CF/MG	5 [1]
Average grit volume at average design flow	CF/day	80
Peak grit loading	CF/MG	20 [1]
Peak grit volume	CF/day	800
Peak grit volume	CY/hr	1.2
Grit slurry, per unit	gpm	430
Grit Removal Unit		
Number of units	----	2
Type	----	Stacked tray
Design capacity, per unit	mgd	20
Tray diameter	feet	12
Number of trays, per unit	----	10
Removal at 95%, average day flow	micron	106
Grit Pumps		
Number of units	----	2
Type	----	Horizontal, recessed impeller
Rated capacity	gpm	430
Total dynamic head	feet	55
Motor	HP	30
Grit Cyclones		
Number of units	----	2
Maximum inlet flow rate	gpm	430
Maximum inlet pressure	psi	10
Underflow rate	gpm	40

Table 4-10: Summary of Grit Removal System Design Criteria

Parameter	Unit	Design Criteria
Grit Classifier		
Number of units	----	2
Type	----	Screw
Classifier diameter	inch	18
Capacity	gpm	40
Motor	HP	1

[1] *Per Water Environment Federation Manual of Practice (MOP) 8, Section 4.2.1.*

4.2.5.3 Influent Flow Measurement

One 48-inch Parshall flume will be located downstream of grit removal for influent flow measurement. The Parshall flume will be capable of measuring flow up to a maximum discharge of 43.9 mgd.

4.2.5.4 Influent / Return Activated Sludge Distribution

The new preliminary treatment structure will include a distribution box to evenly distribute mixed liquor flow to each of the four aeration basins. Preliminary treated influent will be mixed with RAS in a vertical channel prior to entering a horizontal wet well with vertical mixers for quiescent mixing and blending of mixed liquor. Mixed liquor will be distributed over sharp crested weirs to each aeration basin.

4.2.5.5 Waste Activated Sludge Pumping

The new preliminary treatment facility will also house the WAS pumps and magnetic flow meter. The pumps will be located on the lower elevation of the new structure with the grit pumps. The WAS pumps will draw sludge from the RAS force main. The pumps will be sized to waste sludge over an 8-hour period at 16 mgd and 7,000 mg/L MLSS to minimize the impact to the biological process. The pump motor will be equipped with a VFD to be able to turn down at lower flows or a higher concentration of sludge. Table 4-11 provides a summary of design criteria for the WAS pumps.

Table 4-11: Summary of Waste Activated Sludge Pump Design Criteria

Parameter	Unit	Design Criteria
Number of units	----	2 (1 duty, 1 spare)
Time to waste	hours	8
Type	----	Rotary lobe
Rated capacity	gpm	800
Total dynamic head	feet	45

Table 4-11: Summary of Waste Activated Sludge Pump Design Criteria

Parameter	Unit	Design Criteria
Motor	HP	20
Motor start	----	VFD (turn-down to 500 gpm)

4.2.6 Secondary Treatment

It is recommended that the County convert the two oxidation ditches to a plug flow five-stage configuration and construct two new aeration basins for a total capacity of 16 mgd. Table 4-7 summarizes the process volume and HRT for each zone in the five-stage configuration for both the new aeration basins and the converted existing oxidation ditches. Due to the treatment capacity limitation of the existing oxidation ditches, the new aeration basins must be constructed before the oxidation ditches can be taken out of service for the conversion.

The recommended five-stage process configuration requires specific process equipment in each zone. The anaerobic and anoxic zones will be equipped with vertical shaft mixers. The aerobic zone and re-aeration zone will be equipped with fine bubble membrane diffusers. The post-anoxic zone will be constructed as a swing zone. Vertical mixers and membrane diffusers will be installed in this zone. A NRCY pump will be installed at the end of the first aerobic zone to recycle flow back to the anoxic zone. Tables 4-12 and 4-13 summarize the technical design criteria for the vertical shaft mixers and NRCY pumps, respectively. Each basin will be constructed with a fixed effluent weir.

The existing structural design of the oxidation ditches will need to be checked and verified for the recommended retrofit improvements. New basin divider walls are proposed to be added to accommodate the five-stage process configuration. The new walls will be designed as load bearing walls. Hazen recommends adding a strip footing at the base of the new walls supported on the top slab with the assumption that the existing bottom mat will be able to handle the proposed loads. Hazen also recommends adding columns with strip footers to tie into the proposed elevated walkway. During the final design phase, Hazen will confirm that the remaining existing exterior walls and divider wall will be able to support the concentrated loads from the beams framing into them and the proposed loads transferred from the new walls that will intersect with the existing walls.

Table 4-12: Summary of Vertical Mixer Design Criteria

Parameter	Units	Anaerobic Zones	Anoxic Zone	Post Anoxic Zone
Number of mixers, per basin	----	3	1	1
Number of mixers, total	----	12	4	4
Impeller diameter	inches	90	135	130
Shaft diameter	inches	2	3	3
Rated motor output	HP	1	10	10
Input voltage	V	460	460	460

Table 4-13: Summary of Nitrate Recycle Pump Design Criteria

Parameter	Units	NRCY Pumps in Aeration Basins 3 and 4	NRCY Pumps in Aeration Basins 1 and 2
Number per basin	----	1	1
Total number of pumps	----	2	2
Type	----	Submersible propeller	Submersible propeller
Recycle rate	----	150 – 300%Q	150 – 300%Q
Rated capacity, each	gpm	8,333	8,333
Rated capacity, each	mgd	12.0	12.0
Total dynamic head	feet	2.1	1.5
Motor	HP	16	16
Motor start	----	VFD	VFD

4.2.6.1 Aeration System

The existing Lower Dorchester WWTP aeration system consists of bridge-mounted mechanical surface aerators. The results of the BioWin® modeling demonstrated that the treatment capacity of the oxidation ditches with the existing mechanical aeration system is limited to 3 mgd per basin. The conversion of the existing oxidation ditches to plug flow and a diffused aeration system will increase the treatment capacity to 4 mgd per basin. The new aeration basins will also be equipped with a diffused aeration system.

The new aeration system will consist of fine bubble diffusers with air supplied by multi-stage centrifugal blowers. Table 4-14 summarize the process aeration requirements for 16 mgd design flow. The aeration system design is based on the following assumptions:

- Equivalent load and air input to each basin in service.

- A carbonaceous oxygen requirement of 1.2 pound (lb) oxygen per lb BOD_5.
- A nitrogenous oxygen requirement of 4.57 lb oxygen per lb TKN oxidized to nitrate.
- A denitrification oxygen credit of 2.86 lb oxygen per lb nitrate reduced.
- 1 lb TKN assimilated per 30 lb BOD_5 oxidized.
- An alpha-F value of 0.64 based on Hazen's experience for plants configured as a five-stage BNR process.
- A beta factor of 0.95.
- A diffuser submergence of 15.75 feet was assumed to develop standard oxygen transfer efficiency (SOTE) values and required air flows. SOTE values are based on efficiency data for nine-inch membrane disc diffusers provided by Sanitaire on other Hazen projects at similar depth and diffuser densities.
- A total of 2,500 installed diffusers in the aerobic zone per basin.
- Minimum mixing airflow requirement of 0.12 standard cubic feet per minute per square foot (scfm/SF) average in the aerobic zones.

Table 4-14: Process Aeration Requirements for 16 mgd Annual Average Design Flow

Criteria	Units	Current Minimum Day	16 mgd Annual Average	16 mgd Maximum Month	16 mgd Maximum Week	16 mgd Maximum Day
Basins in service	----	2	4	4	4	4
Influent flow	mgd	5	16	16	16	16
Influent BOD_5	mg/L	100	200	300	400	500
Influent TKN	mg/L	25	40	60	80	100
Actual oxygen required	lb O_2/d	7,200	42,500	63,000	83,600	105,000
Dissolved oxygen	mg/L	2	2	2	1	0.5
AOR / SOR	----	0.50 [1]	0.48	0.48	0.55	0.59
Standard oxygen required	lb O_2/d	14,500	87,700	131,000	151,000	177,000
Airflow / diffuser	scfm	0.5	1.0	1.6	1.8	2.3
Standard oxygen transfer efficiency	%	38.2%	35.3%	33.6%	33.0%	31.4%
Process airflow	scfm	1,400	9,900	15,600	18,200	22,500
Minimum mixing airflow	scfm	2,600	5,300	5,300	5,300	5,300

[1] *Minimum mixing requirements control at minimum flow.*

The diffused aeration system will consist of nine inch ethylene propylene diene methylene (EPDM) membrane disc diffusers. The diffused air system will be tapered to provide optimal air distribution throughout the aerobic zone. Five diffuser grids will be provided in each basin. Three grids will be provided in the primary aerobic zone. One grid will be provided in the post-anoxic/swing zone and one grid in the re-aeration zone.

Two dissolved oxygen control zones will be provided for each aeration basin. The first zone will include aerobic zone grids 1 and 2. The second zone will include aerobic zone grid 3, the post-anoxic/swing zone, and the re-aeration zone. Each dissolved oxygen control zone will include:

- An insert Venturi air flow meter to measure the airflow per zone.
- An aeration control valve downstream of each flow meter to control airflow to each control zone to maintain target dissolved oxygen concentrations while maintaining minimum airflow for mixing.
- A dissolved oxygen probe and analyzer for measuring dissolved oxygen.

The required peak day aeration capacity at the 16 mgd design flow is 22,500 scfm at a discharge pressure of 8.8 pounds per square inch gage (psig). Sufficient blower capacity will be provided to meet this requirement with all units in service. A firm capacity (e.g., largest blower out of service) of 18,200 scfm will be provided for the 16 mgd maximum week condition. Minimum blower turndown will provide the mixing airflow requirement of 2,600 scfm assuming two basins in service.

To ensure sufficient oxygen is provided under actual site conditions, the inlet air requirement (inlet cubic feet per minute, icfm) must be calculated in order to select the appropriate blower equipment. Blowers will be sized to provide for the required mass flow rate at the worst case condition, which is a maximum temperature and coincident humidity and minimum barometric pressure. The resulting required inlet air requirement at 16 mgd max day airflow is 25,000 icfm assuming the worst-case site conditions of 14.7 pounds per square inch atmospheric (psia) pressure, 98°F, 60 percent relative humidity, and 0.3 psig inlet loss.

Five multi-stage centrifugal blowers located in a new blower building are proposed for the expansion. Three blowers will provide 4,000 scfm at 8.8 psig discharge pressure. Two larger blowers will provide 6,200 scfm at 8.8 psig. Total and firm capacity are 24,400 scfm and 18,200 scfm, respectively. Inlet throttling valves will be provided on each blower inlet pipe to control capacity. Table 4-15 summarizes the preliminary blower selections from Gardner Denver.

Table 4-15: Summary of Preliminary Blower Selections

Parameter	Units	Small Blowers	Large Blowers
Number of blowers	----	3	2
Design flow per blower	scfm	4,000	6,200
Total capacity	scfm	24,400	
Firm capacity	scfm	18,200	
Minimum flow per blower	scfm	2,400	3,000
Discharge pressure	psig	8.8	8.8
Rated motor output	HP	250	400
Input voltage	V	460	460
Total installed power	HP	1,550	

4.2.7 Mixed Liquor Distribution

A new mixed liquor distribution box will be constructed to distribute mixed liquor evenly to the secondary clarifiers. The existing distribution box does not have hydraulic capacity for an expansion to 16 mgd. The new distribution box will contain vertical shaft mixers to mix the solids prior to distribution to each clarifier via a fixed sharp-crested weir. Each weir may be isolated via stop plates to remove a clarifier from service or adjust the weirs, if necessary.

4.2.8 Secondary Clarification

The Lower Dorchester WWTP operates six out of eight secondary clarifiers of varying diameters and sidewater depth. Clarifiers 1 and 2 were constructed in the 1980s with the original plant and have a diameter of 65 feet and a sidewater depth of 12 feet. Clarifiers 1 and 2 are currently not in service. Clarifiers 3 and 4, constructed in the mid-1990s with similar dimensions to clarifiers 1 and 2, are only used in extreme wet weather events. Clarifiers 5 through 8 were constructed in the 2007 expansion and have a diameter of 100 feet at a sidewater depth of 16 feet. A mechanism is clarifier 8 was added in 2020.

The future use of clarifiers 1 through 4 was evaluated in the expansion analysis. Clarifiers 1 through 4 are small diameter and have a shallow sidewater depth, which increases the difficulty of clarifier operation. The hydraulic profile of these clarifiers is several feet lower than the newer 100-foot diameter clarifiers and therefore require pumping to tertiary filtration and disinfection. Additionally, the clarifiers are showing signs of age-related corrosion. During workshop discussions with County staff, the decision was made to remove from service and/or demolish clarifiers 1 through 4 and recover the space for future infrastructure.

Table 4-16 provides a summary of the clarifier matrix for the expansion from 8 to 16 mgd. The process analysis includes an evaluation of clarifiers out of service for maintenance. Based on operational experience at the Lower Dorchester WWTP, peak hour flow events are sustained for long periods of time (e.g., over 6 to 8 hours). County staff has requested additional secondary clarification capacity to assist with attenuation of these sustained peaks. The evaluation considers the use of the existing four 100-foot diameter clarifiers 5 through 8 with the addition of two new 100-foot diameter clarifiers. Two new clarifiers will be required for the expansion to 16 mgd to maintain surface and sludge overflow rates to within recommended design criteria.

Table 4-16: Secondary Clarifier Process Analysis for 16 mgd Expansion

No. Clarifiers in Service [1, 2]	Plant Flow, mgd [2]	Diameter, feet	SOR, gpd/SF	SOR peak, gpd/SF	SLR, lb/hr/SF	SLR peak, lb/hr/SF [3]
6	16	100	340	850	0.83	1.5
5	16	100	408	1,020	1.0	1.7
4	16	100	510	1,280	1.2	2.2
5	12	100	306	764	0.7	1.3
4	12	100	382	955	0.9	1.6
3	12	100	510	1,280	1.2	2.2
M&E Design Criteria:			200 - 400	600 - 1,000	0.2 - 1.0	1.5
Hazen and Sawyer Design Criteria:			< 350	< 875	0.2 - 1.0	1.5

[1] Calculations assume use of existing four 100-foot diameter clarifiers and two new 100-foot diameter clarifiers.
[2] Calculations assume the four 65-foot diameter clarifiers are removed from service.
[3] An MLSS concentration of 3,500 mg/L was used to evaluate solids loading as a worst-case scenario.

4.2.9 Return Activated Sludge Pumping

A new RAS pump station will be added to serve new clarifiers 9 and 10. The pump station will pump RAS from the clarifiers to a new influent distribution box designed as a component of the new preliminary treatment facility. A total of three pumps in a two duty and one standby configuration will be designed to pump 50 percent of the peak flow (e.g., 2,312 gpm) with turndown to 50 percent of the average flow (e.g., 926 gpm). Table 4-17 provides a summary of the RAS pumps design criteria for the expansion to 16 mgd.

Table 4-17: Summary of Return Activated Sludge Pump Design Criteria for Clarifiers 9 and 10

Parameter	Unit	Design Criteria
Number of units	----	3 (2 duty, 1 spare)
Type	----	Screw centrifugal, flooded suction
Rated capacity	gpm	2,312
Rated capacity	Mgd	3.3
Total dynamic head	feet	44.5
Motor	HP	40

Table 4-17: Summary of Return Activated Sludge Pump Design Criteria for Clarifiers 9 and 10

Parameter	Unit	Design Criteria
Motor start	----	VFD (turn-down to 926 gpm)
Recycle ratio (at peak)	----	1.25

4.2.10 Tertiary Filtration

The Lower Dorchester WWTP tertiary filtration facility was constructed with five disk filters in the 2007 expansion project. A sixth disk filter was added in 2020. For the expansion to 16 mgd, the evaluation determined that a seventh filter will be required to meet the design criteria of 6 gallon per minute per square foot (gpm/SF) with one filter out of service (e.g., N-1) at 75 percent of the peak hourly flow hydraulic loading rate, which is a requirement of DHEC Regulation R.61-67. The majority of the available disk filter equipment is California Title 22 approved at a hydraulic loading rate 6 gpm/SF. Table 4-18 provides a summary of the filter design criteria for the expansion to 16 mgd.

The existing filters are currently separated on the plant site and one additional filter is required for a WWTP expansion to 16 mgd. The split filter locations will hydraulically overload filters 1 through 4 and hydraulically underload filters 5 and 6 when the new secondary clarifiers are in service. It is recommended that filters 5 and 6 be re-located next to filters 1 through 4 with space for the new seventh filter. A new influent header will replace the existing hydraulically undersized header to feed influent to the filters. Additionally, a new tertiary filter effluent box will be constructed to combine tertiary treated flow prior to disinfection. All of the filters are constructed with a bypass in the event of excessive headloss.

Table 4-18: Summary of Proposed Design Criteria for Disk Filters

Capacity Scenario	Peak Flow, mgd	75% of Peak Flow, mgd	Number of Existing Filters	Additional Filters	75% Peak Hydraulic Loading Rate, gpm/SF [1]
8 mgd	20	15	6	0	3.2
12 mgd	30	23	6	0	4.8
16 mgd	40	30	6	1	5.4

[1] Design criteria for one unit out of service (N-1) at 75% of hydraulic peak at 6 gpm/SF.

4.2.11 Disinfection

The existing disinfection system for the Lower Dorchester WWTP consists of a Wedeco TAK55. The system was installed during the 2007 plant expansion. An evaluation of the existing UV system demonstrated that there are hydraulic and treatment limitations with the existing UV equipment. The channels are not hydraulically sized for an expansion to 16 mgd. The existing system was also not sized to appropriately treat to the Enterococci bacterial standard, 35 most probable number (MPN) per 100 mL as a monthly geometric mean and 104 MPN per 100 mL as a daily maximum. Furthermore, County staff has expressed dissatisfaction with the existing system maintenance costs and difficulty to access wear items, such as lamps, for maintenance.

A new UV facility will be constructed for the plant expansion. The new UV facility will allow the County to consider newer UV technologies that have deeper channels, higher intensity lamps, and ballasts located out of the UV channel. Due to the significantly different UV equipment designs and dimensions and power requirements, it is recommended that a pre-selection evaluated bid be conducted for the selection of the UV equipment. Table 4-19 provides a summary of the design criteria for a new UV facility.

Table 4-19: Summary of UV Disinfection Design Criteria

Parameter	Units	Design Criteria
Average design flow	mgd	16
Peak design flow	mgd	40
Minimum UVT [1]	%	60
Maximum TSS	mg/L	5
Minimum UV dose at peak flow	mJ/cm^2 as MS2 RED	30
Number of channels	----	2
Enterococci Limit	cfu/100 mL	35 monthly geometric mean 104 daily maximum
Sleeve fouling factor	----	0.90 (mechanical wipers) 0.95 (mechanical/chemical wipers)
Redundancy Class III Reliability Requirement	----	50% ADF redundancy (50% of the average design flow can be accommodated by the remaining units with the largest unit out of service)

[1] *Minimum UVT based on Lower Dorchester WWTP data from January 2017 to March 2018.*

Three UV manufacturers were contacted for budgetary proposals for the Lower Dorchester WWTP. All three manufacturers provided equipment proposals: Xylem for the Duron UV system, Trojan Technologies for the UVSigna system and Suez/Ozonia for the Aquaray 3X system. Xylem and Trojan are similar systems with inclined lamp technology. The Suez/Ozonia system has a vertical lamp orientation. All three systems have lamp ballasts located in stainless steel panels for ease of access and protection from flooding. Design data for the three proposed UV systems are summarized in Table 4-20.

A new two-channel UV disinfection structure will be constructed to house UV disinfection equipment. The structure will include a common influent channel and motorized slide gates for isolation at the entrance to each channel. An overhead canopy will be necessary to protect the UV equipment from sunlight and algal growth. Water level in the channels will be controlled with fixed serpentine weirs. The new UV facility will include a building to house the electrical equipment.

Table 4-20: Summary of Proposed UV Manufacturer System Design Data

Item	Units	Xylem Duron	Trojan UVSigna	Suez/Ozonia Aquaray 3X
Number of lamps, total	----	256	132	432
Lamps per channel	----	128	132	216
Number of channels	----	2	2	2
Approximate channel width	feet	6.5	5	5
Approximate minimum channel length	feet	36	30	25
Duty banks per channel	----	4	3	3
Number of modules per bank	----	2	1	2
Number of lamps per module	----	16	22	36
Peak flow power (duty lamps and ballasts)	kW	156.6	127	140.1
Peak flow power (duty lamps and ballasts)	kW / mgd	3.9	3.2	3.5
Total connected system power (all lamps)	kW	178.6	139	175.2
Total connected system power (all lamps)	kW / mgd	4.5	3.5	4.4
Level control	----	Fixed weir	Fixed weir	Fixed weir
Maximum delivered peak flow dose	mJ/cm^2	32	32.1	38.3
Number of modules per bank	----	2	1	2

4.2.12 Non-Potable and Reclaimed Water Facilities

The existing Accu-Tab chlorinator system will be removed and replaced with a bulk hypochlorite feed system. The major elements of the proposed reclaimed water disinfection system include a bulk hypochlorite storage tote, piping and valves, and chemical metering pumps. The new chemical storage tote will be housed in the existing chemical storage and feed room. The building ventilation will be modified as required for the new hypochlorite system. Two peristaltic hose metering pumps, one duty and one spare, will be provided. The pumps will be mounted on a shelf adjacent to the chemical storage tote. The design criteria and system design assumptions are provided in Table 4-21.

New washwater pumps are recommended to replace the existing three vertical turbine washwater pumps that were relocated in the 2007 expansion. The washwater pumps convey disinfected effluent to the existing ground storage tank. The three vertical turbine washwater pumps are aging and need to be replaced. Two new submersible pumps will be installed in the effluent wet well of the new UV disinfection structure. The new pumps will be tied into existing piping.

Table 4-21: Summary of Design Criteria for Reclaimed Water Pumping and Chlorination Facilities

Parameter	Units	Criteria
Washwater Pumps		
Number of pumps	----	2
Pump type	----	Submersible
Motor	HP	15
Motor start	----	Constant speed
Chlorine Dosing System		
Storage tank volume, existing	MG	1.0
Point of application	----	Tank fill line
Product feed method	----	Peristaltic metering pumps
Feed description	----	Bulk hypochlorite
Trade concentration	%	12
Density	lb Cl_2/gal	1.043
Peak design flow	gpd	250,000
Average design flow	gpd	100,000
Minimum design flow	gpd	25,000
Average chlorine demand	mg/L	5.0
Design chlorine residual	mg/L	4.0
Design chlorine dose	mg/L	9.0
Minimum bulk storage at average demand and flow	days	15
Hypochlorite Storage Tote		
Number of tanks	----	1
Material	----	HDPE
Nominal capacity	gal	300

Table 4-21: Summary of Design Criteria for Reclaimed Water Pumping and Chlorination Facilities

Parameter	Units	Criteria
Storage at 0.25 mgd flow and 9.0 mg/L chlorine dose	days	42
Hypochlorite Metering Pumps		
Number of metering pumps	----	2 total (1 duty and 1 standby)
Type of metering pumps	----	Peristaltic hose
Metering pump capacity, each	gph	0.07 to 0.75

4.2.13 Effluent Flow Measurement

One 48-inch Parshall flume will be located downstream of the UV and reclaimed water systems for effluent flow measurement. The Parshall flume will be capable of measuring flow up to a maximum discharge of 43.9 mgd.

4.3 Solids Handling Facility Design Criteria

An evaluation of the Lower Dorchester WWTP's existing solids handling facilities was conducted to determine if additional solids handling infrastructure is needed for the expansion to 16 mgd. The proposed solids production estimates from the BioWin® model indicated increased capacity will be required to process the anticipated solids load. Biosolids are transported to the Oak Ridge Landfill in Dorchester County for final disposal. The following sections discuss the evaluation for each biosolids treatment system component.

4.3.1 Dewatering and Thickening Evaluation

The Lower Dorchester WWTP currently operates one 2,000 pound per hour (lb/hr) capacity DCEN with a second DCEN installed in 2020. Waste sludge is pumped to the DCEN at a concentration between 0.7 and 1.0 percent solids, which prevents the DCEN from meeting its rated solids dewatering capacity of 2,000 lb/hr. At the current solids feed percentage, the dewatering process is hydraulically limited by the pumping capacity of the sludge feed pumps at 200 gpm. Based on this hydraulic limitation, the DCENs are only able to operate at approximately one half of their solids loading capacity, or approximately 1,000 lb/hr with a 1.0 percent solids feed concentration. The existing decant basins, which store WAS prior to dewatering, are aerated continuously to control odors. With aeration continuously on, sludge is not able to settle for an extended period of time for an opportunity to thicken by gravity.

Table 4-22 summarizes the dewatering capacity and centrifuge run times with the existing two DCENs at a WWTP capacity of 16 mgd. At a 0.7 percent solids feed rate, the two DCENs would be required to operate 106 hours per week (e.g., greater than 2.5 shifts per day over seven days) at an annual average sludge production rate for the expansion to 16 mgd. At the 16 mgd solids production rate and a solids feed rate greater than 2.0 percent, the existing DCENs will be operating as solids limited and therefore able to run at maximum capacity. Thickening the feed solids will allow the DCENs to operate one shift five days a week (e.g., 40 hours per week) at an annual average solids production rate and on one shift 7 days per week at a maximum month (e.g., 56 hours per week) solids production rate.

Hazen recommended thickening the feed solids prior to dewatering in lieu of the installation of additional dewatering equipment. The advantages of thickening include the use of the entire capacity of the existing centrifuges, more operation flexibility, reduced sludge storage volume, and not providing additional solids cake conveyance. The thickening option will require additional equipment to operate and maintain. However, the installation of additional dewatering equipment will also require additional equipment and a multi-story building for cake conveyance.

Several thickening options were explored with County staff. Table 4-23 summarizes the various thickening options to include gravity thickening, gravity belt thickening (GBTs), and rotary drum thickening (RDTs). Gravity thickeners perform with the least efficiency of the three thickening options. The gravity thickening option also has the largest footprint, which is an issue on the Lower Dorchester WWTP site. GBTs and RDTs provide the best performance for WAS thickening. The RDT technology was selected based on conversations with County staff.

Table 4-22: Dewatering Capacity and Centrifuge Run Times with Existing Equipment at 16 mgd

		Required Weekly Runtimes with Two DCENs (hr/week)		
% Feed Solids	DCEN Capacity (lb/hr)	Annual Average (22,270 lb/d)	Maximum Month (32,149 lb/d)	Maximum Week (39,760 lb/d)
0.70%	1,500	106	150	185
1.0%	2,000	79	112	139
2.0%	4,000	40	56	70
3.0%	4,000	40	56	70
4.0%	4,000	40	56	70
5.0%	4,000	40	56	70

Table 4-23: Summary of Thickening Options and Design Parameters

Parameter	Gravity Thickener	Gravity Belt Thickener	Rotary Drum Thickener
Peak hydraulic loading rate	200 gpd/SF	250 gpm/meter	400 gpm/unit
Peak solids loading rate	6 lb/d·SF	750 gpm/meter	1,200 lb/hr·unit
Effluent solids capability, % total solids	2.0% – 3.0%	4.0% – 5.0%	4.0% – 5.0%
Preliminary design components	2 x 65 foot diameter units (6,600 feet total)	2 x 3 meter units	2 x 400 gpm units
Approximate footprint	150 feet x 100 feet	60 feet x 60 feet	60 feet x 60 feet
Performance reliability	Low, < 2% for WAS only	High	High

The proposed design criteria for the proposed thickening process is summarized in Table 4-24. The thickening system will include two RDTs with space for a third RDT, sludge feed pumps, thickened sludge pumps, and a polymer feed system. Only duty equipment will be provided for each RDT. The RDTs will withdraw sludge from the aerated sludge storage tanks and discharge thickened sludge back to the sludge storage tanks for recuperative thickening to 2 to 3 percent solids. A liquid sludge transfer station will be provided to offload thickened sludge to truck loading at 4.5 percent solids, if necessary.

Chemical totes are recommended for polymer feed storage. The polymer feed use is anticipated to be approximately 30 gpd at 16 mgd, so a 275 gallon tote will last approximately 10 days. Bulk polymer storage is not recommended, as the turn-over in the tank will exceed the shelf life of the polymer. The County currently uses a blend branch linear cationic polymer for centrifuge dewatering. The specific polymer type for the RDT thickening process will be determined during facility start-up.

Table 4-24: Summary of Design Criteria for Thickening System

Parameter	Units	Design Criteria
Rotary Drum Thickeners		
Number of units	----	2
Solids loading rate (dry), per unit	lb/hr-meter	1,200
Hydraulic capacity	gpm	400
Percent solids, feed	%	0.7 – 1.0%
Percent solids, thickened	%	4.0 – 4.5%
Solids capture efficiency	%	95
Sludge Feed Pumps		
Number of pumps	----	2 (2 duty)
Type	----	Rotary lobe, positive displacement
Capacity	gpm	400
Motor	HP	30
Sludge Transfer Pumps		
Number of pumps	----	2 (2 duty)
Type	----	Rotary lobe, positive displacement
Capacity	gpm	400
Motor	HP	30
Polymer Feed System		
Number of pumps	----	2
Type	----	Positive displacement, progressing cavity
Typical thickening polymer dose	lb active polymer solids per dry ton	8
Polymer use, active solids (at 16 mgd)	lb active polymer solids per hour	6
Percent polymer, typical	% active polymer solids	40
Dose, neat (at 16 mgd)	lb neat per hour	15
Polymer use (neat)	gpd	28.9
Days of storage per tote	day	10
Neat polymer metering capacity	gph	0.05 – 20
Dilution water capacity	gpm	3 – 50

4.3.2 Aerated Sludge Storage

The existing solids holding capacity is inadequate for a plant expansion to 16 mgd. The existing aerobic digesters will provide one day of storage at a 0.7 percent solids concentration at the annual average solids production rate for an expansion to 16 mgd. The existing decant basins will provide three days of storage at 0.7 percent solids for a 16 mgd treatment facility. It is recommended that two new aerated sludge holding tanks be constructed to meet the storage requirement for the WWTP expansion. Two rectangular common wall tanks are recommended to enable one tank to be taken out of service for maintenance. Between five and eight days of storage in the proposed sludge holding tanks will be provided at thickened sludge concentrations between 2.0 and 3.0 percent solids. The total storage in the decant basin and thickened sludge in the proposed sludge holding tanks will be between 8 and 11 days.

The aerated sludge storage tanks will be equipped with fine bubble membrane diffusers with air supplied by new positive displacement blowers. Fine bubble diffusers provide more efficient aeration than coarse bubble diffusers, which means smaller blowers. Per manufacturer recommendations, fine bubble diffusers are not reliable for mixing above 3 percent solids as the viscosity is too high for efficient mixing. Table 4-25 provides a summary of the design criteria for the proposed sludge holding tanks and diffused aeration system.

Table 4-25: Summary of Design Criteria for Aerated Solids Holding Tanks

Parameter	Units	Design Criteria
Sludge Holding Tank		
Type	----	Rectangular, common wall
Length	feet	100
Width	feet	50
Sidewater depth	feet	20
Freeboard	feet	2
Total depth	feet	22
Days of storage, 2.5% solids	days	5
Days of storage, 3% solids	days	8
Volume	gal	748,000
Mixing Requirements		
Mixing type	----	Fine bubble membrane diffusers
Mixing energy	scfm/1,000 CF	30
Maximum sludge concentration	%	3
Number of diffusers, per tank	----	825
Number of diffusers, total	----	1,650

Table 4-25: Summary of Design Criteria for Aerated Solids Holding Tanks

Parameter	Units	Design Criteria
Blowers		
Type	----	Positive displacement
Number	----	2 (2 duty, 0 spare)
Blower capacity, each	scfm	1,650
Blower capacity, total	scfm	3,300
Motor, each	HP	200
Motor start	----	VFD

4.4 Electrical Considerations

4.4.1 Existing Electrical System

The Lower Dorchester WWTP is served by two incoming utility services that supply the on-site South Carolina Electric & Gas (SCE&G) utility transformers. These two services supply a 1,500kVA pad mounted transformer and a 300kVA pole mounted transformer. The 1,500 kilo-volt-ampere (kVA) pad mounted transformer is located southwest of the Return Sludge Pump Station #1. The 300kVA pole mounted transformers are located southwest of the original facility maintenance building. Both transformers step down the voltage from 12.47kVAC to 480VAC. The 1500kVA pad mounted transformer provides power to the WWTP's main switchboard MS, located in an existing process building that was converted into an electrical room just south of the Return Sludge Pump Station #1. The 300kVA pole mounted transformers provide power to the original facility switchboard MSP located in the original front office building.

The County owns and operates three emergency generators. The generators are a 1,100kW, 480Vac, 3-phase standby diesel generator; a 300 kilowatt (kW), 480VAC, 3-phase standby diesel generator; and a 150kW, 480V, 3-phase standby diesel generator. The 1,100kW generator is electrically connected to a 2,500amp automatic transfer switch (ATS) that feeds power to switchboard MS. The 1,100kW generator is located between the 1,500kVA pad mounted transformer and the electrical room south of Return Sludge Pump #1. The 300kW generator is located south of the original front office and is electrically connected to a 600amp ATS that feeds switchboard MSP. The 150kW generator is located just west of the reuse pump station between the reuse pump station and reuse above ground storage tank. When utility power is lost, all three auto transfer switches perform a transfer to standby sequence which signals all standby generators to start.

4.4.2 Proposed Modifications to the Existing Electrical System

The proposed expansion project from 8 to 16 mgd has a significant impact on the existing electrical distribution system. A more robust and reliable electrical distribution system for the WWTP expansion is recommended. The demolition of the existing infrastructure will be staged over four different demolition phases (Section 5). MOPO will apply to both the process mechanical and electrical equipment.

An additional concern is the ability of the current electrical distribution system to support the existing loads. Based on the current operating loads, the existing 1,100kW standby generator will not be able to support the 8 mgd WWTP. The estimated operating load for an 8 mgd capacity is 2,622kW. Most electric utility companies allow their utility transformers to be pushed beyond their nameplate rating for short periods of time; however, these transformer limits could possibly be met or exceeded if the WWTP is fully loaded at 8 mgd, which may lead to transformer failure.

The proposed electrical distribution will consist of two new utility transformers, two standby generators, two independent main switchboards that will provide a dual feed to the motor control centers (MCCs), switchboards, and non-automatic and automatic transfer switches (NATS/ATS). The new switchboards and MCCs located at the process areas will have a split bus with a tie breaker that is key interlocked with the two main breakers. The existing MCCs and panelboards will be powered through a NATS/ATS and managed through the plant supervisory control and data acquisition (SCADA) system. One-half of the electrical distribution will be temporarily installed for Phase 1 and 2. During Phase 3, the second half of the electrical equipment while be installed at the new blower building. Once the Phase 3 electrical distribution is running, the temporary electrical distribution in Phase 1 will be relocated to the blower building. The electrical distribution will serve the WWTP at the expanded capacity of 16 mgd.

There are three electrical modes of operation for the WWTP. Only one mode of operation will limit the capacity to average daily flow. The operating modes are as follows:

- Operating Mode 1 – Two utility transformers online, no generators running, both main switchboards feeding local MCCs, Switchboards, and NATS/ATS.
- Operating Mode 2 – No utility transformers online, both generators running, both main switchboards feeding local MCCs, Switchboards, and NATS/ATS.
- Operating Mode 3 (limited operation) – No utility transformers online, single generator running, single main switchboard feeding local MCCs, Switchboards, and NATS/ATS. Plant staff manages local MCCs, switchboard tie breakers, and NATS/ATS while performing load shedding to maintain average daily flow.

REV	ISSUED FOR	DATE	BY

PROJECT ENGINEER:	M. SADLER
DESIGNED BY:	M. SADLER
DRAWN BY:	S. GRUBER
CHECKED BY:	D. NAILOR

IF THIS BAR DOES NOT MEASURE 1" THEN DRAWING IS NOT TO FULL SCALE
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RALEIGH, NORTH CAROLINA 27607

DORCHESTER COUNTY
SOUTH CAROLINA

LOWER DORCHESTER WASTEWATER
TREATMENT PLANT
EXPANSION

PROCESS FLOW SCHEMATIC

DATE:	2019
HAZEN NO.:	32301-011
CONTRACT NO.:	1
DRAWING NUMBER:	Figure 4-1

5. Project Recommendations and Cost Opinion

A combination of existing and new infrastructure is recommended for the Lower Dorchester WWTP expansion project. Per Section 4, several of the existing unit processes are undersized for a WWTP expansion to 16 mgd. The following sections provide a summary of the design criteria, a discussion of maintaining operations during construction, and an opinion of probable construction cost.

5.1 Summary of Design Criteria for Proposed Expansion

Table 5-1 provides a summary of the recommended infrastructure necessary for a WWTP expansion 16 mgd. The WWTP expansion requires that flow distribution (e.g., influent, RAS, MLSS, and secondary effluent) be carefully considered due to the stringent effluent permit limits. Figures 5-1 through 5-3 provide an existing site plan, a demolition plan, and a proposed site plan for the new infrastructure, respectively.

Table 5-1: Summary of Existing and New Infrastructure Required for a 16 mgd Lower Dorchester WWTP Expansion

Unit Process	Unit Process Type	Infrastructure Needed for 16 mgd WWTP Capacity
Influent pumping	On-site influent pump station 2	• New 30-inch force main to replace existing parallel 16-inch force mains • Two new influent jockey pumps to capture lower range of WWTP flow
Preliminary treatment facility	Screens and compactors	• New structure • Two mechanically cleaned screens • One manually cleaned screen
	Grit removal	• Two grit units, 12 stacked trays per unit
	Grit cyclones	• Two units
	Grit classifiers	• Two units
Influent flow measurement	Parshall flume	• One flume at 48-inch throat width • Integral with new preliminary treatment facility
Influent / RAS distribution	Splitter box	• New distribution box • Four distribution weirs • Integral with new preliminary treatment facility
Waste activated sludge pumping	WAS pump station	• Two pumps and magnetic flow meter • Integral with new preliminary treatment facility
Secondary treatment	Aeration basins	• Two new aeration basins in a plug flow five-stage configuration • Retrofit of existing oxidation ditches 3 and 4 to a plug flow five-stage configuration
	NRCY Pumps	• One pump per basin, four total
	Vertical shaft mixers	• Five mixers per basin, 20 total

Table 5-1: Summary of Existing and New Infrastructure Required for a 16 mgd Lower Dorchester WWTP Expansion

Unit Process	Unit Process Type	Infrastructure Needed for 16 mgd WWTP Capacity
Aeration System	Multi-stage centrifugal blowers	• New blower building with electrical room housing main WWTP switchgear • Three 4,000 scfm blowers • Two 6,200 scfm blowers • 2,500 diffusers per basin, 10,000 diffusers total
MLSS distribution	Splitter box	• New distribution box • Six distribution weirs
Secondary clarification and RAS pumping	Secondary clarifiers	• Four existing clarifiers and two new clarifiers • Six total clarifiers
	RAS pump station	• New RAS pump station • Three RAS pumps
Tertiary filtration	Disk filters	• Six existing filters and one new filter • Seven filters total • Filter relocation to common point • New tertiary effluent box
Disinfection	UV disinfection	• New UV structure and electrical room • Two channels
Effluent flow measurement	Parshall flume	• One flume at 48-inch throat width • Integral with new UV structure
Reclaimed water system	Pumping	• Two new reclaimed water transfer pumps • No modifications to existing reclaimed water pump station
	Storage	• No modifications to existing ground storage tank
Solids Handling	Decant basins	• Three days of storage at 0.7% solids • No modifications to existing basins or aeration system
	Aerated sludge holding	• Two new aerated rectangular sludge storage tanks • 5 to 8 storage days at 2% to 3% solids • Two new positive displacement blowers
	Thickening	• New thickening building with electrical room • Two new RDTs, pumps and appurtenances with space for a third RDT
	Dewatering	• No modifications to existing dewatering building with two DCENs

5.2 Maintenance of Plant Operations

The Lower Dorchester WWTP expansion project must be constructed in phases concurrent with the phased demolition of existing infrastructure. Specific infrastructure must remain in service during construction for the facility to remain in compliance with NPDES effluent permit limits. MOPO will be a significant component of the construction phase for the expansion project. For example, the new aeration basins must be in service prior to the oxidation ditches being removed from service for the retrofit to a five-stage process configuration. Figures 5-1, 5-2, and 5-3 provide the existing site plan, demolition plan, and proposed site plan, respectively.

Table 5-2 provides the construction sequence of MOPO in four phases to include demolition. Removing clarifiers 3 and 4 from service in the first phase of construction simplifies MOPO and allows for more flexibility in site planning. The new blower building and main electrical room will be constructed in Phase 3. This MOPO option allows the blower building and electrical room to serve as a main electrical hub for the entire WWTP, which saves approximately $1.5 million for a second electrical service.

Table 5-2: Maintenance of Plant Operations, Demolition, and Construction Phasing

Phase	Demolition	Relocation	Construction
1	• **1A.** Demo existing aeration basins 1 and 2 and associated piping ○ Unknown power feed location for aerator #1 and #2, mixers, and NRCY pumps • **1A.** Demo secondary clarifiers 1 and 2 and associated piping and electrical equipment. • **1A.** Demo aerobic digester 1 and associated piping ○ Demo digester 1 motor controller, existing well pump motor controllers and existing chemical feed room control panel • **1A.** Demo RAS pump station 1 and associated piping. ○ Demo RAS Pump Station 1 electrical distribution – Panel PP3 and PPE • **1A.** Demo secondary clarifiers 3 and 4 with associated piping and RAS pump station 2 with associated piping. ○ Demo RAS pump station 2 electrical distribution – Panel PP4, PPF and existing pump control panel • **1A.** Demo chlorine contact chamber and intermediate pump station. ○ Demo panelboard PPC, 600 amp ATS, 600 amp disconnect switch, and 150 kW Genset	• **1A.** Relocate RAS force main and scum lines from secondary clarifiers 5, 6, 7, and 8 to aeration basins 3 and 4. • **1B.** Relocate drain line from manhole 11 to manhole 9. • **1B.** Relocate reclaimed water high service pump motor controllers from Chlorination Panel PPC to UV-SWBD-C • **1B.** Relocate / extend thickened sludge force main to new aerated sludge storage. • **1D.** Relocate influent pump station VFDs and control panels to new influent pump station electrical building. ○ Addition of power panelboard to feed relocated influent pumps and new ancillary loads for pump station (HVAC, etc.)	• **1B.** Construct new preliminary treatment facility and temporary tie-in of influent distribution lines to oxidation ditches 3 and 4. ○ Install PTF-SWBD-B, SHB-SWBD-D and ancillary electrical in new PTF electrical room. • **1B.** Construct temporary power location, install outdoor rated genset 1 and MSWBD-A, and utility pad mounted transformer for preliminary treatment facility, solids handling building, RAS Pump Station 5, UV disinfection structure and influent pump station. • **1B.** Construct aerated sludge storage and sludge feed tie-in to existing DCEN sludge feed pumps. • **1B.** Construct secondary clarifiers 9 and 10. • **1B.** Construct secondary effluent line from clarifiers 9 and 10. • **1B.** Construct RAS pump station 5 and piping. ○ Install RAS-SWBD-D, motor controllers, and ancillary electrical equipment at RAS pump station #5 for secondary clarifiers 9 and 10 and RAS pump station 5. • **1B.** Construct UV disinfection structure with new washwater pumps. ○ Install new UV-SWBD-C, new washwater pump motor controllers in the new UV disinfection electrical room, re-locate reclaim HSP RVSS into the new UV disinfection electrical room and install new ancillary electrical equipment. • **1C.** Construct MLSS distribution box and piping to clarifiers 9 and 10. • **1C.** Install new RAS line tie-in for RAS pump stations 3 and 4.

Table 5-2: Maintenance of Plant Operations, Demolition, and Construction Phasing

Phase	Demolition	Relocation	Construction

			• **1C.** Install new 30-inch influent force main from on-site influent pump station 1
			• **1C.** Install new influent pump station jockey pumps and motor control panel.
			  ○ Install duct bank from SHB-SWBD-D VIA IJKY-NATS
			• **1D.** Construct new influent pump station electrical building
			  ○ Install duct bank from MSWBD-A/B in Blower Building to IJKY-NATS for influent and jockey pump MCP.

2	• **2A.** Demo aerobic digester 2 and associated piping
	  ○ Demo digester 2 motor controller, existing well pump motor controllers and existing chemical feed room control panel
	• **2B.** Demo existing preliminary treatment facility.
	  ○ Demo grit removal equipment control panel, influent screen control panel, and ancillary electrical equipment powered from existing Switchboard MS.
	• **2E.** Demo existing UV structure, effluent flume, and effluent box.
	  ○ Demo panel PPD and ancillary electrical equipment powered from Panel PPA in RAS pump station 3
		• **2A.** Relocate 36-inch influent force main to new preliminary treatment facility.
		• **2A.** Relocate filters 5 and 6 next to filters 1 through 4.
		  ○ Coordinate re-feeding existing filter 5 and 6 motor controllers from RAS pump station 5 electrical area installed in Phase 1 construction.
			• **2A.** Construct new tertiary effluent box and filter effluent header piping.
			• **2A.** Install filter 7.
			  ○ Coordinate feeding of filter 7 motor controllers from Phase 1 RAS pump station 5 electrical area installed in Phase 1 construction.
			• **2A.** Tie-in filters 1 through 7 to tertiary effluent box.
			• **2B.** Bypass pump from tertiary effluent box to UV disinfection structure.
			• **2B.** Bypass pump effluent to downstream manhole for outfall tie in.
			• **2B.** Raise effluent outfall manholes
			• **2B.** Construct new aeration basins 1 and 2.
			• **2C.** Tie-in new UV structure with existing effluent outfall.
			• **2C.** Construct new secondary effluent line to filters.
			• **2C.** Construct 60-inch effluent line between UV structure and tertiary effluent box.
			• **2D.** Tie-in clarifiers 5 through 10 to secondary effluent line.

Table 5-2: Maintenance of Plant Operations, Demolition, and Construction Phasing

Phase	Demolition	Relocation	Construction

3	• **3B.** Demo existing standby generators and transformers, building housing existing Switchboard MS.
	  ○ Coordinate demo once blower building, new standby generator(s) and utility transformer, and new duct banks have been installed to serve existing remaining processes.
	• **3C.** Demo existing lab building.
	  ○ Demo aerators 1, 2,3, and 4 existing MCCB, SWBD-B and ancillary electrical once sludge holding basin aerator blowers 1, 2, and 3 (from 2018 Improvements Project) moved to new blower building.
		• **3B.** Relocate electrical gear from lab building to new blower building electrical room.	• **3A.** Construct blower building with electrical room serving entire WWTP, generator, and transformer.
			  ○ Install new Genset No. 2, utility transformer, MSWBD-B, and relocate existing Genset No. 1, MSWBD-A, utility transformer and ancillary electrical equipment to permanent location, and install duct banks to existing and propose process structures and building
			• **3A.** Construct MLSS line from aeration basins 1 and 2 to MLSS distribution box.
			• **3B.** Tie in clarifiers 5 through 8 to new MLSS distribution box.
			• **3C.** Place aeration basins 1 and 2 in service.

4	• **4A.** Demo aeration basin 3 and 4 (e.g., oxidation ditch) concrete per the design drawings. • **4A.** Demo existing hexagon MLSS distribution box.		• **4B.** Retrofit of aeration basins 3 and 4 to five-stage process configuration. • **4C.** Tie in aeration basins 3 and 4 to MLSS distribution box. • **4C.** Construct thickening building. ○ Install THKN-MCC-D and ancillary electrical equipment

5.3 Opinion of Probable Construction Cost

The opinion of probable construction cost for the expansion of the Lower Dorchester WWTP to 16 mgd was prepared in accordance with the guidelines of the Association for the Advancement of Cost Engineering (AACE) International for a Class 3 level of estimation. A Class 3 estimate is prepared based on information developed during a preliminary design. The expected accuracy range for a Class 3 level of estimation is +30 percent to –20 percent.

The opinion of probable construction cost is summarized in Table 5-3 and expressed in 2019 dollars. The cost opinion is based on the facility infrastructure recommendations provided in Section 6 for the liquid and solids infrastructure improvements. Construction costs include a 30 percent contingency, 3 percent bonds and insurance, 7 percent County tax on materials, and 20 percent contractor overhead and profit. The cost opinion also includes 15 percent for general conditions to include mobilization, contract administration, trailer, field supervisor, shop drawings, and start-up and training. Labor was escalated to the mid-point of construction at 3.5 percent over a 36 month construction duration. Materials and equipment were escalated to the mid-point of construction at 5 percent. Construction costs were estimated using quotes from equipment vendors and quantity take-offs for concrete, excavation, stone, metal appurtenances, and piping. For smaller ancillary equipment, costs were estimated from similarly sized Hazen and Sawyer projects.

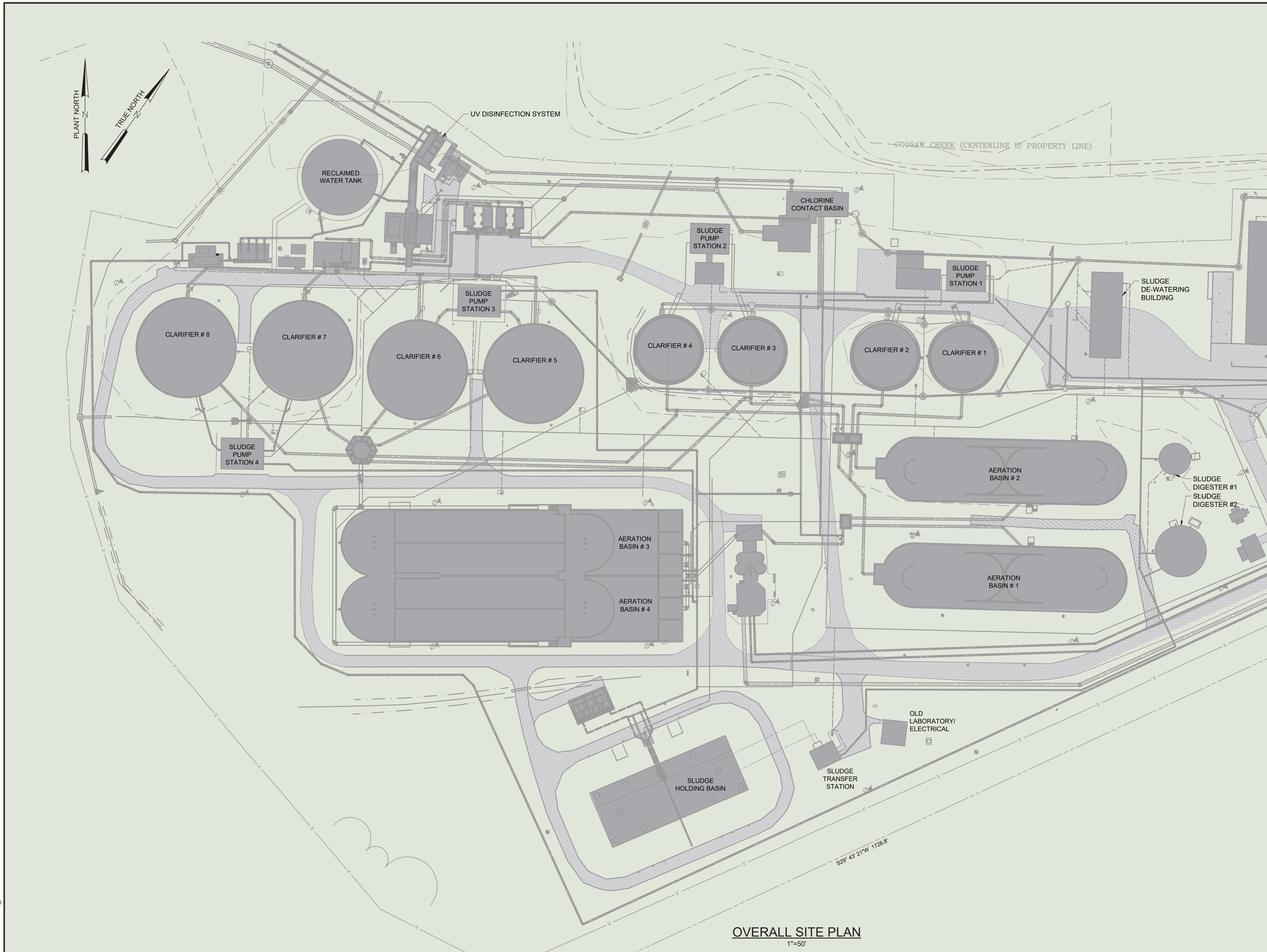
Table 5-3: Opinion of Probable Construction and Project Costs for Lower Dorchester WWTP Expansion to 16 mgd

Project Component [1, 2, 3, 4, 5, 6]	Opinion of Capital Construction Cost for 8 to 16 mgd Expansion of Lower Dorchester WWTP
Demolition	$759,000
Site work	$4,690,000
Yard piping	$8,365,000
Preliminary treatment facility, influent/RAS distribution, WAS pumping	$7,544,000
New aeration basins 1 and 2	$13,245,000
Retrofit of aeration basins 3 and 4	$9,343,000
Mixed liquor suspended solids distribution box	$1,578,000
Secondary clarifiers	$4,595,000
Return activated sludge pump station 5	$1,424,000
Blower building	$4,987,000
Tertiary disk filter and tertiary effluent box	$2,021,000
UV disinfection and building	$5,209,000

Table 5-3: Opinion of Probable Construction and Project Costs for Lower Dorchester WWTP Expansion to 16 mgd

Project Component [1, 2, 3, 4, 5, 6]	Opinion of Capital Construction Cost for 8 to 16 mgd Expansion of Lower Dorchester WWTP
Thickening building	$5,770,000
Aerated sludge holding and blowers	$2,942,000
Electrical work and generator	$6,684,000
General conditions	$11,872,000
Total opinion of probable construction cost	**$91,000,000**
Construction cost opinion range at Class 3 AACE level:	
Low (-20%)	$72,800,000
High (30%)	$118,300,000

[1] Cost opinion includes 3% for bonds and insurance.
[2] Cost opinion includes 20% contractor overhead and profit and 7% County taxes on materials.
[3] Site assumes that shallow foundations will be adequate in lieu of auger cast piles. A geotechnical evaluation is required to confirm this assumption.
[4] Cost opinion includes 30% contingency.
[5] General conditions assumes 15% for mobilization, contract administration, field staff and trailer, shop drawings, and start-up and training.
[6] Labor and materials / equipment were escalated to the mid-point of construction at 3.5% and 5%, respectively.

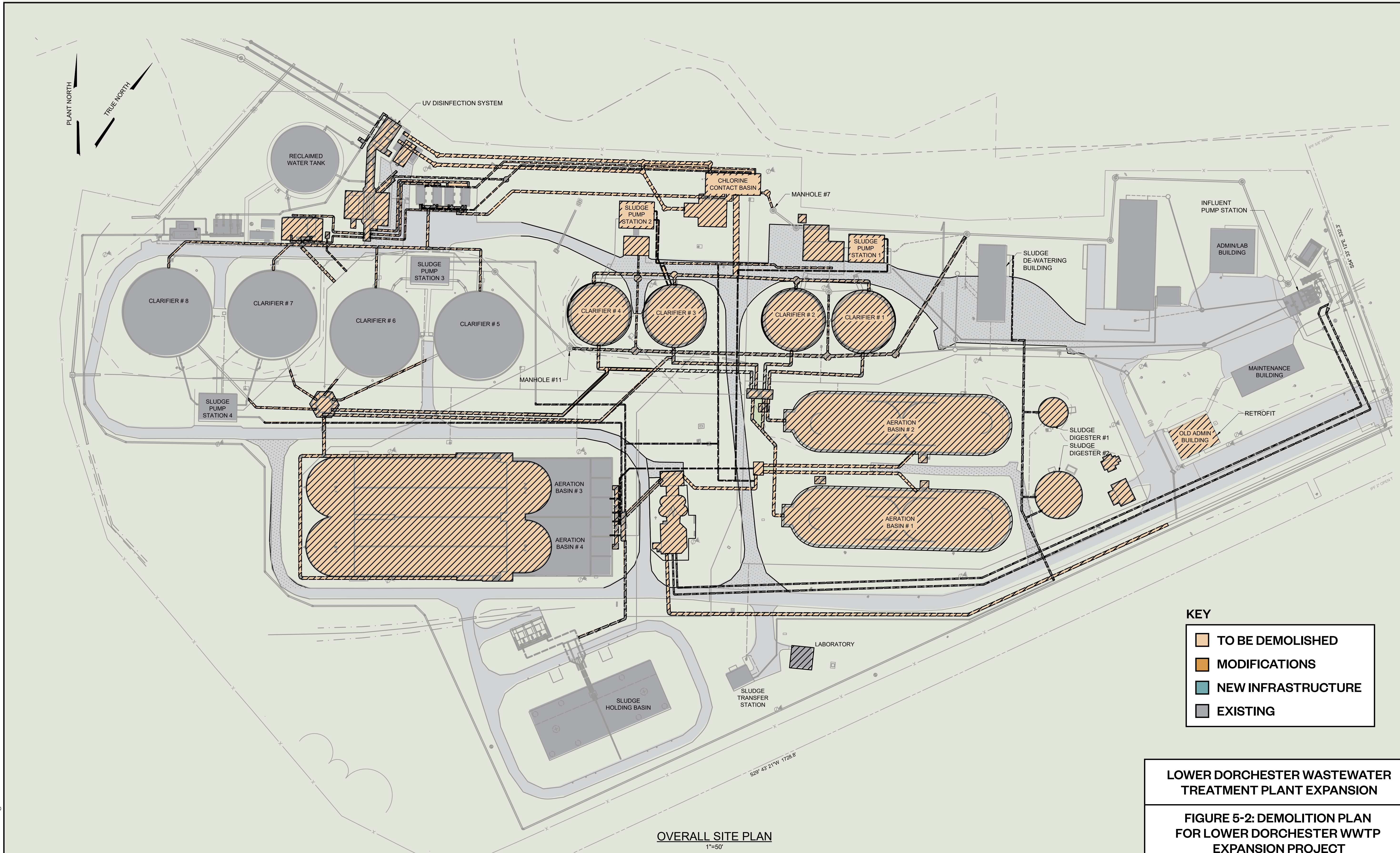


KEY

- TO BE DEMOLISHED
- MODIFICATIONS
- NEW INFRASTRUCTURE
- EXISTING

LOWER DORCHESTER WASTEWATER TREATMENT PLANT EXPANSION

FIGURE 5-1: EXISTING SITE PLAN FOR LOWER DORCHESTER WWTP



KEY

- TO BE DEMOLISHED
- MODIFICATIONS
- NEW INFRASTRUCTURE
- EXISTING

LOWER DORCHESTER WASTEWATER TREATMENT PLANT EXPANSION

FIGURE 5-2: DEMOLITION PLAN FOR LOWER DORCHESTER WWTP EXPANSION PROJECT

32301-011-001-Figure 6-2.ai

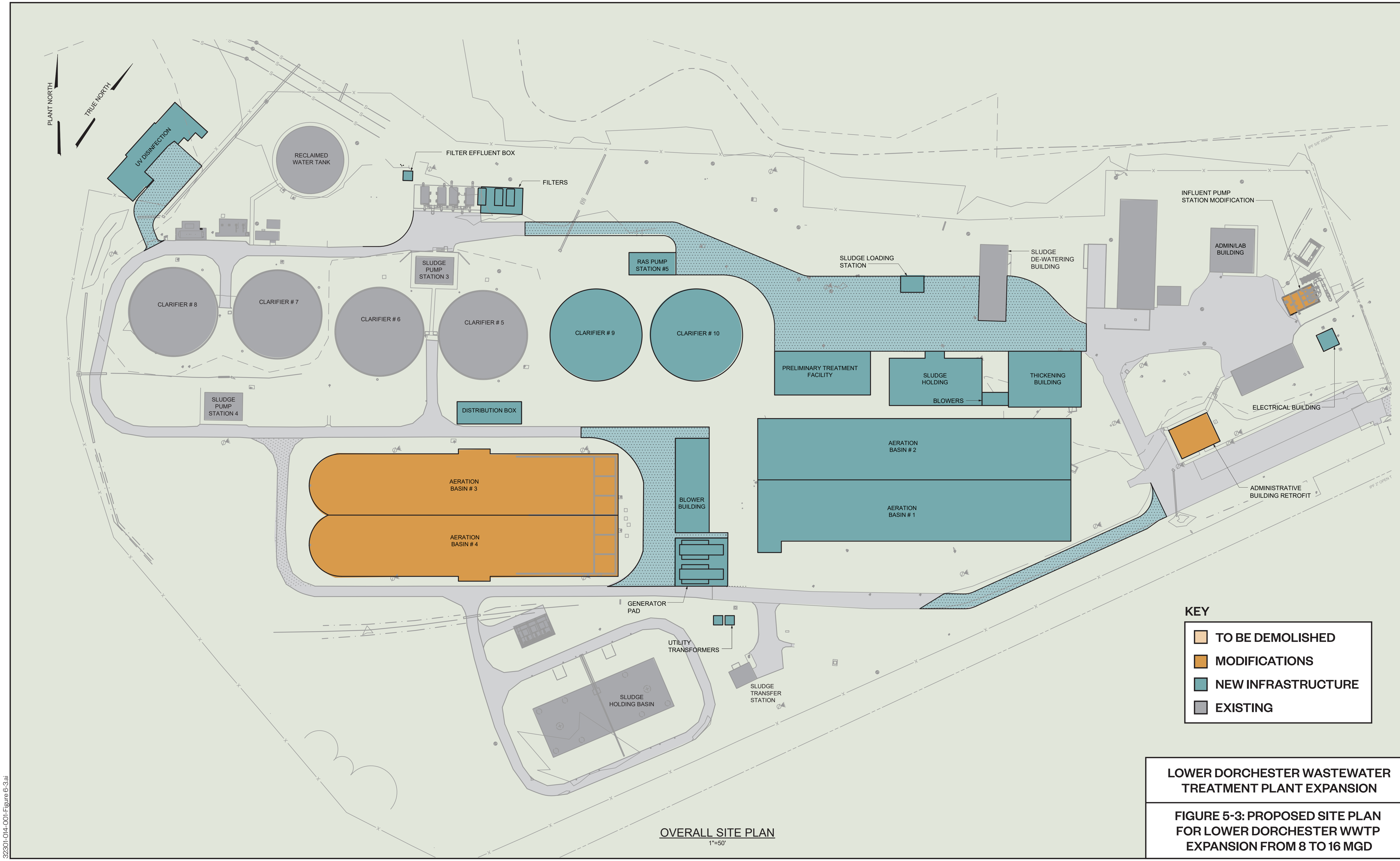


FIGURE 5-3: PROPOSED SITE PLAN FOR LOWER DORCHESTER WWTP EXPANSION FROM 8 TO 16 MGD

32301-014-001-Figure 6-3.ai

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W.K. Dickson and Co., 2019. Preliminary Engineering Report for Central Dorchester WRF. Prepared for Dorchester County Water and Sewer Department.

Appendix A: Preliminary Engineering Report for Central Dorchester Water Reclamation Facility (W.K. Dickson, 2019)

PRELIMINARY ENGINEERING REPORT

for

Central Dorchester WRF

December 2019

Prepared for:

Dorchester County
Water and Sewer Department
235 Deming Way
Summerville, SC 29483

Dorchester
COUNTY

SOUTH CAROLINA
W. K. DICKSON & COMPANY, INC. No. C00177
CERTIFICATE OF AUTHORIZATION

SOUTH CAROLINA
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A. Comprehensive Description of Proposed Project

Project Description

Dorchester County is planning the design and construction of a new wastewater treatment facility in a location central to its wastewater service area. The Central Dorchester Water Reclamation Facility (WRF) will provide additional capacity that will be needed as the existing Lower Dorchester Wastewater Treatment Plant (WWTP) is built out and the central areas of the County continue to grow. A preliminary engineering report for the proposed Lower Dorchester WWTP expansion from 8 to 16 million gallons per day is being prepared as a separate document. The preliminary engineering report for the Central Dorchester WRF has been prepared in a format that addresses the SCDHEC and Berkeley-Charleston-Dorchester Council of Governments (BCDCOG) 208 Major Plan Amendment requirements.

The proposed location of the water reclamation facility (WRF) is at the Pine Hill Business Campus along Highway 17A, west of its intersection with Highway 61 (approximately 10 miles west of Summerville, SC, and 8 miles south of Ridgeville, SC) as shown in Figure 1. The proposed facility will be designed for phased construction of 4.0 and 8.0 million gallons per day (MGD) treatment capacities.

Dorchester County requested a wasteload allocation from SCDHEC in February 2018 for three alternative discharge locations. SCDHEC provided speculative wasteload allocations in June 2018. Based upon SCHEC's response, it was determined that an Ashley River discharge (near the Highway 17A bridge) was the most feasible discharge location. The speculative limits for this discharge location are provided in Table 1, and include stringent ammonia limits and pound per pound allocation of ultimate oxygen demand (UOD) with the Lower Dorchester WWTP.

Table 1 Design Effluent Characteristics (4 MGD)

Parameter	Monthly Average	Weekly Average	Daily Maximum
$cBOD_5$ (mg/L)	5.0	7.5	NA
TSS (mg/L)	30	45	NA
NH_3-N (mg/L)	0.5	0.75	NA
Total Residual Chlorine (mg/L)	0.018	NA	0.032
D.O. (mg/L)	6.0 minimum at all times		
E.Coli (MPN/100mL)	126	NA	349
pH (S.U.)	6.0 – 8.5		
Total Cadmium (mg/L)	0.00060	NA	0.0033
Total Copper (mg/L)	0.015	NA	0.019

Total Lead (mg/L)	0.0054	NA	0.14
Total Zinc (mg/L)	0.24	NA	0.24
Total Mercury (ng/L)	51	NA	74
Chronic WET @ CTC = 100%	25%	NA	40%
UOD (lbs/day)[1]	443 - 710		

1 Assumes F-ratio of 2.2 to 3.8 based on measured values for existing Lower Dorchester WWTP and Summerville CPW WWTP discharging to the Ashley River.

The WRF process train will consist of influent screening, grit removal, biological nutrient removal (BNR), clarifier separation, tertiary filtration and disinfection. Biological nutrient removal will be achieved using a 5-stage Bardenpho process. Effluent will be pumped through a force main and discharged into the Ashley River. Solids will be thickened in a sludge holding tank and dewatered by a screw press, followed by landfill disposal.

Following an extensive, system-wide alternatives evaluation, it is proposed that a WRF be designed and constructed with an initial treatment capacity of 4 MGD (maximum month) with accommodations for efficient expansion to 8 MGD (maximum month). The proposed 4 MGD WRF will include the following new facilities as described below and shown in Figures 2 and 3 and Table 2:

- Influent force main to convey flow from two existing Dorchester County pump stations (Highway 27 Pump Station and Pump Station 67) to the headworks
- Magnesium hydroxide pH adjustment system
- Preliminary treatment with influent screening, grit removal and flow metering
- Secondary treatment by 5-Stage Bardenpho BNR
- Secondary clarification
- Tertiary filtration
- Ultraviolet (UV) disinfection system
- Effluent pump station and force main to a new post aeration system and diffuser in the Ashley River
- Administration and operations building (controls, operations, conference room)
- Electrical & equipment buildings as required by the final design
- Sludge holding tank
- Sludge dewatering building with screw press and space for additional press
- Standby electrical generator

Preliminary Treatment (Screening and Grit Removal)

Preliminary treatment will include a Headworks structure with screening and grit removal. The structure will be designed to accommodate future plant peak flows of 20 MGD (2.5 peaking factor; 8 MGD maximum month average day). Equipment will be installed to initially treat peak flows of 10 MGD (2.5 peaking factor; 4 MGD maximum month average day).

Screening will consist of two mechanically cleaned bar screens with 6 mm openings located upstream of grit removal. Each screen will discharge screenings to a conveyor and compactor for washing and dewatering prior to discharging to a dumpster at grade for collection and off-site disposal. The structure will be arranged to distribute screened influent to the grit removal system.

Grit removal will consist of a stacked tray system including one 9 ft diameter unit with grit pumping, cyclone and classifier. Classified grit will be discharged to the screenings/grit container (common dumpster) for collection and off-site disposal. A second stacked tray module will be added during a future WRF expansion.

Secondary Treatment

Secondary treatment will be achieved using a 5-stage Bardenpho BNR process with clarification. Two bioreactors (common wall) and two secondary clarifiers will be designed to treat peak flows of 10 MGD (2.5 peaking factor; 4 MGD maximum month average day) and allow operational flexibility to take one basin off-line. Accommodations will be made for expansion with the addition of two bioreactors and clarifiers in the future.

Mixing for biological treatment in the anaerobic and anoxic zones will be achieved using vertical shaft mixers. Mixing and aeration in the aerobic zones will be achieved using fine bubble diffusers with air supplied by high-speed, single-stage, turbo blowers. One submersible propeller nitrate recycle (NRCY) pump with variable frequency drives (VFDs) will be installed per bioreactor.

Distribution of mixed liquor from the bioreactors to the secondary clarifiers will be achieved using a distribution (splitter) box with weirs and stop plates for clarifier isolation. Each secondary clarifier will be circular, center-feed, 100 ft diameter with 16 ft sidewater depth. A return activated sludge (RAS) / waste activated sludge (WAS) pump station with duty and spare pumps with VFDs will be designed with space to accommodate future expansion.

Tertiary Filtration

Cloth media disk filters will provide tertiary filtration of the secondary effluent. Two basins will be installed with twelve (12) disks in each basin to treat peak flows of 10 MGD (2.5 peaking factor; 4 MGD maximum month average day) and allow operational flexibility to take one basin off-line. Accommodations will be made for expansion with the addition of two tertiary filtration basins in the future.

Disinfection

Disinfection will be achieved with an ultraviolet (UV) system consisting of two channels with three (3) banks per channel and a minimum dosage of 35 mJ/cm^2 to treat peak flows of 10 MGD (2.5 peaking factor; 4 MGD maximum month average day). The structure will be arranged to accept additional UV banks and fixed weirs will have capacity for future peak flows up to 20 MGD. An overhead canopy will protect the UV equipment from sunlight and algal growth. The UV structure will share a common wall with the effluent pump station, which will initially consist of three 5 MGD vertical turbine pumps with variable frequency drives (two duty and one standby) and space for two additional pumps. Electrical equipment for the UV system and effluent pump station will be located in a nearby building.

Post Aeration

A 24-inch force main will convey effluent from the WRF to the post aeration system located near the Ashley River discharge. A stepped cascade aeration basin will be installed to raise the dissolved oxygen to a minimum of 6 mg/L prior to discharge to a diffuser in the Ashley River.

Solids Handling

Solids handling will consist of waste activated sludge holding and dewatering. One circular sludge holding tank (500,000 gallons) with a fine bubble aeration system (membrane diffusers and positive displacement blowers) will be installed to store, aerate and thicken sludge prior to dewatering. Waste sludge will be pumped from the RAS/Was pump station to the tank at a solids concentration between 0.7 and 1.0 percent. For a 4 MGD WRF, average day sludge production is estimated at 6,400 lbs/day. The fine bubble aeration system will be designed for a volatile suspended solids (VSS) destruction (assumes 80% solids are volatile) of approximately 40 percent. Decanters in the tank will allow sludge thickening between 1 and 2 percent prior to dewatering. Dual sludge pumps (one duty and one standby) and one screw press with a hydraulic loading rate of 41 gpm will be installed to dewater sludge and produce cake between 20 and 24 percent. Dewatered sludge (approximately 3,300 wet tons per year) will be transported to the Oak Ridge Landfill in Dorchester County. A Dewatering Building will include space needed for the blowers, screw press, conveyor, polymer system and associated electrical equipment, and will include space for the addition of a second screw press in the future. The WRF site will include space to accommodate a second sludge holding tank for future expansion.

Figure 1 Central Dorchester WRF, Effluent Force Main and Discharge Location (Not To Scale)

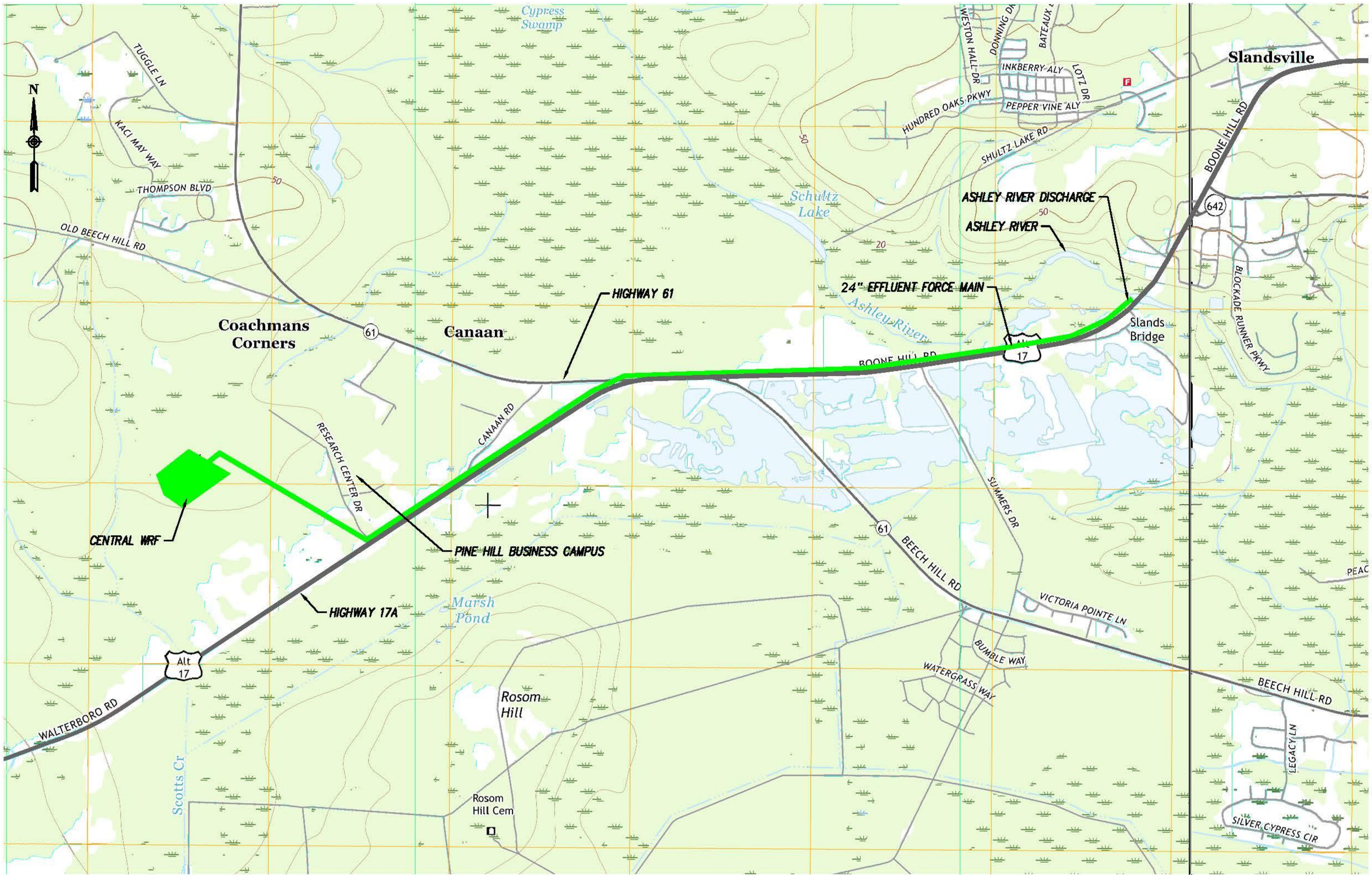

Figure 2 Proposed Liquids and Solids Treatment Process Train (4 MGD)

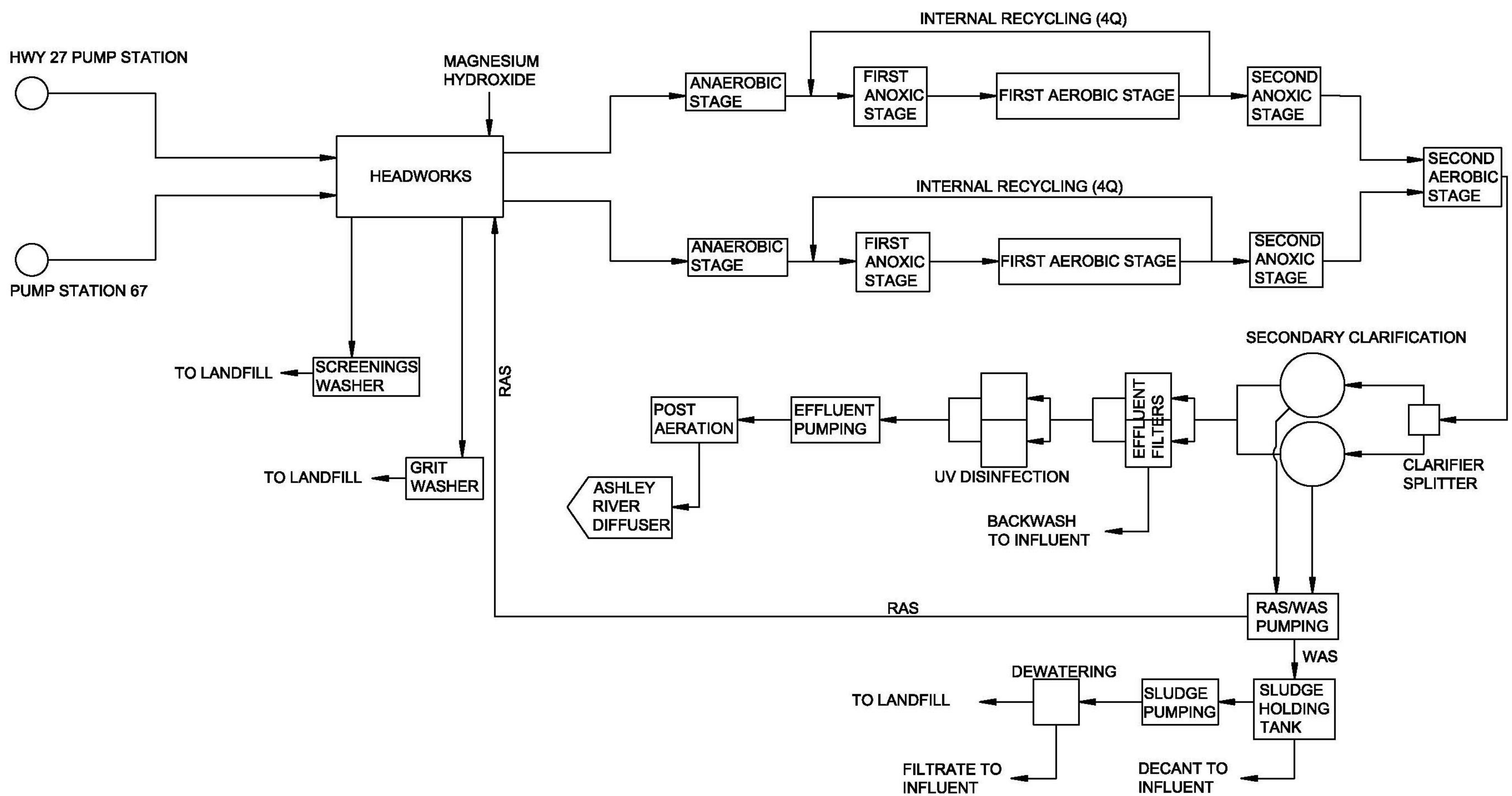

Figure 3 **Central Dorchester WRF Conceptual Site Plan**
(Approximate Scale 1 : 150)

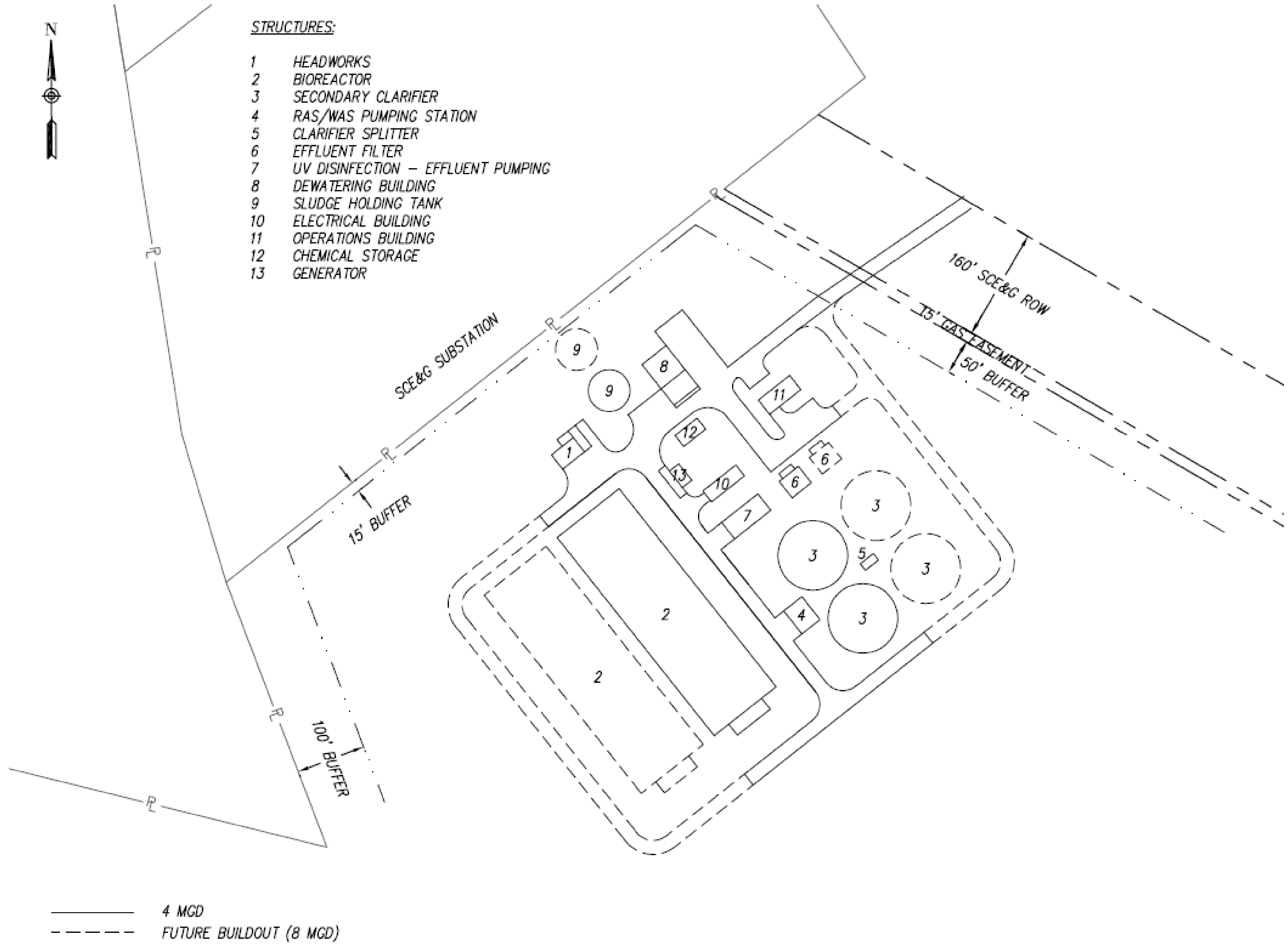

Table 2 Central Dorchester WRF Unit Processes (4 MGD)

Process	Description	Number of Units, Size/Capacity
Alkalinity Adjustment System	Magnesium Hydroxide Feed System	One storage tank, One metering pump skid
Influent Screens	Mechanically Cleaned Bar Screens	Two, 6 mm openings
Grit Removal	Stacked Tray	One, 9 ft diameter
Bioreactor	5-Stage Bardenpho	Two basins, each 2 MG
Secondary Clarification	Circular, Center-feed Clarifiers	Two basins, 100 ft diameter, 16 ft sidewater depth
Tertiary Filtration	Cloth Media Disk Filters	Two basins, each with 12 disks
Disinfection	Ultraviolet (UV)	Two channels, 3 banks / channel, min. dosage 35 mJ/cm^2
Sludge Holding	Circular Sludge Holding Tank	One tank, 0.5 MG, fine bubble aeration
Sludge Dewatering	Screw Press	One press, 41 gpm
Effluent Pumping	Vertical Turbine Pumps	Three, 5 MGD (two duty, one standby)
Post Aeration	Cascade Aeration System	One stepped basin, located near discharge

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Contact: Jon Osterritter

Engineer: W.K. Dickson & Co., Inc.
162 Seven Farms Drive, Suite 210
Charleston, SC 29492
(843) 416-5560
Contact: Bill Young, PE

Service Area

Dorchester County Water and Sewer's designated wastewater service area includes 263 square miles as shown by Figure 4. The County provides water and sewer service to residential, commercial and industrial users. The County serves approximately 8,800 water customers and 25,000 sanitary sewer customers.

The County owns and operates two existing wastewater treatment facilities: the Upper WWTP in Saint George and the Lower WWTP in North Charleston, with existing rated capacities of 1.8 and 8.0 MGD, respectively. The wastewater service area is comprised of 135 pump stations, 115 miles of force main, 10,200 manholes, and 315 miles of gravity sewer lines.

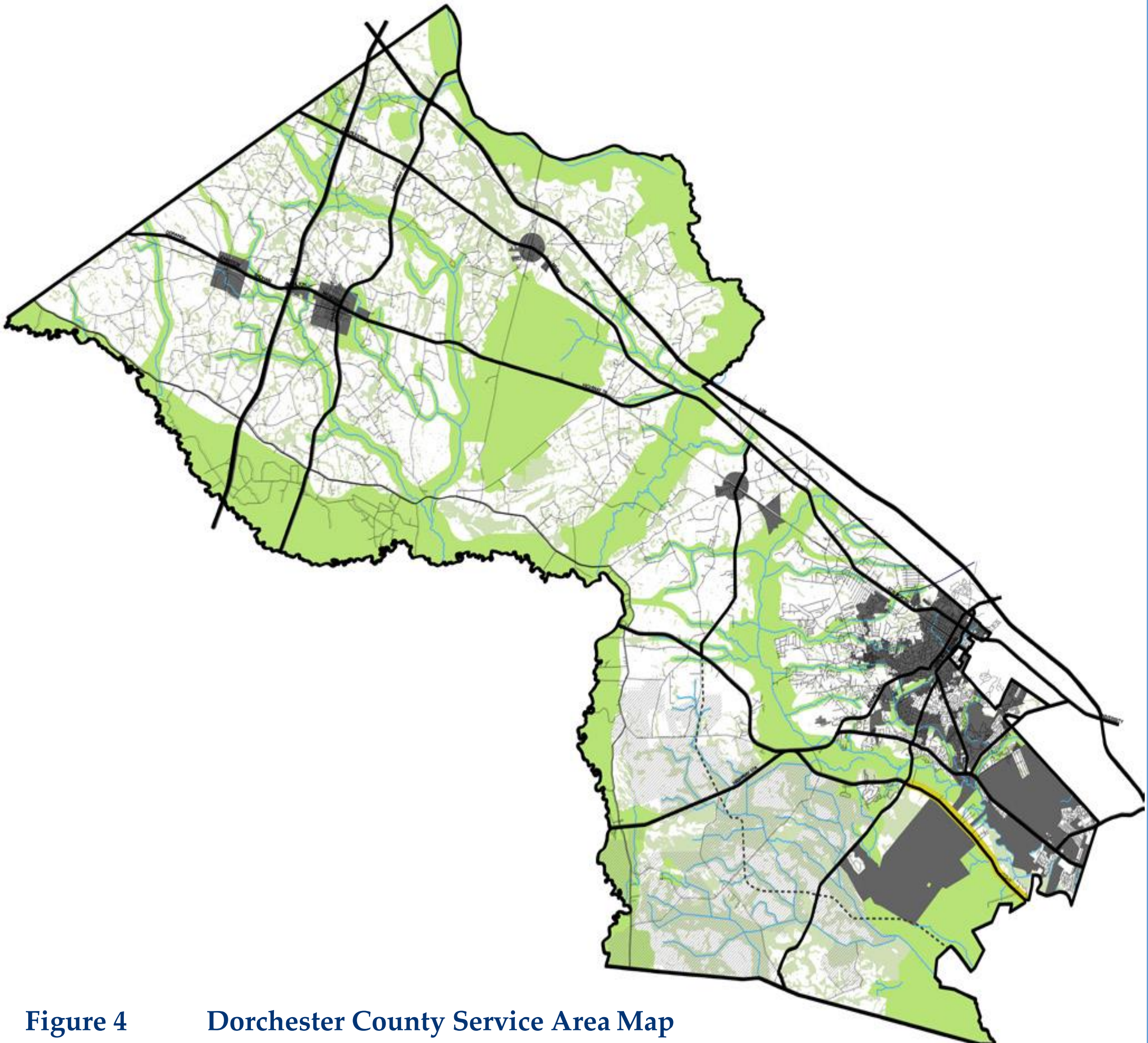


Figure 4 **Dorchester County Service Area Map**
Source: Dorchester County Comprehensive Plan (2018)

Future Land Use and Population Projections

Future land use and population projections were provided by the County as part of the "*Dorchester County Comprehensive Plan 2018*."

The comprehensive plan contains the following population projections for the County through the year 2030:

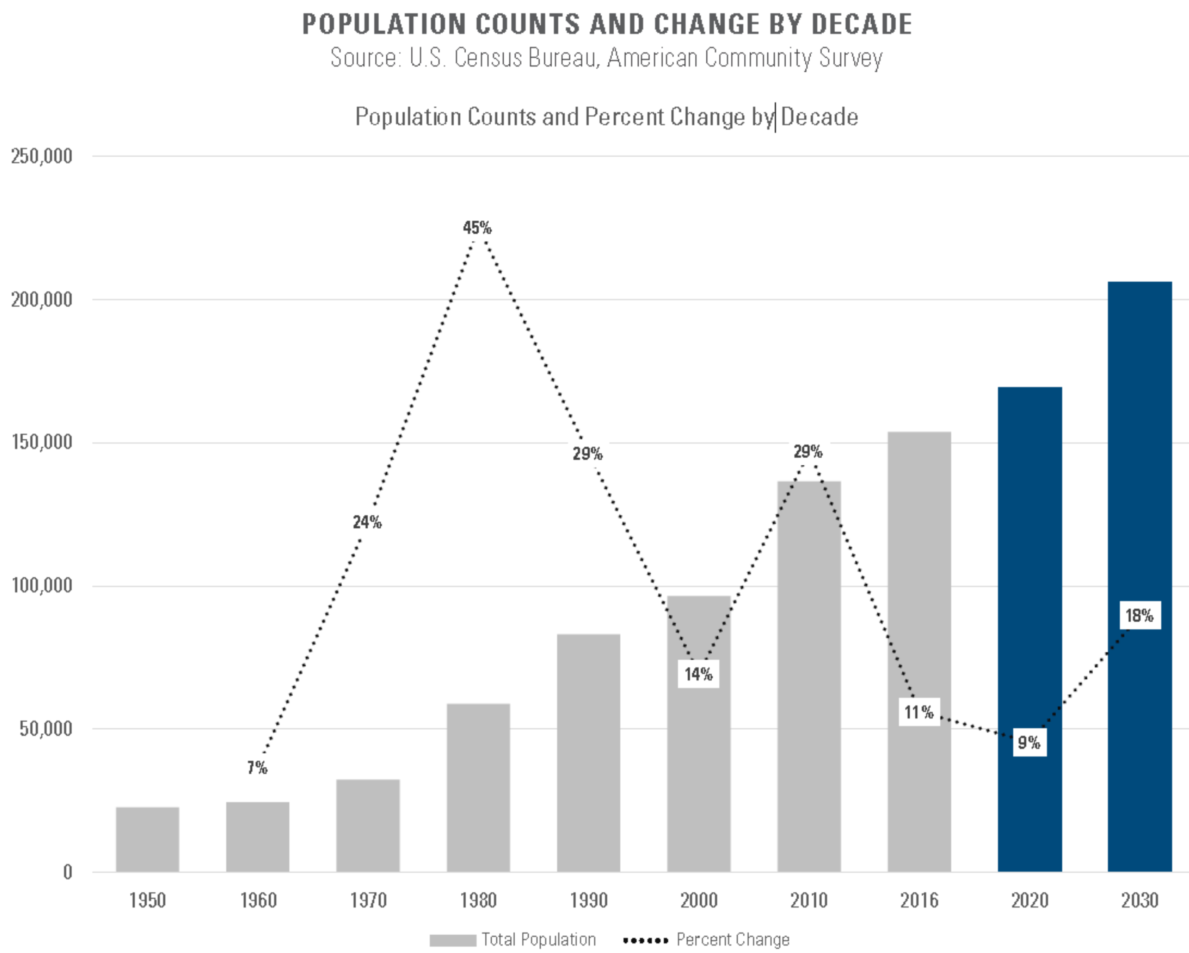


Figure 5 **Dorchester County Population Growth**
Source: Dorchester County Comprehensive Plan (2018)

The U.S. Census Bureau estimated a 2017 Dorchester County population of 156,456 residents with 2.77 people per household. The comprehensive plan estimates a 2030 population of 206,100 persons, which represents a population growth of 49,644 (32%) people over 13 years.

Future Residential Flow Projections

2030 (11 years from present)

As described in the previous section, the population increase from 2017 through the year 2030 is predicted to be 49,644 people. In order to project future flows, the number of households or equivalent residential units (ERUs) this population represents is multiplied by the SCDHEC approved unit contributory loading per ERU (250 gallons per day per ERU).

The number of ERUs is calculated by dividing population by the average household size:

$$\text{New ERUs} = \frac{49{,}644\ people}{2.77\ people\ per\ household} = 17{,}922\ \text{ERUs}$$

From this number of new ERUs, the increase in wastewater flowrate is calculated as:

Q = (17,922 new ERUs)(250 gallons per day per ERU) = 4.5 MGD

Using a consistent rate of growth, the increase in wastewater flowrate from present (2019) to year 2030 is calculated as:

$$4.5 \text{ MGD} \frac{(2030-2019)}{(2030-2017)} = 3.8 \text{ MGD}$$

Future Residential Flow Projections
2039 (20 years from present)

The predicted growth rate described above for the years 2017-2030 is approximately 3,819 people per year. This represents 1,379 new households each year. Applying this figure as calculated above for the years 2019 – 2039 results in the following:

$$(2039 - 2019) = 20 \text{ years},$$

$$(20\ years)(1{,}379\ households\ per\ year) = 27{,}580\ \text{ households}$$

Q = (27,580 new households)(250 gallons per day per household) = 6.9 MGD

The total increase in flow over the twenty-year period of 2019 – 2039 is 6.9 MGD.

Future Residential Flow Projections
2054 (35 years from present)

Applying the same growth rate from the years 2017 – 2030 results in the following calculations for the years 2019 - 2054:

$$(2054 - 2019) = 35 \text{ years},$$

$$(35\ years)(1{,}379\ households\ per\ year) = 48{,}265\ \text{ households}$$

Q = (48,265 new households)(250 gallons per dayper household) = 12.1 MGD

The total increase in flow over the thirty five-year period of 2019 – 2054 is 12.1 MGD.

Table 3 Projected Growth in Residential Flows (beyond Year 2019)

CRITERIA	Assumed Peaking Factor	Year 2030	Year 2039	Year 2054
Annual Average (AA)	1	3.8	6.9	12.1
Maximum Month (MM)	1.15	4.4	7.9	13.9
Peak Day (PD)	1.55	5.9	10.7	18.8
Peak Hour (PH)	2.5	9.5	17.3	30.3

Future Land Use and Commercial / Industrial Flow Projections

Utility Advisors' Network, Inc. completed a rate study for Dorchester County in March 2013 (*Water and Wastewater Rate Study – FY 2014*), which included estimates of metered industrial and commercial flows through the year 2018. The 2018 annual average daily metered commercial and industrial flow in the Lower Dorchester WWTP system was estimated at approximately 0.4 MGD. The County's existing Lower WWTP has a rated treatment capacity of 8 MGD and annual average flows were approximately 6 MGD in 2018. Using this ratio of commercial and industrial flows to total system flows (0.4 MGD ÷ 6 MGD = 7%), projected growth in commercial and industrial flows is estimated in the table below.

Table 4 Projected Growth in Commercial / Industrial Flows (beyond Year 2019)

CRITERIA	Assumed Peaking Factor	Year 2030	Year 2039	Year 2054
Annual Average (AA)	1	0.3	0.5	0.8
Maximum Month (MM)	1.15	0.3	0.6	0.9
Peak Day (PD)	1.55	0.5	0.8	1.2
Peak Hour (PH)	2.5	0.8	1.3	2.0

Combined Residential & Commercial / Industrial Flow Projections

The table below includes growth estimates for the combined residential, commercial and industrial flows.

Table 5 Projected Growth in Total Flows (beyond Year 2019)

CRITERIA	Assumed Peaking Factor	Year 2030	Year 2039	Year 2054
Annual Average (AA)	1	4.1	7.4	12.9
Maximum Month (MM)	1.15	4.7	8.5	14.8
Peak Day (PD)	1.55	6.4	11.5	20.0
Peak Hour (PH)	2.5	10.7	18.6	32.3

The County's existing Upper and Lower WWTPs have rated treatment capacities of 1.8 and 8.0 MGD, respectively. The Upper and Lower WWTPs currently have annual average daily flows of approximately 1 and 6 MGD, respectively. Most of the projected growth in Dorchester County is projected to occur in the central and lower portions of its service area. Based upon the influent wastewater flow projections identified in Table 5, the additional treatment capacity needs (beyond the existing 9.8 MGD total) are summarized in Table 6.

Table 6 Necessary Additional Treatment Capacities for Growth

Year	Maximum Month (MGD)
2030	2.1
2039	5.7
2054	12.0

An expansion of the existing Lower Dorchester WWTP from 8 MGD to 16 MGD is the most cost effective management plan to treat the projected maximum month flows in the 20-year planning period. The Lower Dorchester WWTP expansion to 16 MGD and construction of a new 4 MGD Central Dorchester WRF will be the most cost effective management plan to treat the projected maximum month flows thru Year 2054.

B. Description of Waste

The influent wastewater to the Central Dorchester WRF will consist of domestic, commercial, and industrial wastewaters that are currently being collected by the County's existing wastewater collection system and treated at the Lower Dorchester WWTP.

C. Characteristics of Waste

The characteristics of the influent wastewater to the Central Dorchester WRF will be similar to that received by the County's existing Lower Dorchester WWTP. This is due to the fact that the Central Dorchester WRF will receive flow that is diverted from the Lower Dorchester WWTP. The characteristics of the wastewater include:

Table 7 Influent Characteristics (4 MGD)

Parameter	Monthly Average
$cBOD_5$ (mg/L)	240
TSS (mg/L)	228
NH_3-N (mg/L)	31
TKN (mg/L)	48
TP (mg/L)	10
Oil & Grease (mg/L)	100
pH	7.2[A]

[A] pH and alkalinity will be increased through the addition of magnesium hydroxide in the collection system upstream of the WRF, as well as with an additional magnesium hydroxide system located at the new facility.

D. Treatability of Waste

The influent wastewater is the same wastewater that is currently being treated at the existing Lower Dorchester WWTP. The new Central Dorchester WRF will consist of similar treatment technologies as those at the Lower Dorchester, and as a result the waste will be satisfactorily treated by the new facility (See Appendix A for effluent monitoring data).

Residuals generated by the new facility, along with the treatment and ultimate disposal methods, are provided in Table 8.

Table 8 Central Dorchester WRF Process Residuals

Process	Residual Description	Treatment Method	Disposal Method
Influent Screens	Coarse screenings (> 6.0 mm)	Compacter dewatering	Oakridge Landfill
Grit Removal	Sand, bone, grit (75 microns – 6.0 mm)	Washing	Oakridge Landfill
Activated Sludge	Waste activated sludge	Aeration, thickening, & dewatering	Oakridge Landfill

The residuals generated by the new Central Dorchester WRF are expected to be very similar to the residuals from the Lower Dorchester WWTP (see Appendix B for residuals analyses from the Lower Dorchester WWTP).

E. Location of Plant and Point of Discharge

The proposed location of the Central Dorchester WRF is at the Pine Hill Business Campus along Highway 17A, west of its intersection with Highway 61 (latitude 32° 57′ 36.5″ N - longitude 80° 18′ 35.2″ W; approximately 10 miles west of Summerville, SC, and 8 miles south of Ridgeville, SC) as shown previously in Figure 1. The effluent will be pumped to the Ashley River to an outfall located near the Highway 17A bridge (latitude 32° 58′ 07.2″ N - longitude 80° 15′ 11.1″ W).

F. Physical Characteristics of Proposed Site.

This section is not required by R.61-67, as this project is not directly influenced by local soil/groundwater characteristics. However, it is included to provide information that will be relevant to the design of the project.

The proposed site is currently forested with planted pines. The soils on the site are classified as Coosaw loamy fine sands, Mouzon fine sandy loams, and Yauhannah loamy fine sands.

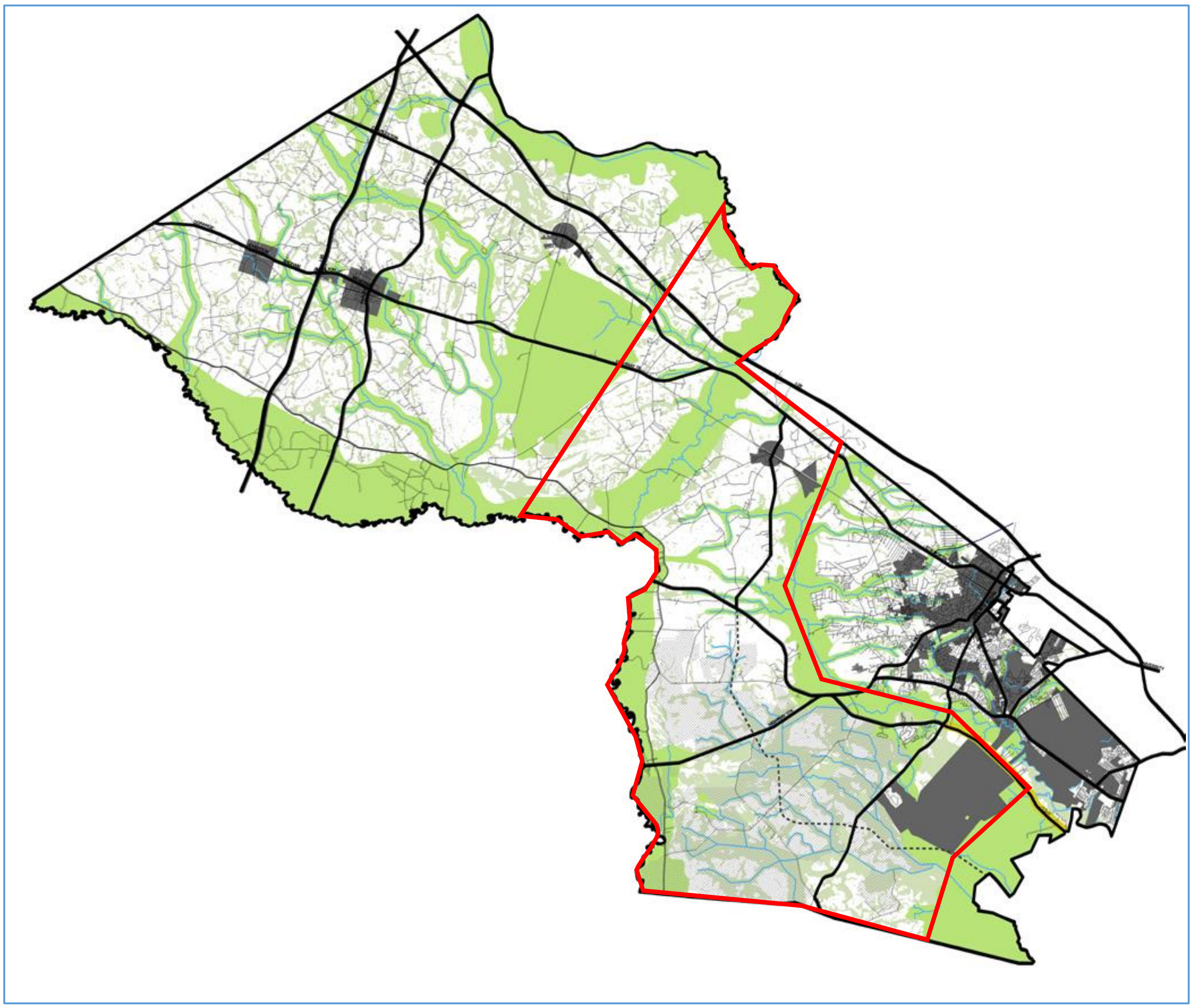
Based upon a review of the U.S. Fish and Wildlife Service's wetland inventory mapping, there are no wetlands on the proposed site.

Based upon a review of FEMA flood hazard mapping, the proposed site is located entirely within Zone X, "Area of Minimal Flood Hazard", which places it above the 100-year flood elevation.

G. General Layout of Area(s) to be Served

The new Central Dorchester WRF will accept and treat wastewater generally from the central and lower portions of Dorchester County that are west of the Ashley River, as indicated by Figure 6.

Figure 6
Central Dorchester WRF Service Area Map
Base image source: Dorchester County Comprehensive Plan (2018)



H. Receiving Waters

The new Central Dorchester WRF outfall is proposed to be located into the Ashley River at the Highway 17A bridge. The river at this point is approximately 70 to 90 feet wide, and is subject to tidal influences. The location of the proposed outfall is shown in Figure 7.

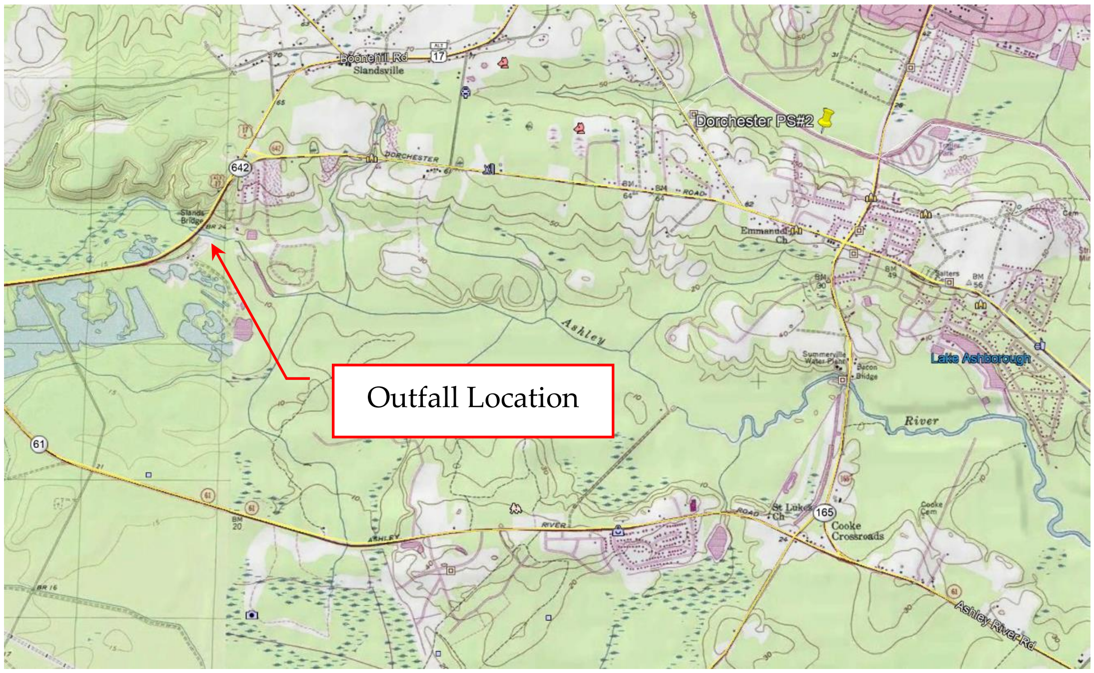


Figure 7 Central Dorchester WRF Outfall Location Map

I. Impact of Discharge on Receiving Waters

In February 2018, Dorchester County requested a wasteload allocation from SCDHEC for a Central Dorchester WRF discharge. Wasteload allocations for three discharge locations were requested to facilitate planning and land acquisition for the proposed treatment facility. On June 14, 2018, SCDHEC provided a response letter with a speculative wasteload allocation to the Ashley River (see Appendix C). SCDHEC stated that the proposed Central Dorchester WRF discharge would be incorporated into the Charleston area TMDL for the Charleston Harbor and Cooper, Wando, and Ashley Rivers. Location #3, Ashley River at Highway 17A Bridge (refer to Figure 7), was determined to be a feasible discharge location per SCDHEC water quality modeling. With regards to ultimate oxygen demand (UOD), SCDHEC stated that a pound per pound allocation of UOD load from another treatment facility on the Ashley River would be required to maintain the TMDL. The speculative limits for discharge Location #3 also included a stringent monthly average ammonia concentration limit of 0.5 mg/L NH_3-N due to uncertainty in DHEC's water quality model results. In an October 2018 meeting, SCDHEC indicated that flexibility may be granted for the ammonia limit in the winter months of November through February.

Dorchester County proposes to re-distribute the Lower Dorchester WWTP UOD allocation between the Lower Dorchester WWTP and the Central Dorchester WRF. The current Lower Dorchester WWTP NPDES permit limits (at 12 MGD) for UOD are 2,365 pound per day (lb/d) in the summer months (March through October) and 4,126 lb/d in the winter months (November through February). For a shared or "bubble" NPDES permit limit for UOD, a maximum $cBOD_5$ and ammonia concentration threshold can be calculated for each facility using a measured f-ratio (f-ratio of 2.22 for Lower Dorchester WWTP). As an example under a UOD "bubble" permit strategy for a future 16 MGD Lower Dorchester WWTP and 4 MGD Central Dorchester WRF, the UOD permit limit of 2,365 lb/d (during the summer months) would be achieved with concentration limits of 5.0 mg/L $cBOD_5$ and 0.8 mg/L of ammonia for the Lower Dorchester WWTP and 3.0 mg/L $cBOD_5$ and 0.5 mg/L of ammonia for the Central Dorchester WRF (assumes f-ratio of 2.22).

During the speculative limit and wasteload allocation discussions, SCDHEC indicated that future nutrient limits are a possibility in the Charleston Harbor and Cooper, Wando, and Ashley Rivers. It was unclear of the timing of the proposed nutrient limits, but they are not anticipated in the next two NPDES permit cycles. SCDHEC is in the process of collecting data in the watershed for model development, calibration, and validation. In response, Dorchester County has agreed to construct a five-stage biological process for the Lower Dorchester WWTP expansion and the Central Dorchester WRF in anticipation of future nutrient limits.

J. Equipment and Service Failure or Shutdown

The new Central Dorchester WRF will be designed to meet, and in some cases exceed the requirements of the facility reliability classification that is issued by SCDHEC.

In the case of electrical service outages, the facility will be designed to include standby power generation for the screenings and grit facilities, aeration basins, clarifiers, return sludge pump station, disinfection, and operations building including critical lighting and ventilation.

In the case of equipment or other system failures, the facility will be designed to include the following:

- Backup pumps for each set of pumps which perform the same function. The capacity of the pumps shall be such that with any one (1) pump out of service, the remaining pumps shall have capacity to handle the peak flow.
- Mechanically cleaned bar screens will include one duty and one backup screen.
- Two 5-stage Bardenpho process treatment trains will be provided (ie. two aeration basins, etc.). The aeration system shall maintain the design oxygen transfer with the largest aerator or blower out of service.
- Two secondary clarifiers will be provided and have the capacity to treat 50% of the design average flow with one clarifier out of service.
- Two basins with tertiary filters will be provided and have the capacity to treat 50% of the design average flow with one basin out of service.

- Two channels with UV disinfection will be provided and have the capacity to treat 50% of the design average flow with one channel out of service.

K. Alternatives Analysis and Consolidation of Facilities

Alternatives Analysis Options:

1) Water recycle or reuse:

   Recycling of water is not applicable to this project. The proposed facility will be designed to produce "reuse" quality water. However, there are currently no significant opportunities to provide service to reuse water customers. This may change over the next 20 years, and Dorchester County will continue evaluating future opportunities for providing reuse water to its customers.

2) Use of other discharge locations:

   Three discharge locations were evaluated for this project. The proposed location was selected based upon engineering considerations and as suggested by SCDHEC.

3) Connection to other wastewater treatment facilities:

   Wholesale wastewater connections to Summerville CPW and the North Charleston Sewer District were evaluated. Life cycle costs analyses determined that both of these options were significantly higher in cost (see Appendix D).

4) Use of land application:

   The use of land application was investigated during the planning phase of this project. The results of the planning level investigation indicated that large enough areas of suitable soils with favorable groundwater conditions do not exist within reasonable distances of the project area.

5) Product or raw material substation:

   Not applicable to this project.

6) Any other treatment option or alternative:

   The facility, as proposed, will be designed to produce high quality, tertiary grade effluent, suitable for reuse purposes. The only option for a higher quality effluent would be to construct membrane bioreactors. This alternative was explored and found to have a higher capital cost than the option selected.

7) Consolidation of Facilities (208 Water Quality Management Plan):

   The future increases in wastewater flow that have been presented in this report were utilized to perform an economic analysis comparing the costs of the following three options:

   a) Construct the new 4 MGD Central Dorchester WRF;

b) Construct pump station and pipeline to convey the future wastewater flows to the North Charleston Sewer District for treatment; and

c) Construct pump station and pipeline to convey the future wastewater flows to Summerville CPW for treatment.

Based upon the results of a life cycle analysis (Appendix D), construction of the new Central Dorchester WRF is the lowest cost option.

L. Pretreatment Facilities

Not applicable to this project.

APPENDICES

A. Effluent Wastewater Monitoring Data

B. Residuals Analysis

C. SCDHEC Speculative Wasteload Allocation

D. Life Cycle Cost Analysis

Appendix A

Effluent Wastewater Monitoring Data

Report of Analysis

Dorchester County Water & Sewer
2900 Landing Parkway
North Charleston, SC 29420
Attention: Bruce Snyder

Project Name: **NPDES Permit Renewal**

Lot Number: **SE12036**
Date Completed: **06/02/2017**

Grant Wilton
Project Manager

The following non-paginated documents are considered part of this report: Chain of Custody Record and Sample Receipt Checklist.

Case Narrative
Dorchester County Water & Sewer
Lot Number: SE12036

Project Name: NPDES Permit Renewal

Project Number:

This Report of Analysis contains the analytical result(s) for the sample(s) listed on the Sample Summary following this Case Narrative. The sample receiving date is documented in the header information associated with each sample.

All results listed in this report relate only to the samples that are contained within this report.

Sample receipt, sample analysis, and data review have been performed in accordance with the most current approved NELAC standards, the Shealy Environmental Services, Inc. ("Shealy") Quality Assurance Management Plan (QAMP), standard operating procedures (SOPs), and Shealy policies. Any exceptions to the NELAC standards, the QAMP, SOPs or policies are qualified on the results page or discussed below.

If you have any questions regarding this report please contact the Shealy Project Manager listed on the cover page.

BOD Analysis - Method SM 5210B
6-The Biochemical Oxygen Demand (BOD) result has been reported as less than the stated value due to oxygen depletion of less than 2.0 mg/l at each sample dilution. The sample was prepared with a standard series of dilutions.

Fecal Coliform Analysis
The holding time expired prior to receipt at the laboratory.

Sample Summary

Dorchester County Water & Sewer

Lot Number: SE12036

Project Name: NPDES Permit Renewal

Project Number:

Sample Number	Sample ID	Matrix	Date Sampled	Date Received
001	PLANT EFFLUENT	Aqueous	05/12/2017 0810	05/12/2017
002	PLANT EFFLUENT	Aqueous	05/12/2017 0810	05/12/2017
003	PLANT EFFLUENT	Aqueous	05/12/2017 0810	05/12/2017
004	PLANT EFFLUENT	Aqueous	05/12/2017 0810	05/12/2017
005	PLANT EFFLUENT	Aqueous	05/12/2017 0810	05/12/2017
006	PLANT EFFLUENT	Aqueous	05/12/2017 0800	05/12/2017
007	PLANT EFFLUENT	Aqueous	05/12/2017 0800	05/12/2017
008	PLANT EFFLUENT	Aqueous	05/12/2017 0800	05/12/2017
009	PLANT EFFLUENT	Aqueous	05/12/2017 0800	05/12/2017

(9 samples)

Executive Summary

Dorchester County Water & Sewer

Lot Number: SE12036

Project Name: NPDES Permit Renewal

Project Number:

Sample	Sample ID	Matrix	Parameter	Method	Result	Q	Units	Page
006	PLANT EFFLUENT	Aqueous	Nitrate-Nitrite - N	353.2	6.3		mg/L	10
006	PLANT EFFLUENT	Aqueous	Phosphorus	365.1	1.2		mg/L	10
006	PLANT EFFLUENT	Aqueous	TKN	351.2	0.41		mg/L	10
007	PLANT EFFLUENT	Aqueous	Hardness (total)	SM 2340C-	71		mg/L	11
007	PLANT EFFLUENT	Aqueous	Zinc	200.8	36		ug/L	12
009	PLANT EFFLUENT	Aqueous	TDS	SM 2540C-	300		mg/L	16

(6 detections)

Client: **Dorchester County Water & Sewer**
Laboratory ID: **SE12036-001**
Description: **PLANT EFFLUENT**
Matrix: **Aqueous**
Date Sampled: **05/12/2017 0810**
Project Name: **NPDES Permit Renewal**
Date Received: **05/12/2017**
Project Number:

Run	Prep Method	Analytical Method	Dilution	Analysis Date	Analyst	Prep Date	Batch
1		(HEM (oil and) 1664B	1	05/18/2017 1124	JMM		42359

Parameter	CAS Number	Analytical Method	Result	Q	PQL	Units	Run
HEM (oil and grease)		1664B	ND		4.7	mg/L	1

PQL = Practical quantitation limit
B = Detected in the method blank
E = Quantitation of compound exceeded the calibration range
H = Out of holding time
ND = Not detected at or above the PQL
J = Estimated result < PQL and ≥ MDL
P = The RPD between two GC columns exceeds 40%
N = Recovery is out of criteria
Where applicable, all soil sample analysis are reported on a dry weight basis unless flagged with a "W"

Client: **Dorchester County Water & Sewer**	Laboratory ID: **SE12036-002**
Description: **PLANT EFFLUENT**	Matrix: **Aqueous**
Date Sampled: **05/12/2017 0810**	Project Name: **NPDES Permit Renewal**
Date Received: **05/12/2017**	Project Number:

Run	Prep Method	Analytical Method	Dilution	Analysis Date	Analyst	Prep Date	Batch
1	624	624	1	05/15/2017 1712	ALL		42003

Parameter	CAS Number	Analytical Method	Result	Q	PQL	Units	Run
Acrolein	107-02-8	624	ND		5.0	ug/L	1
Acrylonitrile	107-13-1	624	ND		5.0	ug/L	1
Benzene	71-43-2	624	ND		2.0	ug/L	1
Bromodichloromethane	75-27-4	624	ND		2.0	ug/L	1
Bromoform	75-25-2	624	ND		2.0	ug/L	1
Bromomethane (Methyl bromide)	74-83-9	624	ND		2.0	ug/L	1
Carbon tetrachloride	56-23-5	624	ND		2.0	ug/L	1
Chlorobenzene	108-90-7	624	ND		2.0	ug/L	1
Chloroethane	75-00-3	624	ND		2.0	ug/L	1
2-Chloroethylvinylether	110-75-8	624	ND		5.0	ug/L	1
Chloroform	67-66-3	624	ND		2.0	ug/L	1
Chloromethane (Methyl chloride)	74-87-3	624	ND		2.0	ug/L	1
Dibromochloromethane	124-48-1	624	ND		2.0	ug/L	1
1,2-Dichlorobenzene	95-50-1	624	ND		2.0	ug/L	1
1,3-Dichlorobenzene	541-73-1	624	ND		2.0	ug/L	1
1,4-Dichlorobenzene	106-46-7	624	ND		2.0	ug/L	1
1,1-Dichloroethane	75-34-3	624	ND		2.0	ug/L	1
1,2-Dichloroethane	107-06-2	624	ND		2.0	ug/L	1
1,1-Dichloroethene	75-35-4	624	ND		2.0	ug/L	1
trans-1,2-Dichloroethene	156-60-5	624	ND		2.0	ug/L	1
1,2-Dichloropropane	78-87-5	624	ND		2.0	ug/L	1
-1,3-Dichloropropene	10061-01-5	624	ND		2.0	ug/L	1
trans-1,3-Dichloropropene	10061-02-6	624	ND		2.0	ug/L	1
Ethylbenzene	100-41-4	624	ND		2.0	ug/L	1
Methylene chloride	75-09-2	624	ND		2.0	ug/L	1
1,1,2,2-Tetrachloroethane	79-34-5	624	ND		2.0	ug/L	1
Tetrachloroethene	127-18-4	624	ND		2.0	ug/L	1
Toluene	108-88-3	624	ND		2.0	ug/L	1
1,2,4-Trichlorobenzene	120-82-1	624	ND		2.0	ug/L	1
1,1,1-Trichloroethane	71-55-6	624	ND		2.0	ug/L	1
1,1,2-Trichloroethane	79-00-5	624	ND		2.0	ug/L	1
Trichloroethene	79-01-6	624	ND		2.0	ug/L	1
Vinyl chloride	75-01-4	624	ND		2.0	ug/L	1

Surrogate	Q	Run 1 % Recovery	Acceptance Limits
1,2-Dichloroethane-d4		110	70-130
Toluene-d8		107	70-130
Bromofluorobenzene		104	70-130

PQL = Practical quantitation limit B = Detected in the method blank E = Quantitation of compound exceeded the calibration range H = Out of holding time
ND = Not detected at or above the PQL J = Estimated result < PQL and ≥ MDL P = The RPD between two GC columns exceeds 40% N = Recovery is out of criteria
applicable, all soil sample analysis are reported on a dry weight basis unless flagged with a "W"

Client: **Dorchester County Water & Sewer** | Laboratory ID: **SE12036-003**
Description: **PLANT EFFLUENT** | Matrix: **Aqueous**
Date Sampled: **05/12/2017 0810** | Project Name: **NPDES Permit Renewal**
Date Received: **05/12/2017** | Project Number:

Run	Prep Method	Analytical Method	Dilution	Analysis Date	Analyst	Prep Date	Batch
1		(Phenolics) 420.4	1	05/17/2017 1603	HKL	05/16/2017 1845	42173

Parameter	CAS Number	Analytical Method	Result	Q	PQL	Units	Run
Phenolics		420.4	ND		0.0050	mg/L	1

PQL = Practical quantitation limit B = Detected in the method blank E = Quantitation of compound exceeded the calibration range H = Out of holding time
ND = Not detected at or above the PQL J = Estimated result < PQL and ≥ MDL P = The RPD between two GC columns exceeds 40% N = Recovery is out of criteria
Where applicable, all soil sample analysis are reported on a dry weight basis unless flagged with a "W"

Client: **Dorchester County Water & Sewer**
Laboratory ID: **SE12036-004**
Description: **PLANT EFFLUENT**
Matrix: **Aqueous**
Date Sampled: **05/12/2017 0810**
Project Name: **NPDES Permit Renewal**
Date Received: **05/12/2017**
Project Number:

Run	Prep Method	Analytical Method	Dilution	Analysis Date	Analyst	Prep Date	Batch
1	10-204-00-1-X	(Cyanide - To) SM 4500-CN E-	1	05/17/2017 1255	HKL	05/17/2017 1007	42214

Parameter	CAS Number	Analytical Method	Result	Q	PQL	Units	Run
Cyanide - Total	57-12-5	SM 4500-CN	ND		0.010	mg/L	1

PQL = Practical quantitation limit
B = Detected in the method blank
E = Quantitation of compound exceeded the calibration range
H = Out of holding time
ND = Not detected at or above the PQL
J = Estimated result < PQL and ≥ MDL
P = The RPD between two GC columns exceeds 40%
N = Recovery is out of criteria
Where applicable, all soil sample analysis are reported on a dry weight basis unless flagged with a "W"

Client: **Dorchester County Water & Sewer**
Laboratory ID: **SE12036-005**
Description: **PLANT EFFLUENT**
Matrix: **Aqueous**
Date Sampled: **05/12/2017 0810**
Project Name: **NPDES Permit Renewal**
Date Received: **05/12/2017**
Project Number:

Run	Prep Method	Analytical Method	Dilution	Analysis Date	Analyst	Prep Date	Batch
1		(Fecal Colifo) Colilert-18 ATP	1	05/13/2017 1307	MSG	05/12/2017 1705	

Parameter	CAS Number	Analytical Method	Result	Q	PQL	Units	Run
Fecal Coliform (MPN)		Colilert-18	ND	H	1	MPN/100mL	1

PQL = Practical quantitation limit B = Detected in the method blank E = Quantitation of compound exceeded the calibration range H = Out of holding time
ND = Not detected at or above the PQL J = Estimated result < PQL and ≥ MDL P = The RPD between two GC columns exceeds 40% N = Recovery is out of criteria
Where applicable, all soil sample analysis are reported on a dry weight basis unless flagged with a "W"

Client: **Dorchester County Water & Sewer**
Laboratory ID: **SE12036-006**
Description: **PLANT EFFLUENT**
Matrix: **Aqueous**
Date Sampled: **05/12/2017 0800**
Project Name: **NPDES Permit Renewal**
Date Received: **05/12/2017**
Project Number:

Run	Prep Method	Analytical Method	Dilution	Analysis Date	Analyst	Prep Date	Batch
1	350.1	(Ammonia - N) 350.1	1	05/17/2017 1344	DMA		42225
1		(Nitrate-Nitr) 353.2	10	05/18/2017 1732	HKL		42384
1		(Phosphorus) 365.1	3	05/17/2017 1229	DMA	05/16/2017 1004	42089
1	351.4	(TKN) 351.2	1	05/18/2017 1232	DMA	05/17/2017 1455	42288

Parameter	CAS Number	Analytical Method	Result	Q	PQL	Units	Run
Ammonia - N (gas diffusion)		350.1	ND		0.10	mg/L	1
Nitrate-Nitrite - N		**353.2**	**6.3**		**0.20**	**mg/L**	**1**
Phosphorus	**7723-14-0**	**365.1**	**1.2**		**0.030**	**mg/L**	**1**
TKN		**351.2**	**0.41**		**0.10**	**mg/L**	**1**

PQL = Practical quantitation limit
B = Detected in the method blank
E = Quantitation of compound exceeded the calibration range
H = Out of holding time
ND = Not detected at or above the PQL
J = Estimated result < PQL and ≥ MDL
P = The RPD between two GC columns exceeds 40%
N = Recovery is out of criteria
Where applicable, all soil sample analysis are reported on a dry weight basis unless flagged with a "W"

Client: **Dorchester County Water & Sewer**	Laboratory ID: **SE12036-007**
Description: **PLANT EFFLUENT**	Matrix: **Aqueous**
Date Sampled: **05/12/2017 0800**	Project Name: **NPDES Permit Renewal**
Date Received: **05/12/2017**	Project Number:

Run	Prep Method	Analytical Method	Dilution	Analysis Date	Analyst	Prep Date	Batch
1		(Hardness (to) SM 2340C-2011	1	05/22/2017 1412	BWS		42593

Parameter	CAS Number	Analytical Method	Result	Q	PQL	Units	Run
Hardness (total)		**SM 2340C-20**	**71**		**10**	**mg/L**	**1**

PQL = Practical quantitation limit B = Detected in the method blank E = Quantitation of compound exceeded the calibration range H = Out of holding time
ND = Not detected at or above the PQL J = Estimated result < PQL and ≥ MDL P = The RPD between two GC columns exceeds 40% N = Recovery is out of criteria
Where applicable, all soil sample analysis are reported on a dry weight basis unless flagged with a "W"

Client: **Dorchester County Water & Sewer** Laboratory ID: **SE12036-007**

Description: **PLANT EFFLUENT** Matrix: **Aqueous**

Date Sampled: **05/12/2017 0800** Project Name: **NPDES Permit Renewal**

Date Received: **05/12/2017** Project Number:

Run	Prep Method	Analytical Method	Dilution	Analysis Date	Analyst	Prep Date	Batch
1	200.2	200.8	1	05/18/2017 0301	BNW	05/17/2017 1036	42187
2	200.2	200.8	1	05/18/2017 1342	BNW	05/17/2017 1036	42187

Parameter	CAS Number	Analytical Method	Result	Q	PQL	Units	Run
Antimony	7440-36-0	200.8	ND		5.0	ug/L	1
Arsenic	7440-38-2	200.8	ND		5.0	ug/L	1
Beryllium	7440-41-7	200.8	ND		1.0	ug/L	2
Cadmium	7440-43-9	200.8	ND		0.10	ug/L	1
Chromium	7440-47-3	200.8	ND		5.0	ug/L	1
Copper	7440-50-8	200.8	ND		10	ug/L	1
Lead	7439-92-1	200.8	ND		2.0	ug/L	1
Nickel	7440-02-0	200.8	ND		10	ug/L	1
Selenium	7782-49-2	200.8	ND		5.0	ug/L	1
Silver	7440-22-4	200.8	ND		5.0	ug/L	1
Thallium	7440-28-0	200.8	ND		0.50	ug/L	1
Zinc	**7440-66-6**	**200.8**	**36**		**10**	**ug/L**	**1**

PQL = Practical quantitation limit B = Detected in the method blank E = Quantitation of compound exceeded the calibration range H = Out of holding time

ND = Not detected at or above the PQL J = Estimated result < PQL and ≥ MDL P = The RPD between two GC columns exceeds 40% N = Recovery is out of criteria

Where applicable, all soil sample analysis are reported on a dry weight basis unless flagged with a "W"

Client: **Dorchester County Water & Sewer**
Laboratory ID: **SE12036-008**
Description: **PLANT EFFLUENT**
Matrix: **Aqueous**
Date Sampled: **05/12/2017 0800**
Project Name: **NPDES Permit Renewal**
Date Received: **05/12/2017**
Project Number:

Run	Prep Method	Analytical Method	Dilution	Analysis Date	Analyst	Prep Date	Batch
1	625	625	1	05/24/2017 1505	SES	05/16/2017 1441	42127

Parameter	CAS Number	Analytical Method	Result	Q	PQL	Units	Run
Acenaphthene	83-32-9	625	ND		1.6	ug/L	1
Acenaphthylene	208-96-8	625	ND		1.6	ug/L	1
Anthracene	120-12-7	625	ND		1.6	ug/L	1
Benzidine	92-87-5	625	ND		8.0	ug/L	1
Benzo(a)anthracene	56-55-3	625	ND		1.6	ug/L	1
Benzo(a)pyrene	50-32-8	625	ND		1.6	ug/L	1
Benzo(b)fluoranthene	205-99-2	625	ND		1.6	ug/L	1
Benzo(g,h,i)perylene	191-24-2	625	ND		1.6	ug/L	1
Benzo(k)fluoranthene	207-08-9	625	ND		1.6	ug/L	1
4-Bromophenyl phenyl ether	101-55-3	625	ND		1.6	ug/L	1
Butyl benzyl phthalate	85-68-7	625	ND		8.0	ug/L	1
bis (2-Chloro-1-methylethyl) ether	108-60-1	625	ND		1.6	ug/L	1
4-Chloro-3-methyl phenol	59-50-7	625	ND		1.6	ug/L	1
bis(2-Chloroethoxy)methane	111-91-1	625	ND		1.6	ug/L	1
bis(2-Chloroethyl)ether	111-44-4	625	ND		1.6	ug/L	1
2-Chloronaphthalene	91-58-7	625	ND		1.6	ug/L	1
2-Chlorophenol	95-57-8	625	ND		1.6	ug/L	1
4-Chlorophenyl phenyl ether	7005-72-3	625	ND		1.6	ug/L	1
Chrysene	218-01-9	625	ND		1.6	ug/L	1
Dibenzo(a,h)anthracene	53-70-3	625	ND		1.6	ug/L	1
3,3'-Dichlorobenzidine	91-94-1	625	ND		8.0	ug/L	1
-Dichlorophenol	120-83-2	625	ND		1.6	ug/L	1
Diethylphthalate	84-66-2	625	ND		8.0	ug/L	1
Dimethyl phthalate	131-11-3	625	ND		8.0	ug/L	1
2,4-Dimethylphenol	105-67-9	625	ND		1.6	ug/L	1
Di-n-butyl phthalate	84-74-2	625	ND		8.0	ug/L	1
4,6-Dinitro-2-methylphenol	534-52-1	625	ND		8.0	ug/L	1
2,4-Dinitrophenol	51-28-5	625	ND		8.0	ug/L	1
2,4-Dinitrotoluene	121-14-2	625	ND		3.2	ug/L	1
2,6-Dinitrotoluene	606-20-2	625	ND		3.2	ug/L	1
Di-n-octylphthalate	117-84-0	625	ND		8.0	ug/L	1
bis(2-Ethylhexyl)phthalate	117-81-7	625	ND		8.0	ug/L	1
Fluoranthene	206-44-0	625	ND		1.6	ug/L	1
Fluorene	86-73-7	625	ND		1.6	ug/L	1
Hexachlorobenzene	118-74-1	625	ND		1.6	ug/L	1
Hexachlorobutadiene	87-68-3	625	ND		1.6	ug/L	1
Hexachlorocyclopentadiene	77-47-4	625	ND		8.0	ug/L	1
Hexachloroethane	67-72-1	625	ND		1.6	ug/L	1
Indeno(1,2,3-c,d)pyrene	193-39-5	625	ND		1.6	ug/L	1
Isophorone	78-59-1	625	ND		1.6	ug/L	1
Naphthalene	91-20-3	625	ND		1.6	ug/L	1
Nitrobenzene	98-95-3	625	ND		1.6	ug/L	1
2-Nitrophenol	88-75-5	625	ND		3.2	ug/L	1
4-Nitrophenol	100-02-7	625	ND		8.0	ug/L	1

PQL = Practical quantitation limit
B = Detected in the method blank
E = Quantitation of compound exceeded the calibration range
H = Out of holding time
ND = Not detected at or above the PQL
J = Estimated result < PQL and ≥ MDL
P = The RPD between two GC columns exceeds 40%
N = Recovery is out of criteria
applicable, all soil sample analysis are reported on a dry weight basis unless flagged with a "W"

Client: **Dorchester County Water & Sewer** Laboratory ID: **SE12036-008**

Description: **PLANT EFFLUENT** Matrix: **Aqueous**

Date Sampled: **05/12/2017 0800** Project Name: **NPDES Permit Renewal**

Date Received: **05/12/2017** Project Number:

Run	Prep Method	Analytical Method	Dilution	Analysis Date	Analyst	Prep Date	Batch
1	625	625	1	05/24/2017 1505	SES	05/16/2017 1441	42127

Parameter	CAS Number	Analytical Method	Result	Q	PQL	Units	Run
N-Nitrosodimethylamine	62-75-9	625	ND		1.6	ug/L	1
N-Nitrosodi-n-propylamine	621-64-7	625	ND		1.6	ug/L	1
N-Nitrosodiphenylamine (Diphenylamine)	86-30-6	625	ND		1.6	ug/L	1
Pentachlorophenol	87-86-5	625	ND		8.0	ug/L	1
Phenanthrene	85-01-8	625	ND		1.6	ug/L	1
Phenol	108-95-2	625	ND		1.6	ug/L	1
Pyrene	129-00-0	625	ND		1.6	ug/L	1
2,4,6-Trichlorophenol	88-06-2	625	ND		1.6	ug/L	1

Surrogate	Q	Run 1 % Recovery	Acceptance Limits
2-Fluorobiphenyl		57	37-129
2-Fluorophenol		36	24-127
Nitrobenzene-d5		55	38-127
Phenol-d5		43	28-128
Terphenyl-d14		42	10-148
2,4,6-Tribromophenol		59	41-144

PQL = Practical quantitation limit B = Detected in the method blank E = Quantitation of compound exceeded the calibration range H = Out of holding time
ND = Not detected at or above the PQL J = Estimated result < PQL and ≥ MDL P = The RPD between two GC columns exceeds 40% N = Recovery is out of criteria
Where applicable, all soil sample analysis are reported on a dry weight basis unless flagged with a "W"

Client: **Dorchester County Water & Sewer** | Laboratory ID: **SE12036-008**

Description: **PLANT EFFLUENT** | Matrix: **Aqueous**

Date Sampled: **05/12/2017 0800** | Project Name: **NPDES Permit Renewal**

Date Received: **05/12/2017** | Project Number:

Run	Prep Method	Analytical Method	Dilution	Analysis Date	Analyst	Prep Date	Batch
1	3520C	8270D	5	05/23/2017 1135	SES	05/16/2017 1441	42129

Parameter	CAS Number	Analytical Method	Result	Q	PQL	Units	Run
1,2-Diphenylhydrazine(as azobenzene)	103-33-3	8270D	ND		20	ug/L	1

Surrogate	Q	Run 1 % Recovery	Acceptance Limits
2-Fluorobiphenyl		60	37-129
2-Fluorophenol		51	24-127
Nitrobenzene-d5		59	38-127
Phenol-d5		52	28-128
Terphenyl-d14		35	10-148
2,4,6-Tribromophenol		52	41-144

PQL = Practical quantitation limit | B = Detected in the method blank | E = Quantitation of compound exceeded the calibration range | H = Out of holding time

ND = Not detected at or above the PQL | J = Estimated result < PQL and ≥ MDL | P = The RPD between two GC columns exceeds 40% | N = Recovery is out of criteria

Where applicable, all soil sample analysis are reported on a dry weight basis unless flagged with a "W"

Client: **Dorchester County Water & Sewer**	Laboratory ID: **SE12036-009**
Description: **PLANT EFFLUENT**	Matrix: **Aqueous**
Date Sampled: **05/12/2017 0800**	Project Name: **NPDES Permit Renewal**
Date Received: **05/12/2017**	Project Number:

Run	Prep Method	Analytical Method	Dilution	Analysis Date	Analyst	Prep Date	Batch
1		(BOD, 5 day) SM 5210B-2011	1	05/18/2017 0845	AMM1	05/13/2017 1142	11343
1		(TDS) SM 2540C-2011	1	05/15/2017 1229	MGM		41977
1		(TSS) SM 2540D-2011	1	05/16/2017 1200	MGM		42087

Parameter	CAS Number	Analytical Method	Result	Q	PQL	Units	Run
BOD, 5 day		SM 5210B-20	ND	6	4.0	mg/L	1
TDS		**SM 2540C-20**	**300**		**10**	**mg/L**	**1**
TSS		SM 2540D-20	ND		1.0	mg/L	1

Footnote(s): 6-D.O. depletion < 2.00 mg/L

PQL = Practical quantitation limit | B = Detected in the method blank | E = Quantitation of compound exceeded the calibration range | H = Out of holding time
ND = Not detected at or above the PQL | J = Estimated result < PQL and ≥ MDL | P = The RPD between two GC columns exceeds 40% | N = Recovery is out of criteria

Where applicable, all soil sample analysis are reported on a dry weight basis unless flagged with a "W"

Appendix B

Residuals Analysis

Client: **Dorchester County Water & Sewer**

Laboratory ID: **TF18010-001**

Description: **Digester Solids**

Matrix: **Solid**

Date Sampled: **06/18/2018 0815**

Project Name: **TCLP**

Date Received: **06/18/2018**

Project Number:

Run	Prep Method	Analytical Method	Dilution	Analysis Date	Analyst	Prep Date	Batch	Leachate Date
1	1311/5030B	8260B	10	06/29/2018 0058	KGT		75997	06/18/2018 1819

Parameter	CAS Number	Analytical Method	Result	Q	LOQ	Units	Run
Benzene	71-43-2	8260B	ND		0.050	mg/L	1
2-Butanone (MEK)	78-93-3	8260B	ND		0.10	mg/L	1
Carbon tetrachloride	56-23-5	8260B	ND		0.050	mg/L	1
Chlorobenzene	108-90-7	8260B	ND		0.050	mg/L	1
Chloroform	67-66-3	8260B	ND		0.050	mg/L	1
1,2-Dichloroethane	107-06-2	8260B	ND		0.050	mg/L	1
1,1-Dichloroethene	75-35-4	8260B	ND		0.050	mg/L	1
Tetrachloroethene	127-18-4	8260B	ND		0.050	mg/L	1
Trichloroethene	79-01-6	8260B	ND		0.050	mg/L	1
Vinyl chloride	75-01-4	8260B	ND		0.010	mg/L	1

Surrogate	Q	Run 1 % Recovery	Acceptance Limits
1,2-Dichloroethane-d4		104	70-130
Bromofluorobenzene		101	70-130
Toluene-d8		108	70-130

LOQ = Limit of Quantitation　B = Detected in the method blank　E = Quantitation of compound exceeded the calibration range

ND = Not detected at or above the LOQ　N = Recovery is out of criteria　P = The RPD between two GC columns exceeds 40%

H = Out of holding time　W = Reported on wet weight basis

Client: **Dorchester County Water & Sewer**	Laboratory ID: **TF18010-001**
Description: **Digester Solids**	Matrix: **Solid**
Date Sampled: **06/18/2018 0815**	Project Name: **TCLP**
Date Received: **06/18/2018**	Project Number:

Run	Prep Method	Analytical Method	Dilution	Analysis Date	Analyst	Prep Date	Batch	Leachate Date
1	1311/3520C	8270D	1	06/22/2018 1645	JCG	06/19/2018 1736	75269	06/18/2018 1819

Parameter	CAS Number	Analytical Method	Result	Q	LOQ	Units	Run
1,4-Dichlorobenzene	106-46-7	8270D	ND		0.040	mg/L	1
2,4-Dinitrotoluene	121-14-2	8270D	ND		0.080	mg/L	1
Hexachlorobenzene	118-74-1	8270D	ND		0.040	mg/L	1
Hexachlorobutadiene	87-68-3	8270D	ND		0.040	mg/L	1
Hexachloroethane	67-72-1	8270D	ND		0.040	mg/L	1
2-Methylphenol	95-48-7	8270D	ND		0.040	mg/L	1
3+4-Methylphenol	106-44-5	8270D	ND		0.040	mg/L	1
Nitrobenzene	98-95-3	8270D	ND		0.040	mg/L	1
Pentachlorophenol	87-86-5	8270D	ND		0.20	mg/L	1
Pyridine	110-86-1	8270D	ND		0.040	mg/L	1
2,4,5-Trichlorophenol	95-95-4	8270D	ND		0.040	mg/L	1
2,4,6-Trichlorophenol	88-06-2	8270D	ND		0.040	mg/L	1

Surrogate	Q	Run 1 % Recovery	Acceptance Limits
2-Fluorobiphenyl		67	37-129
2-Fluorophenol		45	24-127
Nitrobenzene-d5		70	38-127
Phenol-d5		66	28-128
Terphenyl-d14		86	10-148
2,4,6-Tribromophenol		66	41-144

LOQ = Limit of Quantitation | B = Detected in the method blank | E = Quantitation of compound exceeded the calibration range
ND = Not detected at or above the LOQ | N = Recovery is out of criteria | P = The RPD between two GC columns exceeds 40%
H = Out of holding time | W = Reported on wet weight basis

TCLP Herbicides

Client: **Dorchester County Water & Sewer**		Laboratory ID: **TF18010-001**
Description: **Digester Solids**		Matrix: **Solid**
Date Sampled: **06/18/2018 0815**	Project Name: **TCLP**	
Date Received: **06/18/2018**	Project Number:	

Run	Prep Method	Analytical Method	Dilution	Analysis Date	Analyst	Prep Date	Batch	Leachate Date
1	1311/8151A	8151A	1	06/23/2018 0643	DAL1	06/21/2018 2053	75551	06/18/2018 1819

Parameter	CAS Number	Analytical Method	Result	Q	LOQ	Units	Run
2,4-D	94-75-7	8151A	ND		0.020	mg/L	1
2,4,5-TP (Silvex)	93-72-1	8151A	ND		0.0050	mg/L	1

Surrogate	Q	Run 1 % Recovery	Acceptance Limits
DCAA		69	62-117

LOQ = Limit of Quantitation
B = Detected in the method blank
E = Quantitation of compound exceeded the calibration range
ND = Not detected at or above the LOQ
N = Recovery is out of criteria
P = The RPD between two GC columns exceeds 40%
ut of holding time
W = Reported on wet weight basis

Shealy Environmental Services, Inc.
106 Vantage Point Drive West Columbia, SC 29172 (803) 791-9700 Fax (803) 791-9111 www.shealylab.com

Client: **Dorchester County Water & Sewer** Laboratory ID: **TF18010-001**

Description: **Digester Solids** Matrix: **Solid**

Date Sampled: **06/18/2018 0815** Project Name: **TCLP**

Date Received: **06/18/2018** Project Number:

Run	Prep Method	Analytical Method	Dilution	Analysis Date	Analyst	Prep Date	Batch	Leachate Date
1	1311/3520C	8081B	1	06/22/2018 0102	PMS	06/19/2018 1736	75271	06/18/2018 1819

Parameter	CAS Number	Analytical Method	Result	Q	LOQ	Units	Run
gamma-BHC (Lindane)	58-89-9	8081B	ND		0.00040	mg/L	1
Chlordane	57-74-9	8081B	ND		0.0040	mg/L	1
Endrin	72-20-8	8081B	ND		0.00040	mg/L	1
Heptachlor	76-44-8	8081B	ND		0.00040	mg/L	1
Heptachlor epoxide	1024-57-3	8081B	ND		0.00040	mg/L	1
Methoxychlor	72-43-5	8081B	ND		0.0016	mg/L	1
Toxaphene	8001-35-2	8081B	ND		0.0080	mg/L	1

Surrogate	Q	Run 1 % Recovery	Acceptance Limits
Decachlorobiphenyl		72	20-131
Tetrachloro-m-xylene		64	26-132

LOQ = Limit of Quantitation B = Detected in the method blank E = Quantitation of compound exceeded the calibration range

ND = Not detected at or above the LOQ N = Recovery is out of criteria P = The RPD between two GC columns exceeds 40%

H = Out of holding time W = Reported on wet weight basis

Shealy Environmental Services, Inc.
106 Vantage Point Drive West Columbia, SC 29172 (803) 791-9700 Fax (803) 791-9111 www.shealylab.com

Client: **Dorchester County Water & Sewer**	Laboratory ID: **TF18010-001**
Description: **Digester Solids**	Matrix: **Solid**
Date Sampled: **06/18/2018 0815**	Project Name: **TCLP**
Date Received: **06/18/2018**	Project Number:

Run	Prep Method	Analytical Method	Dilution	Analysis Date	Analyst	Prep Date	Batch	Leachate Date
1	1311/3010A	6010D	1	06/24/2018 2013	CJZ	06/21/2018 2300	75554	06/18/2018 1819
1	1311/7470A	7470A	1	06/21/2018 1609	JCF	06/20/2018 1516	75354	06/18/2018 1819

Parameter	CAS Number	Analytical Method	Result	Q	LOQ	Units	Run
Arsenic	7440-38-2	6010D	ND		0.15	mg/L	1
Barium	7440-39-3	6010D	ND		0.25	mg/L	1
Cadmium	7440-43-9	6010D	ND		0.050	mg/L	1
Chromium	7440-47-3	6010D	ND		0.10	mg/L	1
Lead	7439-92-1	6010D	ND		0.10	mg/L	1
Mercury	7439-97-6	7470A	ND		0.0020	mg/L	1
Selenium	7782-49-2	6010D	ND		0.20	mg/L	1
Silver	7440-22-4	6010D	ND		0.10	mg/L	1

LOQ = Limit of Quantitation　B = Detected in the method blank　E = Quantitation of compound exceeded the calibration range
ND = Not detected at or above the LOQ　N = Recovery is out of criteria　P = The RPD between two GC columns exceeds 40%
H = Out of holding time　W = Reported on wet weight basis

Appendix C

SCDHEC Speculative Wasteload Allocation

Healthy People. Healthy Communities

June 14, 2018

Mr. Larry Harper, Director
Water & Sewer Department
Dorchester County
235 Deming Way
Summerville, SC 29483

RE: Dorchester County - Wasteload Allocation
Proposed NPDES Discharge for Central Dorchester WWTP
Dorchester County

Dear Mr. Harper:

A letter from WK Dickson dated February 14, 2018, on behalf of Dorchester County (County), requested wasteload allocations at the proposed flows of 2.0, 4.0, and 8.0 MGD for the following proposed outfall locations:

1. Unnamed tributary to Ashley River upstream of Cypress Swamp;
2. Unnamed tributary to Ashley River immediately southwest of the flooded sand pits and;
3. Ashley River at Highway 17 Alt. Bridge.

The proposed discharge would likely be incorporated into the Charleston area TMDL (Charleston Harbor, Cooper, Wando and Ashley Rivers). As such, the facility can expect tight speculative effluent limits and should be prepared to use a high level of treatment at the proposed facility. The County would likely need to allocate UOD loading from its downstream facility to maintain the TMDL for the Ashley River.

The wasteload information presented herein will replace or supersede all previous wasteload information provided based on new information presented by the Water Quality Modeling Section. A detailed report from the Water Quality Modeling Section and corresponding wasteload allocation worksheets are enclosed with this letter. We will summarize the proposed effluent limitations at each location and flow below.

<u>Location #1: Unnamed tributary to Ashley River upstream of Cypress Swamp</u>

This proposed location is the least supported candidate for a potential discharge and a WLA will not be issued at this time. The small nature of the channel and nearness to Cypress Swamp are problematic and would require additional coordination and studies to determine if either the channel or swamp could accept the discharge volume and what water quality and biological studies (and future monitoring) would be necessary for the swamp.

Location #2: Unnamed tributary to Ashley River immediately SW of the flooded sand pits

Based on the wasteload allocation for this proposed discharge location, the NPDES limits can be expected to be as follows:

Proposed Concentration Limits at Proposed Location #2 for all design flows			
Parameter	Monthly Average	Weekly Average	Daily Maximum
BOD_5, mg/L[1]	6.2	9.3	---
TSS, mg/L	30	45	---
NH_3-N, mg/L[1]	0.5	0.75	---
TRC, mg/L	0.011	---	0.019
DO, mg/L	6.0 (Minimum at all times)		
E.Coli, MPN/100 mL	126	---	349
pH (Standard Units)	6.0 – 8.5		
Total Nitrogen, mg/L[2]	MR	MR	---
Total Phosphorus, mg/L[2]	MR	MR	---
Total Cadmium, mg/L[3]	0.00034	---	0.0019
Total Copper, mg/L[3]	0.0097	---	0.013
Total Lead, mg/L[3]	0.0035	---	0.091
Total Zinc, mg/L[3]	0.16	---	0.16
Total Mercury, ng/L[3,4]	51	---	74
Chronic WET @ CTC=100%	25%	---	40%

1. These limits will be used to help determine the amount of UOD to be allocated from downstream facilities to maintain the TMDL for the Ashley River. UOD limits, in addition to BOD_5 and NH_3-N limits, may be requested by the permittee, which could slightly increase the BOD_5 and NH_3-N limits. See Condition 6 below.
2. See Condition 10 below.
3. Future monitoring data may demonstrate a lack of reasonable potential to cause an excursion of water quality criteria; if future data demonstrates that there is a lack of reasonable potential to cause an excursion of water quality criteria, the need for numeric limits may be re-evaluated upon request.
4. Monitoring Station CSTL-560 is impaired for Mercury downstream of the proposed discharge. Discharge will be required to meet in-stream water quality criteria for Mercury.

Location #3: Ashley River at Highway 17 Alt. Bridge

Based on the wasteload allocation for this proposed discharge location, the NPDES limits can be expected to be as follows:

Proposed Concentration Limits at Proposed Location #3 for 2.0 MGD design flow			
Parameter	Monthly Average	Weekly Average	Daily Maximum
BOD_5, mg/L[1]	5.0	7.5	---
TSS, mg/L	30	45	---
NH_3-N, mg/L[1]	0.5	0.75	---
TRC, mg/L	0.015	---	0.025
DO, mg/L	6.0 (Minimum at all times)		
E.Coli, MPN/100 mL	126	---	349

pH (Standard Units)	6.0 – 8.5		
Total Nitrogen, mg/L[2]	MR	MR	---
Total Phosphorus, mg/L[2]	MR	MR	---
Total Cadmium, mg/L[3]	0.00086	---	0.0047
Total Copper, mg/L[3]	0.019	---	0.026
Total Lead, mg/L[3]	0.0071	---	0.18
Total Zinc, mg/L[3]	0.31	---	0.31
Total Mercury, ng/L[3,4]	51	---	74
Chronic WET @ CTC=100%	25%	---	40%

1. These limits will be used to help determine the amount of UOD to be allocated from downstream facilities to maintain the TMDL for the Ashley River. UOD limits, in addition to BOD_5 and NH_3-N limits, may be requested by the permittee, which could slightly increase the BOD_5 and NH_3-N limits. See Condition 7 below.
2. See Condition 10 below.
3. Future monitoring data may demonstrate a lack of reasonable potential to cause an excursion of water quality criteria; if future data demonstrates that there is a lack of reasonable potential to cause an excursion of water quality criteria, the need for numeric limits may be re-evaluated upon request.
4. Monitoring Station CSTL-560 is impaired for Mercury downstream of the proposed discharge. Discharge will be required to meet in-stream water quality criteria for Mercury.

Proposed Concentration Limits at Proposed Location #3 for 4.0 MGD design flow			
Parameter	**Monthly Average**	**Weekly Average**	**Daily Maximum**
BOD_5, mg/L[1]	5.0	7.5	---
TSS, mg/L	30	45	---
NH_3-N, mg/L[1]	0.5	0.75	---
TRC, mg/L	0.018	---	0.032
DO, mg/L	6.0 (Minimum at all times)		
E.Coli, MPN/100 mL	126	---	349
pH (Standard Units)	6.0 – 8.5		
Total Nitrogen, mg/L[2]	MR	MR	---
Total Phosphorus, mg/L[2]	MR	MR	---
Total Cadmium, mg/L[3]	0.00060	---	0.0033
Total Copper, mg/L[3]	0.015	---	0.019
Total Lead, mg/L[3]	0.0054	---	0.14
Total Zinc, mg/L[3]	0.24	---	0.24
Total Mercury, ng/L[3,4]	51	---	74
Chronic WET @ CTC=100%	25%	---	40%

1. These limits will be used to help determine the amount of UOD to be allocated from downstream facilities to maintain the TMDL for the Ashley River. UOD limits, in addition to BOD_5 and NH_3-N, may be requested by the permittee, which could slightly increase the BOD_5 and NH_3-N concentration limits. See Condition 7 below.
2. See Condition 10 below.
3. Future monitoring data may demonstrate a lack of reasonable potential to cause an excursion of water quality criteria; if future data demonstrates that there is a lack of reasonable potential to cause an excursion of water quality criteria, the need for numeric limits may be re-evaluated upon request.
4. Monitoring Station CSTL-560 is impaired for Mercury downstream of the proposed discharge. Discharge will be required to meet in-stream water quality criteria for Mercury.

Proposed Concentration Limits at Proposed Location #3 for 8.0 MGD design flow			
Parameter	**Monthly Average**	**Weekly Average**	**Daily Maximum**
BOD_5, mg/L[1]	5.0	7.5	---
TSS, mg/L	30	45	---
NH_3-N, mg/L[1]	0.5	0.75	---
TRC, mg/L	0.026	---	0.044
DO, mg/L	6.0 (Minimum at all times)		
E.Coli, MPN/100 mL	126	---	349
pH (Standard Units)	6.0 - 8.5		
Total Nitrogen, mg/L[2]	MR	MR	---
Total Phosphorus, mg/L[2]	MR	MR	---
Total Cadmium, mg/L[3]	0.00047	---	0.0026
Total Copper, mg/L[3]	0.012	---	0.016
Total Lead, mg/L[3]	0.0044	---	0.11
Total Zinc, mg/L[3]	0.20	---	0.20
Total Mercury, ng/L[3,4]	51	---	74
Chronic WET @ CTC=100%	25%	---	40%

1. These limits will be used to help determine the amount of UOD to be allocated from downstream facilities to maintain the TMDL for the Ashley River. UOD limits, in addition to BOD_5 and NH_3-N limits, may be requested by the permittee, which could slightly increase the BOD_5 and NH_3-N limits. See Condition 7 below.
2. See Condition 10 below.
3. Future monitoring data may demonstrate a lack of reasonable potential to cause an excursion of water quality criteria; if future data demonstrates that there is a lack of reasonable potential to cause an excursion of water quality criteria, the need for numeric limits may be re-evaluated upon request.
4. Monitoring Station CSTL-560 is impaired for Mercury downstream of the proposed discharge. Discharge will be required to meet in-stream water quality criteria for Mercury.

The following conditions should be noted. The wasteload is informational purposes only until the following actions occur:

1. A determination whether the project is consistent with the applicable 208 Water Quality Plan must be made on the discharge prior to the NPDES permit decision. The Berkeley-Charleston-Dorchester Council of Governments (COG) would also need to allocate UOD loading from a downstream facility (Dorchester County/Lower Dorchester or Summerville).

2. Because this would be a new discharge into surface waters whose quality is greater than water quality standards (i.e., higher quality waters), an alternatives analysis shall be included in the engineering report. The report should also show that the proposal is necessary to important social and economic development in the area of the receiving waters such that the discharge should be allowed under the anti-degradation provisions of Regulation 61-68 (Water Quality Standards). The alternatives analysis shall demonstrate that none of the following applicable alternatives are economically and technologically reasonable:

   (a) Reuse that would minimize or eliminate the need to lower water quality;
   (b) Use of other discharge locations;

(c) Connection to other wastewater treatment facilities;
(d) Use of land application;
(e) Product or raw material substitution; and
(f) Any other treatment option or alternative, which would minimize or eliminate the need to lower water quality.

3. An NPDES permit application and preliminary engineering report (which may also help address a 208 review) is provided on the proposal. Please note that the NPDES permitting action must be completed in accordance with Regulation 61-9, before a Construction Permit could be considered for this project.

4. The selected wasteload allocation is subject to EPA Region IV review since this would be a major facility.

5. Additional metals testing and/or requirements may be necessary subject to information provided with the NPDES application and/or PER. Submission of future effluent metals data may result in specific pollutants to be added or deleted from the limits.

6. Proposed Location #2: The assimilative capacity of UOD along the unnamed tributary will lessen the amount of UOD that enters the Ashley River. The following table displays a possible range of the amount of UOD remaining at the end of the unnamed tributary as it enters the Ashley River (as simulated by the Qual2e model) for the three different design flows. This UOD would likely need to be allocated from a downstream facility (Dorchester County/Lower Dorchester or Summerville) to maintain the TMDL for the Ashley River.

Design Flow (MGD)	Modeled UOD entering the Ashley River at proposed effluent limits (lbs/day)*
2.0	197 – 319
4.0	396 – 641
8.0	791 – 1,280

* Assumes F-ratio of 2.2 – 3.8 based on measured values for existing Lower Dorchester County and Summerville facilities on the Ashley River (to be confirmed by permittee).

7. Proposed Location #3: The direct discharge into the Ashley River would likely require a pound-for-pound allocation of UOD loading from a facility on the river (Dorchester County/ Lower Dorchester or Summerville) to maintain the TMDL for the Ashely River. The following table displays a possible range of the amount of UOD discharged to the Ashley River for the three different design flows.

Design Flow (MGD)	Modeled UOD entering the Ashley River at proposed effluent limits (lbs/day)*
2.0	222 – 355
4.0	443 – 710
8.0	886 – 1,420

* Assumes F-ratio of 2.2 – 3.8 based on measured values for existing Lower Dorchester County and Summerville facilities on the Ashley River (to be confirmed by permittee).

8. The stream critical flow and Average Annual Flow (AAF) in the unnamed tributary to Ashley River immediately southwest of the flooded sand pits at the proposed discharge location are 0.0 cfs and 0.08 cfs, respectively. The stream critical flow and AAF in the Ashley River at the Highway 17 Alt. Bridge proposed discharge location are 4.1 cfs and 128 cfs, respectively. The higher critical flows in the Ashley River at the Highway 17 Alt. Bridge may provide for a larger dilution allowance for other parameters (e.g., metals) than would be available in the unnamed tributary to Ashley River immediately southwest of the flooded sand pits.

9. The applicant would need to address capacity and flow questions related to the channel for Proposed Discharge Location #2, which could not be readily answered during the site visit or from aerial imagery/GIS. The unnamed tributary comes in close contact with the flooded gravel and sand pits and also recent and ongoing residential development. Among other things, the applicant would need to:

   (a) Demonstrate that the receiving tributary flows toward the Ashley from the point of discharge and that the proposed design flows would not impact the waterbodies to the north or residential developments to the south;
   (b) Confirm that the flow continues to move east and then north toward the Ashley River and not accumulate in the low lying/swamp area east of Summers Drive. Due to the large effluent volume proposed, up to 8 MGD or 12.4 cfs, and the relatively small receiving channel, the applicant must provide additional information and assurance on the flow path and capacity questions for Proposed Discharge Location #2. The purpose is to show the effluent would flow as assumed in the model analysis and that the water would not inundate private property or cause ponding in the wetland area east of the 10-foot contour (see model report for more information); and
   (c) Confirm that the discharge quantities would not adversely impact the use of the recreational path that travels over the channel on Summers Drive.

10. DHEC is developing nutrient criteria for estuaries. The facility would be subject to the criteria (downstream waters) once implemented.

If you have any questions, please do not hesitate to call me at (803) 898-4228.

Sincerely,

Kyle Maurer

Kyle Maurer Sr, PhD, PE, Manager
Domestic Wastewater Permitting Section

cc: Christine Sanford-Coker, BEHS Lowcountry Region
Berkeley-Charleston-Dorchester COG
Wade Cantrell, Manager, Water Quality Modeling Section
William H. Young, PE, W.K. Dickson & Co., Inc

Encl: Water Quality Modeling Section Report dated May 14, 2018
Wasteload Allocation for Proposed Discharge Location #2
Wasteload Allocation for Proposed Discharge Location #3

Dorchester County Water and Sewer Department – Proposed Discharges to the Ashley River

Dorchester County, South Carolina

Matthew S. Baumann

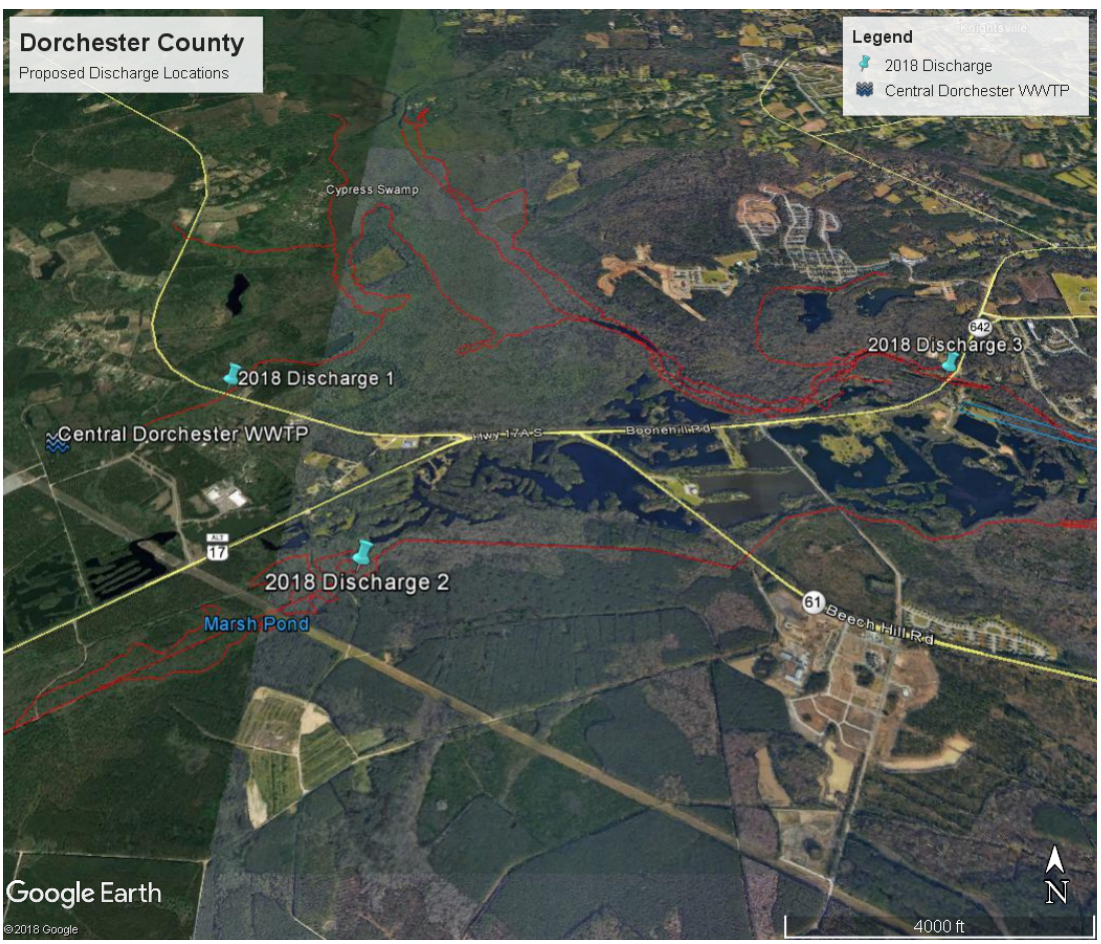
May 14, 2018

Overview

Dorchester County Water and Sewer Department (Dorchester) has requested speculative effluent limits for design flows of 2.0, 4.0 and 8.0 MGD at three proposed discharge locations in the Ashley River watershed in central and southwestern Dorchester County (Exhibit 1):

1. Unnamed tributary to Ashley River upstream of Cypress Swamp,
2. Unnamed tributary to Ashley River immediately southwest of the flooded sand pits and
3. Ashley River at Highway 17 Alt. Bridge.

Exhibit 1. Proposed discharge locations and site for a new wastewater treatment plant. The red outlines depict the stream layer shapefile for the area. The blue shape near discharge three is the upper cell of the EFDC grid for the Ashely River in the Ashley, Cooper and Wando (Charleston Harbor) TMDL .



Each of the proposed discharge locations are evaluated using methods appropriate for its geographic character. The current request follows a 2017 request by Dorchester in which 2.0 and 4.0 MGD design flows were evaluated for an unnamed tributary to Fishburne Creek (and then to Rantowles Creek/Stono River; refer to technical memorandum dated May 9, 2017 and associated WLAs/email correspondence).

Site Visit and Geographic Description

A site visit was conducted by DHEC and engineers from W.K. Dickson to each of the three proposed discharge locations on April 3, 2018.

Proposed discharge 1 – Unnamed Tributary to the Ashley River upstream of Cypress Swamp

The proposed discharge location is adjacent to Highway 61 in Ridgeville, SC. The site is a small creek with little upstream flow adjacent to private property (Exhibits 2 and 3). The width of the stream was estimated at 6-8 feet with a water depth of a few inches. Water movement through the channel was sluggish. The waterbody is effectively swamp/wetland approximately 50 feet upstream of the Highway 61 bridge (Exhibit 2).

The creek enters Cypress Swamp 0.75 miles downstream of the Highway 61 bridge. There, water must travel an estimated 1.5 miles of swamp habitat before joining the Ashley River on the eastern end of the wetland. The Ashley River leaves Cypress Swamp approximately 2.5 miles upstream of discharge location 3 near Highway 17 Alt. overpass.

Exhibit 2. View upstream of proposed discharge 1. Vegetation in the channel indicates the presence of low flow/wetland type character.

Exhibit 3. Downstream view of proposed discharge location 1.

Proposed discharge 2 – Unnamed Tributary to the Ashley River southwest of flooded sand pits

The actual site identified in Exhibit 1 was not visited because access to the WestRock-owned land was not available. The site was indirectly evaluated at two downstream locations east of the site: Highway 61 and Summers Drive. The channel is well-defined and straight at the Highway 61 access point (Exhibit 4). The width of the channel was 20 feet with water depths of 1-1.3 feet (0.3-0.4 meters) at the time of visit. The Summers Drive access point 0.4 miles east of the Highway 61 indicated a fairly similar well-defined channel upstream (Exhibit 5). Upstream width and depth are approximately the same as the Highway 61 access point. However, the creek develops more natural in appearance (perhaps swamp-like) character downstream. It is possible that flow slows or ceases downstream of Summers Drive (Exhibit 6).

Exhibit 4. View upstream of proposed discharge location 2 from the Highway 61 access point.

Exhibit 5. View upstream of proposed discharge location 2 from Summers Drive access point

Exhibit 6. View downstream of proposed discharge location 2 from the Summers Drive access point.

Proposed discharge 3 – Ashley River at Highway 17 Alt. Bridge

The Ashley River at the Highway 17 Alt. bridge (Slands Bridge) is a broad section of the river with an approximate width of 70-90 feet (Exhibit 7). USGS flow data at the site indicates that it is tidally influenced to some degree. This area of the Ashley River previously received wastewater from a small (0.03 MGD) discharger (CWS Teal on the Ashley, SC0030350) cancelled in 2006. The facility was permitted a UOD loading of 3.4 pounds per day as determined by an allowable DO deficit of 0.1 mg/L using DHEC's Simplified Math Model.

Exhibit 7. Downstream view of the Ashley River at the Highway 17 Alt. bridge.

Analysis of the discharge locations

Overview

Dorchester is aware that the proposed discharger would likely be incorporated into the Charleston area TMDL (Charleston Harbor, Cooper, Wando and Ashley rivers). As such, the facility expects tight speculative effluent limits and is prepared to use a high level of treatment (e.g., membrane filtration) at the proposed facility. The applicant understands that they would need to allocate UOD loading from their downstream facility to maintain the TMDL for the Ashley River.

1. *Proposed discharge 1 – Unnamed Tributary to the Ashley River upstream of Cypress Swamp*

Proposed discharge location 1 was selected by the applicant due to its nearness to the anticipated location of the wastewater treatment facility (Exhibit 1). The discharge site off Highway 61 is approximately 0.6 miles from the expected location of the plant. The stream is small with little flow (7Q10 of zero cfs). It is not clear if the stream could accept a discharge water quantity of between 2.0 and 8.0 MGD. In addition, the close proximity and necessary traverse through Cypress Swamp complicates the site. As indicated on the topographic map for this area, there is no defined channel once the stream enters the swamp which precludes typical DO modeling approaches and raises concerns about the impact of the large volume of effluent on the natural hydrology in the swamp.

The applicant would need to demonstrate that both the stream and Cypress Swamp could accept an additional flow of up to 8.0 MGD from a discharge pipe at Highway 61 and that the level of discharge would not be a threat to the homeowner adjacent to location. Dorchester would need to coordinate directly with the DHEC's wastewater permitting division (J. deBessonet and K. Mauer) if they choose to explore a wetland dispersal technique similar to Great Swamp in the Beaufort/Jasper area. A DO model is not recommended for site 1 at this time because the questions related to the environmental impact to the channel and Cypress Swamp are associated with the quantity of water and other effluent parameters (e.g., pharmaceuticals, prescription drugs and other emerging contaminants/chemicals) rather than the UOD treatment level of the effluent.

Summary and Recommendation: Proposed location 1 is the least supported candidate for a potential discharge and a WLA is not issued at this time. The small nature of the channel and nearness to Cypress Swamp are problematic. If the applicant wishes to pursue either a direct discharge at Hwy 61 or a dispersal technique to the swamp, they would need to coordinate directly with permitting division at DHEC to determine if either the channel or swamp could accept the discharge volume and what water quality and biological baseline studies (and future monitoring) would be necessary for the swamp.

2. *Proposed discharge 2 – Unnamed Tributary to the Ashley River southwest of flooded sand pits*

Proposed discharge location 2 occurs on a highly structured and channelized tributary to the Ashley River on land owned by WestRock immediately southwest of flooded former sand and gravel pits. From the proposed location, there is approximately 3.1 miles of channel before the tributary merges with the Ashley River. The discharge would need to be included in the Ashley River TMDL; however, given the nature of the stream a DO model was constructed to investigate the assimilative capacity of the waterway before draining to the Ashley River.

The tributary comes in close contact with the flooded gravel and sand pits and also recent and ongoing residential development. The applicant would need to demonstrate that the receiving tributary flows toward the Ashley at the discharge point and would not impact the waterbodies to the north or residential developments to the south. Further, the stream moves past the 10 ft. contour (that of Ashley River locally) 0.3 miles east of Summers Drive. Dorchester would need to confirm that the flow continues to move east and then north toward the Ashley River (which was assumed in the DO model) and not accumulate in the low lying area east of Summers Drive. Lastly, there is a recreational path that travels over the stream on Summers Drive. The applicant would need to confirm that the discharge quantities would not adversely impact its use.

QUAL2E Model

A QUAL2E model was developed for the 3.1 mile stretch of the unnamed Ashley River tributary downstream of proposed discharge location 2. The purpose of the model is to gain an understanding of the assimilative capacity of the tributary prior to merging with the Ashley River based on a reasonable range of restrictive effluent limits.

A 7Q10 flow and Unit 7Q10 of zero were assumed for this model and; therefore, the only flow in the channel under 7Q10 conditions is from the discharge. Each design flow was analyzed under four effluent scenarios:

1. Reuse BOD_U (7.5 mg/L; F-ratio 1.5* 5.0 mg/L BOD_5) and ammonia nitrogen (0.5 mg/L) effluent limits with an aerated discharge of 7.0 mg/L DO,

2. Statewide background BOD_U (2.0 mg/L) and ammonia nitrogen (0.11 mg/L) effluent limits with a standard discharge DO of 6.0 mg/L,

3. Enhanced effluent DO (7.0 mg/L) , reuse ammonia nitrogen (0.5 mg/L) and adjusted BOD_U limits such that the minimum in-stream DO standard of 5.0 mg/L is met and

4. Standard discharge DO (6.0 mg/L) , reuse ammonia nitrogen (0.5 mg/L) and adjusted BOD_U limits such that the minimum in-stream DO standard of 5.0 mg/L is met.

The effluent DO concentrations are higher than background in-stream values during the March through October critical period for the area. [The critical period average daily in-stream DO concentration at the Ashley River USGS gage (02172080) was 3.52 mg/L from 2001 through 2005.]. The default WMU 0303 critical temperature of 29°C (84.2°F) was used for stream reaches and the discharge.

Because total in-stream flows from the discharge are large relative to the model drainage area (total DA = 4.7 sq. miles), the Borders (1983) approach to estimate velocity was not appropriate. Instead the Jobson (1996) method was used and returned approximately the same velocity for three design flows (0.31 fps). The result is reasonable as the model area is extremely flat and water movement would be sluggish regardless of flow. The highly-structured and channelized portion of the tributary has a width of around 20 feet. Assuming a rectangular configuration the depth of the channel would be approximately 0.5 – 2.0 feet for the three design flows should the velocity be constant (0.5 feet for 2.0 MGD, 1.0 feet for 4.0 MGD and 2.0 feet for 8.0 MGD). The applicant may need to determine if this level of water could pose a threat to the local ponds or residential development/recreational path.

For each of the four model scenarios, the results are relatively similar across the requested design flows. In general, the channel assimilates 25-30% of discharged BOD_U and ammonia by the end of the model/convergence with the Ashley (Exhibit 8).

Under reuse limits and enhanced effluent DO (Scenario 1 above), assimilation of BOD_U and ammonia consumes approximately 2.3 mg/L of oxygen (Exhibit 8). Under relatively more restrictive limits (statewide background concentrations, Scenario 2), in-stream DO indicates a slight enhancement from the discharge concentration of 6.0 mg/L. This observation may be explained by an underestimated F-ratio in the model (1.5) for what would like be a relatively refractory pool of discharged organic matter from the treatment plant. However, the stream would only assimilate approximately 0.5 mg/L and 0.03 mg/L of the discharged BOD_U and ammonia, respectively, under these conditions (Exhibit 8).

Scenarios 3 and 4 provide information related to what discharge limits could be available to Dorchester to 1) satisfy the state DO standard of 5.0 mg/L minimum concentration and 2) estimate how much TMDL allocation would need to be acquired based on BOD and ammonia levels entering the Ashley. The two scenarios highlight tradeoffs between level of treatment and the amount of potentially necessary TMDL allocation. Under scenario 3 conditions (enhanced effluent DO), less restrictive treatment limits for BOD are possible (Exhibit 8), but these limits result in higher end of model in-stream BOD concentrations and, thus, a greater required TMDL allocation. Conversely, under scenario 4 conditions (standard effluent DO), more restrictive discharge limits for BOD are necessary to meet the state DO standard (Exhibit 8), but would result in lower end of model BOD concentrations.

The end of model BOD and ammonia concentrations potentially represent a starting point for determining the allocation quantity necessary to satisfy the TMDL. Several questions need to be addressed:

1. Should local background BOD_5 and ammonia concentrations be subtracted from the end of model values to yield net concentrations and, if so, what are the background concentrations? Are the CSTL-102 averages presented above reasonable?
    a. In situations where there is background flow in the QUAL2E model (i.e., 7Q10>0) the background load is typically subtracted from the model permit which yields the net load delivered to the downstream water. Here, with zero 7Q10, there is no background load in the QUAL2E model, so in this case the total model load in the permit run should be used.

2. What F-ratio should be applied to the end of model BOD_5 concentrations?

a. The F-ratio of 2.2 or 3.8 measured for Dorchester's existing plant or Summerville's plant, respectively, could be used to roughly estimate the UOD delivered to the Ashley. The value would eventually be confirmed by the applicant to determine more accurately UOD loading. For the purpose of determining the impact on the river and TMDL, it is reasonable to assume the existing Dorchester facility or Summerville F-ratios to estimate a possible range of UOD loading to the Ashley River. The default F-ratio 1.5 used in the QUAL2E simulation is paired in that model with the default CBOD decay rate 0.3/day. This is standard QUAL2E modeling procedure per the WLA EPA Agreement. This approach is used to evaluate the impact of the discharge in the tributary. The EFDC model for the Ashley River uses measured F-ratios (higher than default) and decay rates (lower than default).

Exhibit 8. Summary results of the DO model scenarios described above. Scenarios 1 and 2 use predefined BOD_U and ammonia limits. In scenarios 3 and 4, ammonia was held constant at 0.5 mg/L and BOD_U was adjusted to return the state standard minimum in-stream DO of 5.0 mg/L (right-hand column). The asterisk (*) denotes predefined values consistent with the respective model scenario. An F-ratio of 1.5 is used in all model runs.

Model Scenario		Model Results			
		Min. In-stream DO (mg/L)	BOD_U at Ashley (mg/L)	Ammonia at Ashley (mg/L)	BOD_U Effluent Limit (mg/L)
2.0 MGD Design Flow					
1	•Reuse effluent limits •Enhanced effluent DO of 7.0 mg/L	5.708	5.546	0.385	7.5*
2	•Background effluent limits •Standard effluent DO of 6.0 mg/L	6.013	1.522	0.085	2.0*
3	•Min. in-stream DO of 5.0 mg/L •Enhanced effluent DO of 7.0 mg/L •Ammonia discharge limit of 0.5 mg/L	5.0*	8.725	0.386	11.8
4	•Min. in-stream DO of 5.0 mg/L •Standard effluent DO of 6.0 mg/L •Ammonia discharge limit of 0.5 mg/L	5.0*	6.854	0.387	9.27
4.0 MGD Design Flow					
1	•Reuse effluent limits •Enhanced effluent DO of 7.0 mg/L	5.709	5.546	0.385	7.5*
2	•Background effluent limits •Standard effluent DO of 6.0 mg/L	6.012	1.522	0.085	2.0*
3	•Min. in-stream DO of 5.0 mg/L •Enhanced effluent DO of 7.0 mg/L •Ammonia discharge limit of 0.5 mg/L	5.0*	8.725	0.386	11.8
4	•Min. in-stream DO of 5.0 mg/L •Standard effluent DO of 6.0 mg/L •Ammonia discharge limit of 0.5 mg/L	5.0*	6.891	0.387	9.32
8.0 MGD Design Flow					
1	•Reuse effluent limits •Enhanced effluent DO of 7.0 mg/L	5.709	5.545	0.385	7.5*
2	•Background effluent limits •Standard effluent DO of 6.0 mg/L	6.011	1.523	0.085	2.0*
3	•Min. in-stream DO of 5.0 mg/L •Enhanced effluent DO of 7.0 mg/L •Ammonia discharge limit of 0.5 mg/L	5.0*	8.722	0.386	11.8
4	•Min. in-stream DO of 5.0 mg/L •Standard effluent DO of 6.0 mg/L •Ammonia discharge limit of 0.5 mg/L	5.0*	6.875	0.387	9.30

<u>Summary and Recommendation:</u> The DO modeling indicates that the channel could assimilate 25-30% of BOD and ammonia based on the selected effluent limits and a relatively high concentration of DO (6.0 mg/L). The results of scenario 4 are used as a starting point to estimate the amount of loading that the applicant could need to acquire on a pound-for-pound basis from a downstream facility to maintain the TMDL (Exhibit 9). The applicant would also need to confirm 1) the channel could support the requested discharge volumes (e.g., 0.5 to 2.0 feet of continuous water depth as estimated above), 2) it would not disrupt the surrounding area and 3) the stream east of Summers Drive drains to the Ashley River under all flow conditions.

Exhibit 9. End of model UOD loadings to the Ashley River based on effluent limits of 6.2 mg/L (BOD_5) and 0.5 mg/L (ammonia) for proposed location 2. These values represent the speculative effluent limits for this discharge location and the allocation that could need to be acquired from a downstream facility on the Ashley River. The limits are based on achieving a minimum in-stream DO concentration of 5.0 mg/L with an effluent DO concentration of 6.0 mg/L. Background concentrations for these parameters are not subtracted because 7Q10 flow in this channel is assumed to be zero.

Design Flow	Effluent BOD_5 (mg/L)	End of Model BOD_5 (mg/L)	F-ratio	Effluent Ammonia (mg/L)	End of Model Ammonia (mg/L)	UOD (lbs/day)
2.0 MGD	6.18	4.57	2.2 - 3.8	0.5	0.387	197 - 319
4.0 MGD	6.21	4.59		0.5	0.387	396 - 641
8.0 MGD	6.20	4.58		0.5	0.387	791 - 1280

3. *Proposed discharge 3 – Ashley River at Highway 17 Alt. Bridge*

a. Simplified Math Model

Prior to the site visit, the discharge to the Ashely River at the Highway 17 Alt. bridge was evaluated using the Simplified Math Model (SMM). Model input data were obtained from USGS gage 02172080 located at the Ashley River adjacent to the proposed discharge location. The gage recorded flow data from October 2001 through September 2003. The period includes severe drought conditions of 2001 and 2002. The minimum average 7-day flow corrected for tidal forcing was 4.09 cfs for the drought period. The period of record for other hydrographic parameters (temperature, gage height, specific conductance and DO) extends from approximately August 2001 through April 2005.

Because of the uncertainties and limitations of the SMM, sensitivity of the model output (maximum DO deficit in mg/L and downstream DO sag point in miles) was analyzed for each design flow with 'reuse' limits for UOD concentration (9.8 mg/L; BOD_5 of 5.0 mg/L [F-ratio of 1.5] and ammonia of 0.5 mg/L [4.57 scalar]). Specifically, DO output sensitivity was evaluated for the following inputs and associated ranges:

- Non-conservative decay rate: 0.2 – 0.5 1/day,
- Net freshwater flow: 4.09 – 10 cfs,
- Dispersion coefficient: 0.7-1.3 sq. mi./day,
- Average depth and low tide: 2 – 6 feet and
- Tidal range: 0 – 2.0 feet.

In each case, the reaeration coefficient was recalculated based on selected inputs that affect flow velocity. Regardless of the design flow and combination of inputs, the expected DO deficit exceeds the allowable concentration of 0.1 mg/L at reuse limits per State Water Quality Standards (R. 61-68):

- 0.57 to 1.03 mg/L DO deficit for the 2.0 MGD design flow,
- 0.88 to 1.75 mg/L DO deficit for the 4.0 MGD design flow and
- 1.15 to 2.61 mg/L DO deficit for the 8.0 MGD design flow.

A limitation of the SMM is that the approach does not consider absolute concentrations of DO or expected impact on in-stream DO should the effluent have a higher concentration than the receiving waterbody. The upper Ashley River is naturally low in DO with summertime average concentrations around 3.0 mg/L.

It is possible that the facility could be permitted at their requested design flows without acquiring allocation from their downstream facility if effluent meets background BOD_5 and ammonia concentrations. DHEC monitoring station CSTL-102 is located approximately 3.5 miles downstream of the proposed discharge location and upstream of active dischargers. The station has a period of record beginning in 1999. Since then, the average BOD_5 concentration at the station is 2.65±0.59 mg/L (± 1σ, n=34) and the ammonia concentration is 0.145±0.085 mg/L (± 1σ, n=102). Nominal effluent limits considered equivalent to background levels would be 3.0 mg/L BOD5 and 0.20 mg/L ammonia.

b. Charleston Harbor TMDL EFDC Model

An attempt was made to use the Charleston Harbor TMDL EFDC model to better understand the proposed discharger's potential impact on the Ashley River. Both discharge locations (location 1 and the receiving tributary of discharge 2) occur in the most upstream grid cell in the EFDC model for the Ashley River. Historically, the focus of the model has been on the lower reaches of the Ashley, Copper River and the Harbor. Therefore, the model in the vicinity of the proposed discharger is not considered calibrated, and accuracy has not been demonstrated, in this area near the upper boundary. Because water does not exchange with the 'river' outside of the model (upstream of the boundary), adding a discharger of up to 8.0 MGD at the model boundary in a tidal area may complicate the usefulness of the results. In addition, the upstream area of the Ashley River is naturally low in DO and high in refractory organic carbon relative to the anticipated effluent limits of the facility. These natural features could further complicate the interpretation of the local DO impact.

Despite the concerns, several full year model runs were conducted to estimate the concentration of BOD_5 (assumed F-ratio of 3.8 with a 50/50 labile and refractory carbon split based on measured data for the existing Summerville discharge) that could theoretically be discharged at 8.0 MGD such that DO depression by the facility alone would not exceed 0.1 mg/L. Effluent ammonia and DO for the discharger were set to background levels for these runs. The model runs are considered 'no-load' in that they include the discharge volumes of, in particular, the two large downstream facilities (Summerville and Dorchester County), but with their UOD and DO loadings set to respective background concentrations (time-varying ambient concentrations at the discharge locations as predicted by the model). The simulations indicated that the proposed facility could discharge between 10-15 mg/L of BOD_5 while maintaining the DO standard in absence of the other facilities' UOD loadings in the absence of UOD loading from the downstream facilities.

Two TMDL runs (i.e., permit run with all facilities and their respective loadings) were conducted with BOD_5 concentrations of 10 mg/L and 13.5 mg/L, respectively, background concentrations of ammonia and an effluent DO of 6.0 mg/L for the proposed facility. Based on the 90th percentile impact (DO deficit or ΔDO), the two runs produce a similar downstream trend in the deficit relative to the existing TMDL run in the Ashely River. Relative to the existing TMDL, the 10 mg/L BOD5 run shows a slight decrease in the 90th percentile DO deficit in the upper Ashley, while the 13.5 mg/L BOD5 run demonstrates an increase in the deficit in the same region. As noted previously, the upstream Ashley River, particularly at the model boundary where the proposed discharger is located, is naturally high in background organic carbon relative to the loading for the proposed facility. The high background organic carbon concentrations entering the model challenge the interpretation and inference of the relative impact that the proposed facility could have on in-stream DO concentrations.

To further investigate model behavior related to the background condition, the no-load run was unchanged but the load (permit) run was modified to include the sum of the background carbon load plus the calculated load from the discharge (based on 10 mg/L BOD5 and Summerville effluent characteristics). Two runs, with and without effluent DO load added to the background showed large DO deficits (0.6 mg/L or more). Both the original runs described above and the follow-up runs described here appear to suffer from simplifying assumptions regarding the high background carbon in the river and the introduction of new loading from the proposed discharge, with opposite results (e.g., seemingly plenty of available assimilative capacity in the original runs versus essentially no available assimilative capacity in the follow-up runs).

Summary and Recommendation: The SMM and EFDC models were used in an attempt to interpret the facility's expected impact on in-stream DO locally. Both models have limitations in application in this area of the Ashely River. Without sufficient understanding of the impact that the facility would have on DO concentrations here due to inconclusive results from the available models, reuse level effluent limits are recommended, or 5 mg/L BOD5, 0.5 mg/L ammonia and 6 mg/L DO, and offsetting pound-for-pound UOD load reduction from existing facilities is needed due to the TMDL. At these limits, the resulting discharge is considered effectively background conditions as indicated by the EFDC model and data at Bacon Bridge. Exhibit 10 below presents a range of daily UOD loads based on reuse level effluent limits and the requested discharge volumes assuming the F-ratios of Dorchester County and Summerville facilities on the Ashley River. The final UOD reallocation requirement would depend on measured LTBOD and computed F-ratio for the new facility as provided by the applicant once the plant is operational.

Exhibit 10. Speculative effluent limits and equivalent UOD loading in lbs/day. The F-ratios used to estimate UOD loading are the measured values from the Dorchester County and Summerville facilities on the Ashley River.

Design Flow	Effluent BOD_5 (mg/L)	F-ratio	Effluent Ammonia (mg/L)	UOD (lbs/day)
2.0 MGD	5.0	2.2 - 3.8	0.5	222 - 355
4.0 MGD	5.0		0.5	443 - 710
8.0 MGD	5.0		0.5	886 - 1420

S.C. DEPARTMENT OF HEALTH AND ENVIRONMENTAL CONTROL
BUREAU OF WATER
DIVISION OF WATER QUALITY
WATER QUALITY MODELING SECTION

WASTELOAD ALLOCATION WORKSHEET AND COORDINATION FORM

Date: May 14, 2018 Engineer: K. Mauer WLA Type: Proposal
Discharger: Site 2 - Cent. Dorch. County - 2, 4 and 8 MGD NPDES: SC00proposed
County: Dorchester Couty WMU: 0303 HUC: 03050201-06-03
Receiving waters: Unnamed trib. to Ashley River On 303(d) list? yes

I. Water Quality Modeling Section

A. Model Data:

Model used: QUAL2E

Name: DC220.IN, DC240.IN and DC280.IN

USGS station / site: ---

Unit 7Q10 (cfs/mi^2): 0.0

Stream critical flow (cfs): 0.0 Critical flow type: 7Q10

Avg. annual flow (cfs): 0.08

Drainage area (mi^2): 0.13

Stream Q: waste Q ratio: 0:1

Temp critical (F/C): 84.2 / 29

Temp seasonal (F/C): 60.8 / 16

Velocity (ft/s): 0.31

Slope (ft/mi): 0 - 2.28

K1 (d^{-1}): 0.3 - 0.4

K2 (d^{-1}): 2.0 and 4.0 MGD: 1.0 - 1.3, 8.0 MGD: 0.9 - 1.0

K3 (d^{-1}): 0.3

F ratio: 1.5

Stream characteristics: See attched summary of site visit. AAQ estimated using eight inches of runoff per year (Borders, 1980).

B. Model Input Sources

Waters in question? yes Literature: DHEC/EPA Agreement
Similar waters: ---
Field data available? good
Describe field data: ---

C. Model Validity:

Intensive survey? no

Calibrated? no

Verified? no

Analyst's assessment of simulation: good

Comments: ---

D. Model Outputs:

Ammonia model: chronic toxicity (mg/l): 2.0, 4.0 and 8.0 MGD: Crit: 1.72,

Seasonal: 3.97

Dissolved oxygen model:

Critical BOD5 (mg/l): 6.2

Critical NH3-N (mg/l): 0.5

Effluent DO (mg/l): 6.0

Equivalent UOD: ---

UOD formula: ---

Predicted minimum instream DO (mg/l): 5.0

Effluent flow (mgd/cfs): 2.0 / 3.1, 4.0 / 6.2, 8.0 / 12.4

Other parameters: MR TP, MR TN*

Comments: There is ~3.1 miles of channel from the proposed discharge location to its discharge to the Ashley River. The DO model indicates that the channel could assimilate ~1/3 of the discharged BOD5 and ammonia based on restrictive effluent limits of 6.2 mg/L BOD5 (F-ratio of 1.5) and 0.5 mg/L ammonia (reuse). The facility would need to acquire UOD allocation on a 1:1 TMDL ratio from a downstream facility for all loading that is discharged to the Ashley at the end of the model. Because 7Q10 flow in the stream is zero, background BOD5 and ammonia are not subtracted from the end of model results. The applicant would also need to determine that the channel could accept and contain the anticipated flow and that the water would move toward the Ashley east of Summers Drive. Please see the summary for additional information related to these concerns.

*DHEC is developing nutrient criteria for the Ashley River. The facility will be subject to the criteria once implemented.

E. Have studies been conducted or is information available which would have an influence on the level of wastewater treatment needed? no If yes, attach comments.

F. Stream Classification: FW

G. Could the discharge be considered a wetland discharge? no If yes, attach comments from WQ Certification and Wetland Programs Section as needed.

H. Will the proposed discharge and recommended limits protect the existing uses of the waterbody? yes If no, attach a detailed explanation.

I. Is there evidence that the practical use of the stream is different from its classified use and may warrant alternate consideration? no If yes, attach comments.

J. Is there reason to believe that questionable benefits will result from requiring model recommendations? no If yes, attach comments.

Analyst: Matthew S. Baumann Date: May 14, 2018

Reviewer: [signature] Date: 6/4/18

II. Engineering Section

A. Do the model outputs exceed established technological limits for this type of wastewater? Yes - No If yes, explain below in the space provided.

B. Are there factors which make the model outputs inconsistent with best engineering judgment and/or federal effluent guidelines? Yes - No. If yes, explain below in the space provided.

C. Are there other factors which would make the WLA either more stringent or less stringent? Yes - No. If yes, explain below in the space provided.

D. Are there factors that make the water quality model outputs impractical or un-implementable at this time? Yes - No. If yes, explain below in the space provided.

E. Recommended limits

Flow: 2/4/8 MGD

BOD5 critical: 6.2 mg/L BOD5 seasonal: —

NH3-N critical: 0.5 mg/L NH3-N seasonal: —

UOD critical: — UOD seasonal: —

Effluent DO: 6.0 min

Phosphorus: MR

Other parameters: TN = MR, Hg limits due to impairment, various metals

Engineering comments: Nutrient criteria (future) may lead to TP, TN limits

F. Is there agreement with water quality model outputs? Yes No

Engineer: [signature]

Date: 6/13/18

III. Water Quality Modeling Section

Is full agreement concluded? Yes - No

If full agreement is not reached, see the wasteload allocation procedures for further steps.

If yes, the wasteload allocation is:

Flow: 2/4/8 MGD

BOD5 critical: 6.2 mg/L BOD5 seasonal: –

NH3-N critical: 0.5 mg/L NH3-N seasonal: –

UOD critical: – UOD seasonal: –

Critical limits apply: – through –

Seasonal limits apply: – through –

Effluent DO: 6.0 mg/L (minimum)

Phosphorus: MR

Other parameters: TN = MR, Hg limits as recommended

Comments: Future nutrient criteria may lead to TP and TN limits.

Approval: ______________________ Date: 6/13/18

SCDHEC Ammonia Toxicity Calculation

Based on 1999 EPA Water Quality Criteria for Ammonia as adopted by S.C. DHEC R.61-68 promulgated December 14, 2000, effective June 22, 2001.

Division of Water Quality

April 23, 2001, updated 10/05

Discharger Name:	Proposed Central Dorchester WWTP - 8.0 MGD
Permit Number:	Proposed
Receiving Stream:	Proposed Site No. 2 - Unnamed Trib. To the Ashley River
Date:	May 14, 2018
Analyst:	Matthew S. Baumann

Input Data

Upstream Flow (cfs):	0
Upstream Total Ammonia Concentration (mg N/L):	0
Critical Stream Temperature (deg. C):	29
Seasonal Stream Temperature (deg. C):	16
Stream pH:	7.5
Discharge Flow (mgd):	8
Are Salmonids Present? (yes/no):	no
Are Fish ELS Present? (yes/no):	yes

Instream Total Ammonia Toxicity Results

Season:	*Critical*	*Seasonal*
Criterion Maximum Concentration, CMC (mg N/L):	19.890	19.890
Criterion Continuous Concentration, CCC (mg N/L):	1.716	3.966

Discharge Total Ammonia Results

Season:	*Critical*	*Seasonal*
Max. Conc. Protecting Against Acute Toxicity (mg N/L):	19.89	19.89
Max. Conc. Protecting Against Chronic Toxicity (mg N/L):	1.72	3.97

Comments

Discharge total ammonia results are the same for the proposed 2.0 MGD and 4.0 MGD design flows.

S.C. DEPARTMENT OF HEALTH AND ENVIRONMENTAL CONTROL
BUREAU OF WATER
DIVISION OF WATER QUALITY
WATER QUALITY MODELING SECTION

WASTELOAD ALLOCATION WORKSHEET AND COORDINATION FORM

Date: May 14, 2018 Engineer: K. Mauer WLA Type: Proposal
Discharger: Site 3 - Cent. Dorch. County - 2, 4 and 8 MGD NPDES: SC00proposed
County: Dorchester Couty WMU: 0303 HUC: 03050201-06-02
Receiving waters: Ashley River at Hwy 17 Alt. - Slands Bridge On 303(d) list? yes

I. Water Quality Modeling Section

A. Model Data:

Model used: ---

Name: ---

USGS station / site: 02172080

Unit 7Q10 (cfs/mi^2): 0.0

Stream critical flow (cfs): 4.1 Critical flow type: Other

Avg. annual flow (cfs): 128

Drainage area (mi^2): 217.3

Stream Q: waste Q ratio: ---

Temp critical (F/C): ---

Temp seasonal (F/C): ---

Velocity (ft/s): ---

Slope (ft/mi): ---

K1 (d^{-1}): ---

K2 (d^{-1}): ---

K3 (d^{-1}): ---

F ratio: ---

Stream characteristics: See attched summary of site visit. Crtical flow calculated from tidal filtered data from USGS gage 02172080 (October 2001 through September 2003). Average annual flow estimated using 8 inches of runoff per year (Borders, 1980) and an upstream drainage area of 217.3 sq miles (HUC12s: 030502010501 through 030502010506).

B. Model Input Sources

Waters in question? yes

Literature: DHEC/EPA Agreement

Similar waters: ---

Field data available? good

Describe field data: ---

C. Model Validity:

Intensive survey? no

Calibrated? no

Verified? no

Analyst's assessment of simulation: good

Comments: ---

D. Model Outputs:

Ammonia model: chronic toxicity (mg/l): ---

Dissolved oxygen model:

Critical BOD5 (mg/l): 5.0

Critical NH3-N (mg/l): 0.5

Effluent DO (mg/l): 6.0

Equivalent UOD: ---

UOD formula: ---

Predicted minimum instream DO (mg/l): ---

Effluent flow (mgd/cfs): 2.0 / 3.1, 4.0 / 6.2, 8.0 / 12.4

Other parameters: MR TP, MR TN*

Comments: An attempt was made to evaluate the discharge location using both the Simplified Math Model (SMM) and the Charleston Harbor TMDL EFDC model (see attached summary). A limitation of the SMM is that the approach does not consider absolute concentrations of DO or expected impact on in-stream DO should the effluent have a higher concentration than the receiving waterbody. The upper Ashley River is naturally low in DO with summertime average concentrations around 3.0 mg/L. The proposed discharge location occurs in the upstream most cell of the Ashely River EFDC grid (boundary). Several full year EFDC model runs were conducted to evaluate the efficacy and reliability of the modeled DO impact in this area. There is not enough confidence in the DO results to use the EFDC model at this time because the proposed discharge location occurs at the model boundary and this area of the Ashley River is naturally high in refractory organic carbon and low in DO.

Without sufficient understanding of the impact the facility would have on Ashley River DO based on reliable modeling, reuse level effluent limits are recommended for proposed discharge in this location. All UOD loading by the facility would have to be acquired on a 1:1 TMDL allocation ratio from a downstream facility. Monitoring station data indicates that the area is naturally high in organic carbon and effluent treated to reuse level would approximate background level conditions in the upper reaches of the Ashley River.

*DHEC is developing nutrient criteria for the Ashley River. The facility will be subject to the criteria once implemented.

E. Have studies been conducted or is information available which would have an influence on the level of wastewater treatment needed? no If yes, attach comments.

F. Stream Classification: FW

G. Could the discharge be considered a wetland discharge? no If yes, attach comments from WQ Certification and Wetland Programs Section as needed.

H. Will the proposed discharge and recommended limits protect the existing uses of the waterbody? yes If no, attach a detailed explanation.

I. Is there evidence that the practical use of the stream is different from its classified use and may warrant alternate consideration? no If yes, attach comments.

J. Is there reason to believe that questionable benefits will result from requiring model recommendations? no If yes, attach comments.

Analyst: Matthew S. Baumann Date: May 14, 2018

Reviewer: [signature] Date: 6/4/18

II. Engineering Section

A. Do the model outputs exceed established technological limits for this type of wastewater? Yes No If yes, explain below in the space provided.

B. Are there factors which make the model outputs inconsistent with best engineering judgment and/or federal effluent guidelines? Yes No If yes, explain below in the space provided.

C. Are there other factors which would make the WLA either more stringent or less stringent? Yes - No If yes, explain below in the space provided.

D. Are there factors that make the water quality model outputs impractical or un-implementable at this time? Yes No If yes, explain below in the space provided.

E. Recommended limits

Flow: 2/4/8

BOD5 critical: 5.0 mg/L BOD5 seasonal: —

NH3-N critical: 0.5 mg/L NH3-N seasonal: —

UOD critical: — UOD seasonal: —

Effluent DO: 6.0 min

Phosphorus: MR

Other parameters: TN = MR, Hg limits due to impairment, various metals

Engineering comments: future nutrient criteria may lead to TP, TN limits

F. Is there agreement with water quality model outputs? Yes No

Engineer: Kyle Hamm

Date: 4/13/18

III. Water Quality Modeling Section

Is full agreement concluded? Yes No

If full agreement is not reached, see the wasteload allocation procedures for further steps.

If yes, the wasteload allocation is:

Flow: 2/4/8 MGD

BOD5 critical: 5.0 mg/L BOD5 seasonal: —

NH3-N critical: 0.5mg/L NH3-N seasonal: —

UOD critical: — UOD seasonal: —

Critical limits apply: — through —

Seasonal limits apply: — through —

Effluent DO: 6.0 mg/L (minimum)

Phosphorus: MR

Other parameters: TN=MR, Hg limits as recommended

Comments: Future nutrient criteria may lead to TP and TN limits.

Approval: [signature] Date: 6/13/18

Appendix D

Life Cycle Cost Analysis

CAPITAL COST SUMMARY

CENTRAL DORCHESTER WRF-4 MGD

Cost Component	Probable Cost	
Headworks	$1,439,000	
Bioreactor	$5,097,000	
Secondary Clarifiers	$2,536,000	
Effluent Filters	$1,164,000	
UV Disinfection	$843,000	
Effluent Structure	$975,000	
Post Aeration and Diffuser	$684,000	
RAS/WAS Pumping Station	$435,000	
Dewatering Building	$1,629,000	
Sludge Holding Tank	$777,000	
Operations Building	$535,000	
Chemical Feed Systems	$142,000	
Plant Drain Pump Station	$135,000	
Electrical Building	$374,000	
Generator	$474,000	
Site Development	$1,296,000	
Yard Piping	$1,644,000	
SCADA System	$475,000	
Electrical	$4,110,000	
SUBTOTAL	$24,764,000	
General Conditions, OH&P	$4,953,000	
Equipment Taxes	$450,000	
SUBTOTAL	$30,167,000	
Contingency @ 30%	$9,050,000	
TOTAL	$39,217,000	($9.80/Gal)

CAPITAL COST SUMMARY

CONVEYANCE-4 MGD

Cost Component	Probable Cost
Influent Force Main (24" 1,000 LF)	$ 230,000
Effluent Force Main (Open Cut; 24" 11,000 LF)	$2,530,000
Effluent Force Main (HDD; 24" 8,000 LF)	$3,680,000
Bore & Jack Crossings (36" Casing 200 LF)	$180,000
SUBTOTAL	$6,620,000
Contingency @ 30%	$1,986,000
TOTAL	$8,606,000

OPERATING COST SUMMARY

	YEAR 1 [(1)]
LABOR (with OH)	
Operator (4) @$35/hr.	$291,200
Supervisor (1) @ $50/hr.	$104,000
Total	$395,200
CHEMICALS	
Polymer	$117,700
Magnesium Hydroxide	$138,300
Total	$256,000
SLUDGE DISPOSAL	
WT/Day	9.1
Landfill Fees @ $20.15/WT	$67,000
Hauling @ $20/WT	$66,500
	$133,500
MAINTENANCE & REPAIR	$111,200
POWER	$321,000
TOTAL ANNUAL O&M COST	**$1,216,900**

(1) 4.0 MGD (Qmax30)

YEAR 1

POWER	NPHP	Run	Hrs.	kWh
Headworks	26	1	4	66
Anaerobic Mixers	3	4	24	182
First Anoxic Mixers	6.6	4	24	400
Aeration	75	4	24	4,566
Second Anoxic Mixers	6.6	4	24	400
Reaeration	10	2	24	304
Clarifiers and Filters	6	2	24	182
UV Disinfection	--	1	24	404
RAS	40	2	24	1,218
WAS Pumps	5	1	16	50
Effluent Pumps	150	1	17	1,616
Sludge Storage Aeration	20	1	24	304
Sludge Feed Pumps	5	2	8	50
Sludge Press/Conveyor	12.5	1	12	112
Utility Water Pumps	20	1	8	102
Plant Return Pumps	7.5	1	16	76
Building Lighting/HVAC	-	-	-	300
TOTAL, kWh/Day				10,332
kWh/MG				2,952
Power Cost @ $0.085/kWh				$321,000/yr.

NET PRESENT WORTH COST SUMMARY

COST COMPONENT	CENTRAL WRF	SCPW CONTRACT	NCSD CONTRACT
Pumping Station Upgrades	……	$ 1,500,000	$ 390,000[4]
Conveyance to Treatment	$ 299,000	$ 5,170,000	……
Treatment	$ 39,217,000	$ 13,000,000[2]	……
Conveyance to Discharge	$ 8,307,000	……	……
Land Acquisition/Easements	$ 1,000,000	$ 10,000	……
Engineering and Administration	$ 7,173,000	$ 2,950,000	$ 58,000
Total Capital Costs	$ 55,996,000	$ 22,630,000	$ 448,000
Annual Volumetric Costs	……	$ 3,832,500[3]	$ 7,173,000[5]
Annual O&M Costs (Dorchester Co.)	$ 1,216,900	$ 52,000	……
Present Worth of Annual Costs[1]	$ 21,960,000	$ 70,098,000	$ 129,444,000
Net Present Worth of Alternative	**$ 77,956,000**	**$ 92,728,000**	**$ 129,892,000**

1 Net Present Worth Costs based upon 20 years, I=3.0%
2 Summerville CPW (SCPW) Estimate (4 MGD)
3 SCPW Estimate $3.00/1000 gallons inflated 2%/yr.; Average Annual flow of 3.5 MGD
4 North Charleston Sewer District (NCSD) Estimate for 2 MGD; 4 MGD will require PS expansion and parallel force main
5 NCSD Estimate $5.615/1000 gallons inflated 2%/yr.; Average Annual flow of 3.5 MGD

Appendix B: Total Present Worth of Evaluated Alternatives

Expansion of Lower Dorchester WWTP from 8 to 16 mgd

Present Worth Analysis

COMPONENT	CAPITAL COST	ANNUAL O&M COST	SALVAGE VALUE USEFUL LIFE (yrs)	SALVAGE VALUE SALVAGE AMOUNT
Lower Dorchester WWTP Expansion from 8 to 16 mgd				
Demolition	$759,000			
Site Work	$4,690,000			
Yard Piping	$8,356,000		30	$2,785,333
Preliminary Treatment Facility, Influent/RAS Distribution, WAS Pumping	$7,544,000		30	$2,514,667
New Aeration Basins 1 and 2	$13,245,000		30	$4,415,000
Retrofit of Aeration Basins 3 and 4	$9,343,000		30	$3,114,333
Mixed Liquor Suspended Solids Distribution Box	$1,578,000		30	$526,000
Secondary Clarifiers	$4,595,000		30	$1,531,667
Return Activated Sludge Pump Station 5	$1,424,000		30	$474,667
Blower Building	$4,987,000		30	$1,662,333
Tertiary Disk Filter and Tertiary Effluent Box	$2,021,000		30	$673,667
UV Disinfection and Building	$5,209,000		30	$1,736,333
Thickening Building	$5,770,000		30	$1,923,333
Aerated Sludge Holding and Blowers	$2,942,000		30	$980,667
Electrical Work and Generator	$6,684,000		30	$2,228,000
General Conditions	$11,872,000			
Lower Dorchester WWTP Annual Treatment Costs		$6,302,000		
Administrative and Engineering Costs	$11,832,000			
TOTAL	**$102,900,000**	**$6,300,000**		**$24,600,000**

CAPITAL COST	PRESENT WORTH OF O&M	PRESENT WORTH OF SALVAGE
$103,000,000	$162,000,000	$23,000,000

Total Present Worth of Lower Dorchester WWTP Expansion, 8 to 16 mgd **$242,000,000**

<u>Total Present Worth Notes</u>

All costs in 2019 dollars		
Piping & Structural Life	50	years
Mechanical & Electrical Life	20	years
Aggregate Structural / Mechanical / Electrical	30	years
Time Period =	20	
Interest Rate =	0.4%	2020 Discount Rates for OMB Circular No. A-94, M-20-07

<u>Capital Cost Opinion Notes:</u>

1. Cost opinion includes 3% for bonds and insurance.
2. Cost opinion includes 20% contractor overhead and profit and 7% taxes.
3. Cost opinion assumes 30% contingency included in each line item.
4. General conditions assumes 15% to include mobilization, contract administration, trailor, field supervisor, shop drawings, start-up / training, etc.
5. Present Worth of O&M based on a variable rate using 2019 O&M costs from County's CAFR and a 2019 average day flow of 6.8 mgd.
6. O&M costs exclude staffing expenses.

Expansion of Lower Dorchester WWTP from 8 to 12 mgd AND Proposed Central WWTP at 4 mgd

Present Worth Analysis

COMPONENT	CAPITAL COST	ANNUAL O&M COST	SALVAGE VALUE: USEFUL LIFE (yrs)	SALVAGE VALUE: SALVAGE AMOUNT
Lower Dorchester WWTP Expansion from 8 to 12 mgd				
Demolition	$759,000			
Site Work	$4,690,000			
Yard Piping	$7,938,000		30	$2,646,000
Preliminary Treatment Facility, Influent/RAS Distribution, WAS Pumping	$6,790,000		30	$2,263,333
New Aeration Basins 1 and 2	$13,245,000		30	$4,415,000
Retrofit of Aeration Basins 3 and 4	$9,343,000		30	$3,114,333
Mixed Liquor Suspended Solids Distribution Box	$1,578,000		30	$526,000
Secondary Clarifiers	$2,298,000		30	$766,000
Return Activated Sludge Pump Station 5	$1,343,000		30	$447,667
Blower Building	$4,837,000		30	$1,612,333
Tertiary Disk Filter and Tertiary Effluent Box	$1,441,000		30	$480,333
UV Disinfection and Building	$5,209,000		30	$1,736,333
Thickening Building	$5,770,000		30	$1,923,333
Aerated Sludge Holding and Blowers	$2,942,000		30	$980,667
Electrical Work and Generator	$6,684,000		30	$2,228,000
General Conditions	$11,230,000			
Administrative and Engineering Costs (13%)	$11,193,000			
Lower Dorchester WWTP Annual Treatment Costs		$6,302,000		
Proposed Central Dorchester WWTP of 4 mgd (From W.K. Dickson PER, 2019)				
Conveyance to Treatment	$299,000			
Treatment				
Headworks	$1,439,000		30	$479,667
Bioreactor	$5,097,000		30	$1,699,000
Secondary clarifiers	$2,536,000		30	$845,333
Effluent filters	$1,164,000		30	$388,000
UV Disinfection	$843,000		30	$281,000
Effluent structure	$975,000		30	$325,000
Post Aeration and Diffuser	$684,000		30	$228,000
RAS/WAS Pumping Station	$435,000		30	$145,000
Dewatering Building	$1,629,000		30	$543,000
Sludge Holding Tank	$777,000		30	$259,000
Operations Building	$535,000		30	$178,333
Chemical Feed Systems	$142,000		30	$47,333
Plant Drain Pump station	$135,000		30	$45,000
Electrical Building	$374,000		30	$124,667
Generator	$474,000		30	$158,000
Site Development	$1,296,000			
Yard Piping	$1,644,000		30	$548,000
SCADA System	$475,000		30	$158,333
Electrical	$4,110,000		30	$1,370,000
General Conditions and O&P	$4,953,000			
Equipment Taxes	$450,000			
Contingency (30%)	$9,050,000			
Conveyance to Discharge	$8,307,000			
Land AcquisitionJEasements	$1,000,000			
Administrative and Engineering Costs (13%)	$7,173,000			
Central Dorchester WWTP Annual Treatment Costs (start-up flow 2 mgd)		$1,854,000		
TOTAL	**$153,300,000**	**$8,200,000**		**$31,000,000**

CAPITAL COST	PRESENT WORTH OF O&M	PRESENT WORTH OF SALVAGE
$153,000,000	$190,000,000	$29,000,000

Total Present Worth of Lower Dorchester WWTP Expansion, 8 to 12 mgd and Proposed Central WWTP of 4 mgd **$314,000,000**

Total Present Worth Notes

All costs in 2019 dollars		
Piping & Structural Life	50	years
Mechanical & Electrical Life	20	years
Aggregate Structural / Mechanical / Electrical	30	years
Time Period =	20	
Interest Rate =	0.4%	2020 Discount Rates for OMB Circular No. A-94, M-20-07

Capital Cost Opinion Notes:

1. Lower Dorchester WWTP cost opinion includes 3% for bonds and insurance.
2. Lower Dorchester cost opinion includes 20% contractor overhead and profit and 7% taxes.
3. Lower Dorchester cost opinion assumes 30% contingency included in each line item.
4. Lower Dorchester WWTP general conditions assumes 15% to include mobilization, contract administration, trailor, field supervisor, shop drawings, start-up / training, etc.
5. Proposed Central WWTP Cost Opinion from W.K. Dickson PER (2019) Appendix D.
6. Present Worth of O&M based on a variable rate using 2019 O&M costs from County's CAFR, a 2019 average day flow of 6.8 mgd at Lower, and a startup flow of 2 mgd at Central.

7. O&M costs exclude staffing expenses.

North Charleston Sanitary District Contract

Present Worth Analysis

COMPONENT	CAPITAL COST	ANNUAL O&M COST	SALVAGE VALUE USEFUL LIFE (yrs)	SALVAGE VALUE SALVAGE AMOUNT
Lower Dorchester WWTP Expansion from 8 to 12 mgd				
Demolition	$759,000			
Site Work	$4,690,000		30	$1,563,333
Yard Piping	$7,938,000		30	$2,646,000
Preliminary Treatment Facility, Influent/RAS Distribution, WAS Pumping	$6,790,000		30	$2,263,333
New Aeration Basins 1 and 2	$13,245,000		30	$4,415,000
Retrofit of Aeration Basins 3 and 4	$9,343,000		30	$3,114,333
Mixed Liquor Suspended Solids Distribution Box	$1,578,000		30	$526,000
Secondary Clarifiers	$2,298,000		30	$766,000
Return Activated Sludge Pump Station 5	$1,343,000		30	$447,667
Blower Building	$4,837,000		30	$1,612,333
Tertiary Disk Filter and Tertiary Effluent Box	$1,441,000		30	$480,333
UV Disinfection and Building	$5,209,000		30	$1,736,333
Thickening Building	$5,770,000		30	$1,923,333
Aerated Sludge Holding and Blowers	$2,942,000		30	$980,667
Electrical Work and Generator	$6,684,000		30	$2,228,000
General Conditions	$11,230,000			
Administrative and Engineering Costs (13%)	$11,193,000			
Lower Dorchester WWTP Annual Treatment Costs (at 3%)		$6,302,313		
Improvements Needed for NCSD Lease (From WK Dickson PER, 2019)				
Pump Station Upgrades (Lower Dorchester WWTP to NCSD WWTP)	$3,000,000		30	$1,000,000
15.5 miles of force main, 20-inch diameter	$19,640,000		30	$6,546,667
NCSD Contract Costs per year ($5.615/1,000 gals, 2% per year)		$8,197,900		
Administrative and Engineering Costs	$2,943,200			
TOTAL	**$122,900,000**	**$14,500,000**		**$32,200,000**

CAPITAL COST	PRESENT WORTH OF O&M	PRESENT WORTH OF SALVAGE
$123,000,000	$278,000,000	$30,000,000

Total Present Worth of Lower Dorchester WWTP Expansion, 8 to 12 mgd and NCSD Contract **$371,000,000**

Total Present Worth Notes

All costs in 2019 dollars		
Piping & Structural Life	50	years
Mechanical & Electrical Life	20	years
Aggregate Structural / Mechanical / Electrical	30	years
Time Period =	20	
Interest Rate =	0.4%	2020 Discount Rates for OMB Circular No. A-94, M-20-07
Interest Rate (NCSD Contract) =	2.0%	

Capital Cost Opinion Notes:

1. Per W.K. Dickson PER: North Charleston Sewer District (NCSD) estimate for 2 mgd; 4 mgd will require pump station expansion and parallel force main. Hazen assumed $3 million for pump station expansion and $12/inch-diameter foot for force main.
2. NCSD Estimate $5.615/1000 gallons inflated 2% per year.
3. WKD's administrative costs for the NCSD option updated to reflect the additional force main required.
4. Present Worth of O&M based on a variable rate using 2019 O&M costs from County's CAFR, a 2019 average day flow of 6.8 mgd at Lower, and an initial flow of 2 mgd to NCSD.

Summerville CPW Contract

Present Worth Analysis

COMPONENT	CAPITAL COST	ANNUAL O&M COST	SALVAGE VALUE: USEFUL LIFE (yrs)	SALVAGE VALUE: SALVAGE AMOUNT
Lower Dorchester WWTP Expansion from 8 to 12 mgd				
Demolition	$759,000			
Site Work	$4,690,000			
Yard Piping	$7,938,000		30	$2,646,000
Preliminary Treatment Facility, Influent/RAS Distribution, WAS Pumping	$6,790,000		30	$2,263,333
New Aeration Basins 1 and 2	$13,245,000		30	$4,415,000
Retrofit of Aeration Basins 3 and 4	$9,343,000		30	$3,114,333
Mixed Liquor Suspended Solids Distribution Box	$1,578,000		30	$526,000
Secondary Clarifiers	$2,298,000		30	$766,000
Return Activated Sludge Pump Station 5	$1,343,000		30	$447,667
Blower Building	$4,837,000		30	$1,612,333
Tertiary Disk Filter and Tertiary Effluent Box	$1,441,000		30	$480,333
UV Disinfection and Building	$5,209,000		30	$1,736,333
Thickening Building	$5,770,000		30	$1,923,333
Aerated Sludge Holding and Blowers	$2,942,000		30	$980,667
Electrical Work and Generator	$6,684,000		30	$2,228,000
General Conditions	$11,230,000			
Administrative and Engineering Costs (13%)	$11,193,000			
Lower Dorchester WWTP Annual Treatment Costs		$6,302,000		
Improvements Needed for Summerville Lease (From WK Dickson PER, 2019)				
Pumping Station Upgrades	$1,500,000			
Conveyance to Treatment	$5,170,000			
Treatment (see note 4)	$15,050,000			
Land Acquisition and Easements	$10,000			
Administrative and Engineering Costs	$2,950,000			
Summerville Contract Costs per year ($3.00/1,000 gals, 2% per year)		$4,380,000		
TOTAL	**$122,000,000**	**$10,700,000**		**$23,100,000**

CAPITAL COST	PRESENT WORTH OF O&M	PRESENT WORTH OF SALVAGE
$122,000,000	$218,000,000	$21,000,000

Total Present Worth of Lower Dorchester WWTP Expansion, 8 to 12 mgd and Summerville Contract **$319,000,000**

Total Present Worth Notes

All costs in 2019 dollars		
Piping & Structural Life	50	years
Mechanical & Electrical Life	20	years
Aggregate Structural / Mechanical / Electrical	30	years
Time Period =	20	
Interest Rate =	0.4%	2020 Discount Rates for OMB Circular No. A-94, M-20-07
Interest Rate (Summerville Contract) =	2.0%	

Capital Cost Opinion Notes:

1. SCPW Estimate $3.00/1000 gallons inflated 2% per year.
2. Present Worth of O&M based on a variable rate using 2019 O&M costs from County's CAFR, a 2019 average day flow of 6.8 mgd at Lower, and an initial flow of 2 mgd to Summerville.
3. Summerville treatment costs adjusted for inflation.
4. Salvage value for Summerville CPW system does not count as a Dorchester County asset.

Land Application

Present Worth Analysis

COMPONENT	CAPITAL COST	ANNUAL O&M COST	SALVAGE VALUE USEFUL LIFE (yrs)	SALVAGE VALUE SALVAGE AMOUNT
Lower Dorchester WWTP Expansion from 8 to 16 mgd				
Demolition	$759,000			
Site Work	$4,690,000			
Yard Piping	$8,356,000		30	$2,785,333
Preliminary Treatment Facility, Influent/RAS Distribution, WAS Pumping	$7,544,000		30	$2,514,667
New Aeration Basins 1 and 2	$13,245,000		30	$4,415,000
Retrofit of Aeration Basins 3 and 4	$9,343,000		30	$3,114,333
Mixed Liquor Suspended Solids Distribution Box	$1,578,000		30	$526,000
Secondary Clarifiers	$4,595,000		30	$1,531,667
Return Activated Sludge Pump Station 5	$1,424,000		30	$474,667
Blower Building	$4,987,000		30	$1,662,333
Tertiary Disk Filter and Tertiary Effluent Box	$1,010,500		30	$336,833
UV Disinfection and Building	$2,604,500		30	$868,167
Thickening Building	$5,770,000		30	$1,923,333
Aerated Sludge Holding and Blowers	$2,942,000		30	$980,667
Electrical Work and Generator	$6,684,000		30	$2,228,000
General Conditions	$11,872,000			
Administrative and Engineering Costs (13%)	$11,363,000			
Lower Dorchester WWTP Annual Treatment Costs (at 3%)		$6,302,000		
Land Application Costs				
Land Acquisition and Easements	$57,000,000			
Spray Field Infrastructure	$30,936,000			
60-day storage pond	$19,113,000			
Effluent Pump Station to Land Application Site	$871,000		30	$290,333
Effluent Force Main	$7,260,000		30	$2,420,000
Contingency (30%)	$34,554,000			
Annual application O&M per year (assumes $1.00/1,000 gallons)		$730,000		
Administrative and Engineering Costs (8%)	$11,978,720			
TOTAL	**$260,500,000**	**$7,000,000**		**$26,100,000**

CAPITAL COST	PRESENT WORTH OF O&M	PRESENT WORTH OF SALVAGE
$261,000,000	$176,000,000	$24,000,000

Total Present Worth of Lower Dorchester WWTP Expansion, 8 to 16 mgd and Land Application of 4 mgd **$413,000,000**

Total Present Worth Notes

All costs in 2019 dollars		
Piping & Structural Life	50	years
Mechanical & Electrical Life	20	years
Aggregate Structural / Mechanical / Electrical	30	years
Time Period =	20	
Interest Rate =	0.4%	2020 Discount Rates for OMB Circular No. A-94, M-20-07

Capital Cost Opinion Notes:

1. Assumes $20,000 per acre to acquire land from established timber company.
2. Assumes $30,000 per acre for spray irrigation system.
3. Present Worth of O&M based on a variable rate using 2019 O&M costs from County's CAFR.
4. O&M costs exclude staffing expenses.

Reuse via Land Application

Present Worth Analysis

COMPONENT	CAPITAL COST	ANNUAL O&M COST	SALVAGE VALUE USEFUL LIFE (yrs)	SALVAGE AMOUNT
Lower Dorchester WWTP Expansion from 8 to 16 mgd				
Demolition	$759,000			
Site Work	$4,690,000			
Yard Piping	$8,356,000		30	$2,785,333
Preliminary Treatment Facility, Influent/RAS Distribution, WAS Pumping	$7,544,000		30	$2,514,667
New Aeration Basins 1 and 2	$13,245,000		30	$4,415,000
Retrofit of Aeration Basins 3 and 4	$9,343,000		30	$3,114,333
Mixed Liquor Suspended Solids Distribution Box	$1,578,000		30	$526,000
Secondary Clarifiers	$4,595,000		30	$1,531,667
Return Activated Sludge Pump Station 5	$1,424,000		30	$474,667
Blower Building	$4,987,000		30	$1,662,333
Tertiary Disk Filter and Tertiary Effluent Box	$2,021,000		30	$673,667
UV Disinfection and Building	$5,209,000		30	$1,736,333
Thickening Building	$5,770,000		30	$1,923,333
Aerated Sludge Holding and Blowers	$2,942,000		30	$980,667
Electrical Work and Generator	$6,684,000		30	$2,228,000
General Conditions	$11,872,000			
Administrative and Engineering Costs (13%)	$11,832,000			
Lower Dorchester WWTP Annual Treatment Costs (at 3%)		$6,302,000		
Land Application of Reuse Costs				
Land Acquisition and Easements	$45,000,000			
Spray Field Infrastructure	$30,936,000			
60-day storage pond	$9,557,000			
Effluent Pump Station to Land Application Site	$871,000		30	$290,333
Effluent Force Main	$7,260,000		30	$2,420,000
Contingency (30%)	$28,087,200			
Annual application O&M per year (assumes $1.00/1,000 gallons)		$730,000		
Administrative and Engineering Costs (8%)	$9,736,896			
TOTAL	**$234,000,000**	**$7,000,000**		**$27,000,000**

CAPITAL COST	PRESENT WORTH OF O&M	PRESENT WORTH OF SALVAGE
$234,000,000	$176,000,000	$25,000,000

Total Present Worth of Lower Dorchester WWTP Expansion, 8 to 16 mgd and Land Application of 4 mgd Reclaimed Water **$385,000,000**

Total Present Worth Notes

All costs in 2019 dollars		
Piping & Structural Life	50	years
Mechanical & Electrical Life	20	years
Aggregate Structural / Mechanical / Electrical	30	years
Time Period =	20	
Interest Rate =	0.4%	2020 Discount Rates for OMB Circular No. A-94, M-20-07

Capital Cost Opinion Notes:

1. Assumes $30,000 per acre to acquire land from established timber company.
2. Assumes $30,000 per acre for spray irrigation system.
3. Present Worth of O&M based on a variable rate using 2019 O&M costs from County's CAFR.
4. O&M costs exclude staffing expenses.

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