McKim & Creed Project No. 06496-0009

Hendersonville
Water Sewer

Final

# WASTEWATER TREATMENT FACILITY MASTER PLAN

JUNE 2022

---


SUBMITTED TO:

**CITY OF HENDERSONVILLE**
305 Williams Street, Hendersonville, NC 28792
828.697.3073

SUBMITTED BY:

**MCKIM & CREED**
8020 Tower Point Drive, Charlotte, NC 28227
Firm License No. F-1222
704.841.2588

McKIM&CREED
ENGINEERS SURVEYORS PLANNERS

# CITY OF HENDERSONVILLE
## WASTEWATER TREATMENT FACILITY

## Final Master Plan

Date: June 2022

**Prepared for:**

Hendersonville Water Sewer

City of Hendersonville
305 Williams Street
Hendersonville, NC 28792

NORTH CAROLINA PROFESSIONAL SEAL 046291 ENGINEER ZACHARY M. TRAMMEL
6/24/2022


**Prepared by:**
McKim & Creed, Inc.
8020 Tower Point Dr.
Charlotte, NC 28227
Firm License No. F-1222


**McKim & Creed Project 06496-0009**

# CONTENTS

# TABLES



# FIGURES



# APPENDICES

# LIST OF ACRONYMS

| Abbreviation | Definition |
|---|---|
| AADF | Annual Average Daily Flow |
| BFP | Belt Filter Press |
| $BOD_5$ | Biochemical Oxygen Demand, Five Day |
| CIP | Capital Improvement Plan |
| CMU | Concrete Masonry Unit |
| CPU | Central Processing Unit |
| DMZ | Demilitarized Zone (computing) |
| DO | Dissolved Oxygen |
| EPA | Environmental Protection Agency |
| EQ | Equalization |
| FEMA | Federal Emergency Management Agency |
| FOG | Fats, Oils, and Grease |
| GBT | Gravity Belt Thickener |
| HMI | Human Machine Interface |
| HRT | Hydraulic Retention Time |
| IPS | Influent Pumping Station |
| L | Liters |
| MG | Million Gallons |
| mg | milligram |
| MGD | Million Gallons per Day |
| MLSS | Mixed Liquor Suspended Solids |
| MMADF | Maximum Monthly Average Daily Flow |
| NCDEQ | North Carolina Department of Environmental Quality |
| $NO_3$ | Nitrate |
| NPDES | National Pollutant Discharge Elimination System |
| PF | Peaking Factor |
| PHF | Peak Hourly Flow |
| PLC | Programmable Logic Controller |
| RAS | Return Activated Sludge |
| RDT | Rotary Drum Thickener |
| RPS | Recycle Pumping Station |
| RRVS | Resistor Reduced Voltage Starter |
| R&R | Repair and Replacement |
| SB | Switchboard |
| SCADA | Supervisory Control and Data Acquisition |
| SLR | Solids Loading Rate |
| SRT | Solids Retention Time |
| SSAIA | Sanitary Sewer Asset Inventory and Assessment |
| SSO | Sanitary Sewer Overflow |
| TKN | Total Kjeldahl Nitrogen |
| TM | Technical Memorandum |
| TM1 | Technical Memorandum No. 1 |
| TM2 | Technical Memorandum No. 2 |
| TM3 | Technical Memorandum No. 3 |
| TP | Total Phosphorus |

| | |
|---|---|
| TS | Total Solids |
| TSS | Total Suspended Solids |
| UV | Ultraviolet |
| UVD | Ultra-Violet Disinfection |
| VSS | Volatile Suspended Solids |
| WAS | Waste Activated Sludge |
| WWTF | Wastewater Treatment Facility |

# 1. EXECUTIVE SUMMARY

## 1.1 Introduction

The City of Hendersonville owns and operates a wastewater treatment facility (WWTF) located at 99 Balfour Road in Hendersonville. The Hendersonville WWTF is designed to utilize an extended aeration activated sludge treatment process to achieve biological removal of organic pollutants and achieve complete nitrification prior to surface water discharge of treated effluent to Mud Creek per NPDES permit number NC00255343. The Hendersonville WWTF is permitted to discharge up to 4.8 million gallons per day (MGD) of treated effluent on a maximum monthly average daily flow (MMADF) basis.

The purpose of this Master Plan document is to provide an executive summary of the findings of the three prior technical memoranda (TMs) that have been prepared for the Hendersonville WWTF, and to present the recommended Capital Improvement Plan (CIP) for the facility based on the findings and recommendations presented in the prior TMs. TM's 1, 2, and 3 (attached as appendices) are referenced throughout this executive summary and are summarized below:

- <u>TM1 – Preliminary Evaluations and Condition Assessments</u>: This TM provides a review of previous engineering studies and planning documents to document previous flow projections and recommendations. This TM also provides a review of condition assessments performed for the existing processes, equipment, and major systems and provides recommendations for rehabilitation, replacement, and upgrades to address issues noted by the condition assessments. This TM also summarizes the following information:
    - Facility history
    - Master plan goals and objectives
    - Permit requirements
    - Historical influent flow data and future flow projections
    - Influent and effluent water quality data
    - Influent water quality data for process modeling
    - Process data
    - Asset management data for major process equipment
    - Current capital improvement plan (prior to this Master Plan)

- TM2 – Treatment Process Evaluations: This TM provides a detailed review of existing treatment processes to document capacity limitations, identify replacement and expansion needs, and evaluate alternatives to meet replacement and expansion needs. The following information is summarized in this TM:
  - Hydraulic evaluation of the existing facility at current and future flows
  - Current capacity analyses for each treatment process
  - BioWin wastewater process modeling
  - Evaluation and comparison of treatment technology alternatives for improvements and expansion
  - Recommendations for improvements and expansion, including conceptual cost opinions
- TM3 – Flow Equalization Preliminary Engineering Evaluation: This TM evaluates alternatives to provide new flow equalization facilities to address hydraulic limitations identified by previous engineering studies. Basis of design criteria are provided in this TM for the recommended flow equalization facilities.

## 1.2 Existing Condition Assessments

The initial phase of the master plan for the Hendersonville WWTF included condition assessments of existing major equipment, processes, systems, and structures at the WWTF. Condition assessments were performed with lead engineers from each major discipline, including civil, process/mechanical, structural, electrical, and instrumentation and controls (SCADA). The findings and recommendations of the existing condition assessments, as described in TM1, are summarized in **Table 1.1** and **Table 1.2** below.

*Table 1.1 – Summary of Existing Condition Assessment Recommendations*

| Process Area | Facility Need | Expected Timeframe (years) | Preliminary Priority Ranking |
|---|---|---|---|
| Administration Building | Perform engineering analysis of existing footings and pile caps to determine repair modifications to remove potential for continuing settlement. Engineering analysis to include subsurface soil investigation. Perform associated foundation and wall repairs per recommendations of the engineering analysis. | 5 | 53 |
| Power Distribution | Replace switchboards 'SB-1' and 'SB-2'. | 10 | 46 |
| Septage Receiving | Install weigh scales or flow meter to track septage receiving. | 10 | 50 |
| Influent Pumping Station | Repair cracks in exterior top of wet well wall. | 1 | 12 |
| Influent Pumping Station | Replace influent pumps. | 10 | 32 |

| Process Area | Facility Need | Expected Timeframe (years) | Preliminary Priority Ranking |
|---|---|---|---|
| Influent Pumping Station | Replace influent flow measurement. | 10 | 33 |
| Influent Pumping Station | Replace wet well level measurement equipment. | 5 | 40 |
| Influent Pumping Station | Repair cracks in walls and slabs. | 10 | 41 |
| Influent Pumping Station | IPS ventilation system improvements. | 5 | 42 |
| Influent Pumping Station | Evaluate and implement modifications to alleviate FOG build-up in wet well. | 10 | 47 |
| Screening and Grit Removal | Replace screening and grit removal equipment. Recommend relocation upstream of Influent Pumping Station. | 10 | 31 |
| Screening and Grit Removal | Repair cracks in slabs. | 10 | 53 |
| Screening and Grit Removal | Repair continuous crack between aeration basin and north wall. | 5 | 53 |
| Aeration Basins | Perform engineering analysis of bowing/deflection in aeration basin #2 north wall to develop repair recommendations. | 1 | 1 |
| Aeration Basins | Survey aeration basin #2 north wall to measure and monitor deflection. | 1 | 2 |
| Aeration Basins | Perform engineering analysis of aeration basins to verify structural integrity and develop repair plans. | 1 | 4 |
| Aeration Basins | Repair aeration basin #2 north wall bowing/deflection following recommendations of engineering analysis. | 2 | 5 |
| Aeration Basins | Repair cracks in faces of exterior walls following recommendations of engineering analysis. | 2 | 6 |
| Aeration Basins | Replace air header isolation valves in aeration basin #1 at time of diffuser replacement. | 1 | 9 |
| Aeration Basins | Repair cracks in walkway slabs and top of walls. | 10 | 53 |
| Blower Building | Perform subsurface soils investigation to identify repair strategies to correct settling issues. | 1 | 11 |
| Blower Building | Recoat blower discharge piping to protect from corrosion. | 2 | 16 |
| Blower Building | Repair/replace sidewalks, pipe supports, access stair framing, columns, footings, and roof framing (if required) following recommendations of subsurface soils investigation. | 2 | 17 |
| Blower Building | Replace existing blowers and provide variable speed control. | 10 | 24 |

| Process Area | Facility Need | Expected Timeframe (years) | Preliminary Priority Ranking |
|---|---|---|---|
| Blower Building | Replace existing RRVS motor controllers at time of blower replacement. Provide variable speed control for future blowers. | 10 | 24 |
| Secondary Clarifiers | Have equipment manufacturer inspect clarifier mechanical and drive mechanisms and provide rehabilitation recommendations. | 5 | 23 |
| Secondary Clarifiers | Rehabilitate/rebuild existing clarifier mechanical and drive mechanisms. | 10 | 34 |
| Secondary Clarifiers | Replace clarifier scum boxes. | 10 | 34 |
| Secondary Clarifiers | Repair cracks in exterior walls. | 10 | 53 |
| Recycle Pumping Station | Replace RAS pump #2 and WAS pumps. | 5 | 27 |
| Recycle Pumping Station | Perform RPS heating and ventilation system improvements. | 5 | 43 |
| Recycle Pumping Station | Repair cracks in walls, slabs, and exterior top of walls. | 10 | 53 |
| Tertiary Filters | Replace Tertiary Filter #2. | 5 | 25 |
| Tertiary Filters | Install clear span structure over tertiary filters. | 5 | 26 |
| Tertiary Filters | Repair cracks in north wall. | 10 | 53 |
| Utility Building | Replace seal water pumping system. | 10 | 48 |
| Disinfection Basin | Replace UV Disinfection System in new channel. | 5 | 13 |
| Disinfection Basin | Install isolation transformer with UVD system replacement. | 5 | 14 |
| Disinfection Basin | Repair cracks in walls and slabs. | 10 | 54 |
| Disinfection Basin | Replace existing fiberglass grating. | 5 | 55 |
| Sludge Thickening | Evaluate cost-benefit for new aerated sludge holding tank vs. new GBT/RDT and conversion of existing thickeners to aerated sludge holding tanks. | 1 | 8 |
| Sludge Thickening | Repair cracks in gravity thickener #1 and install interior/exterior coating system to rehabilitate and protect existing concrete basin. | 2 | 15 |
| Sludge Thickening | Have equipment manufacturer inspect gravity thickener mechanical and drive mechanisms and provide rehabilitation recommendations. | 5 | 22 |
| Sludge Thickening | Replace belt filter press feed pumps. | 5 | 28 |
| Sludge Thickening | Relocate isolation valves on thickened sludge suction piping. | 5 | 28 |

| Process Area | Facility Need | Expected Timeframe (years) | Preliminary Priority Ranking |
|---|---|---|---|
| Sludge Thickening | Install aerated sludge holding tank or install new GBT/RDT and convert existing thickeners to aerated thickened sludge holding. | 10 | 35 |
| Sludge Thickening | Rehabilitate/rebuild existing gravity thickener mechanical and drive mechanisms. | 10 | 36 |
| Sludge Thickening | Install interior coating systems in gravity thickener #2. | 5 | 39 |
| Sludge Thickening | Repair cracks in thickening building and install new steel beams to support roof slab (if required). | 10 | 49 |
| Sludge Dewatering | Evaluate pressing schedule and process automation to improve operation, improve dewatered cake consistency, and reduce odor issues. | 1 | 7 |
| Sludge Dewatering | Replace BFP #1 filter belts. | 2 | 18 |
| Sludge Dewatering | Replace roller bearings on BFP #1 and #2. | 2 | 18 |
| Sludge Dewatering | Repair damaged CMU lintel beam on BFP room east wall entry door. | 1 | 19 |
| Sludge Dewatering | Replace dewatered cake conveyor belt, chain, rollers and bearings. | 5 | 21 |
| Sludge Dewatering | Replace polymer makedown skids. | 5 | 38 |
| Sludge Dewatering | Replace existing BFPs. | 10 | 45 |
| Biosolids Storage | Replace biosolids storage shelter roof. | 5 | 20 |
| Biosolids Storage | Install new protective coatings on structural steel members. | 5 | 37 |
| Lightning Protection | Install surge protective devices on all power distribution equipment. | 5 | 30 |
| Lightning Protection | Install facility wide grounding/lightning protection system. | 5 | 44 |
| Instrumentation and Control | Identify and correct all erroneous process/equipment data in SCADA HMI application. | 1 | 10 |
| Instrumentation and Control | Extend SCADA HMI application to control plant processes and equipment. | 5 | 29 |
| Instrumentation and Control | Implement intermediate DMZ network between plant control system and external networks. | 5 | 29 |
| Instrumentation and Control | Install SCADA historical database and permanent offline storage for long term data storage and use. | 5 | 29 |

| Process Area | Facility Need | Expected Timeframe (years) | Preliminary Priority Ranking |
|---|---|---|---|
| Instrumentation and Control | Implement SCADA system dashboards and reports to inform operations staff and improve facility operations. | 5 | 29 |
| Site/Civil | Test standing water on north side of aeration basins for indicators of wastewater contamination to determine presence of leaks from adjacent aeration basins. | 1 | 3 |
| Site/Civil | Investigate in-plant manhole #1 for damage to incoming piping due to potential settlement. Repair as necessary. | 1 | 51 |
| Site/Civil | Repair sidewalk settlement on west side of RPS to eliminate trip hazard from valve operating nuts. | 5 | 52 |
| Site/Civil | Regrade access road north of aeration basins to alleviate standing water issues. | 1 | 56 |

*Table 1.2 – Summary of Existing Condition Assessment Operational Recommendations*

| Process Area | Operational Recommendations |
|---|---|
| Screening and Grit Removal | Relocate upstream ultrasonic level transducer to reduce impacts from turbulence. |
| Aeration Basins | Reduce MLSS concentration to approx. 3,100 mg/L and corresponding SRT. |
| Aeration Basins | Install online DO and $NO_3$ analyzers in aeration basins to improve process monitoring and control. |
| Secondary Clarifiers | Install effluent launder covers to limit/eliminate algae growth. |
| Secondary Clarifiers | Install density current baffles. |
| Recycle Pumping Station | Automate sludge recycle and wasting operations to improve process control and consistency. |
| Recycle Pumping Station | Evaluate and implement improvements to provide adequate mixing or removal of scum from WAS wet well. |
| Tertiary Filters | Perform periodic chemical cleaning of cloth filter media. |
| Tertiary Filters | Replace cloth media every 5 to 10 years or as needed. |
| Sludge Thickening | Increase dewatering schedule to reduce sludge residence time in thickeners and prevent anaerobic conditions. |
| Sludge Dewatering | Automate sludge dewatering operations to improve operational efficiency. |
| Sludge Dewatering | Automate polymer makedown and feed systems to improve operational consistency. |
| Instrumentation and Control | Maintain stock of PLC spare parts on-site. |
| Instrumentation and Control | Maintain spare ethernet switches on site. |

## 1.3 Flow and Loading Projections

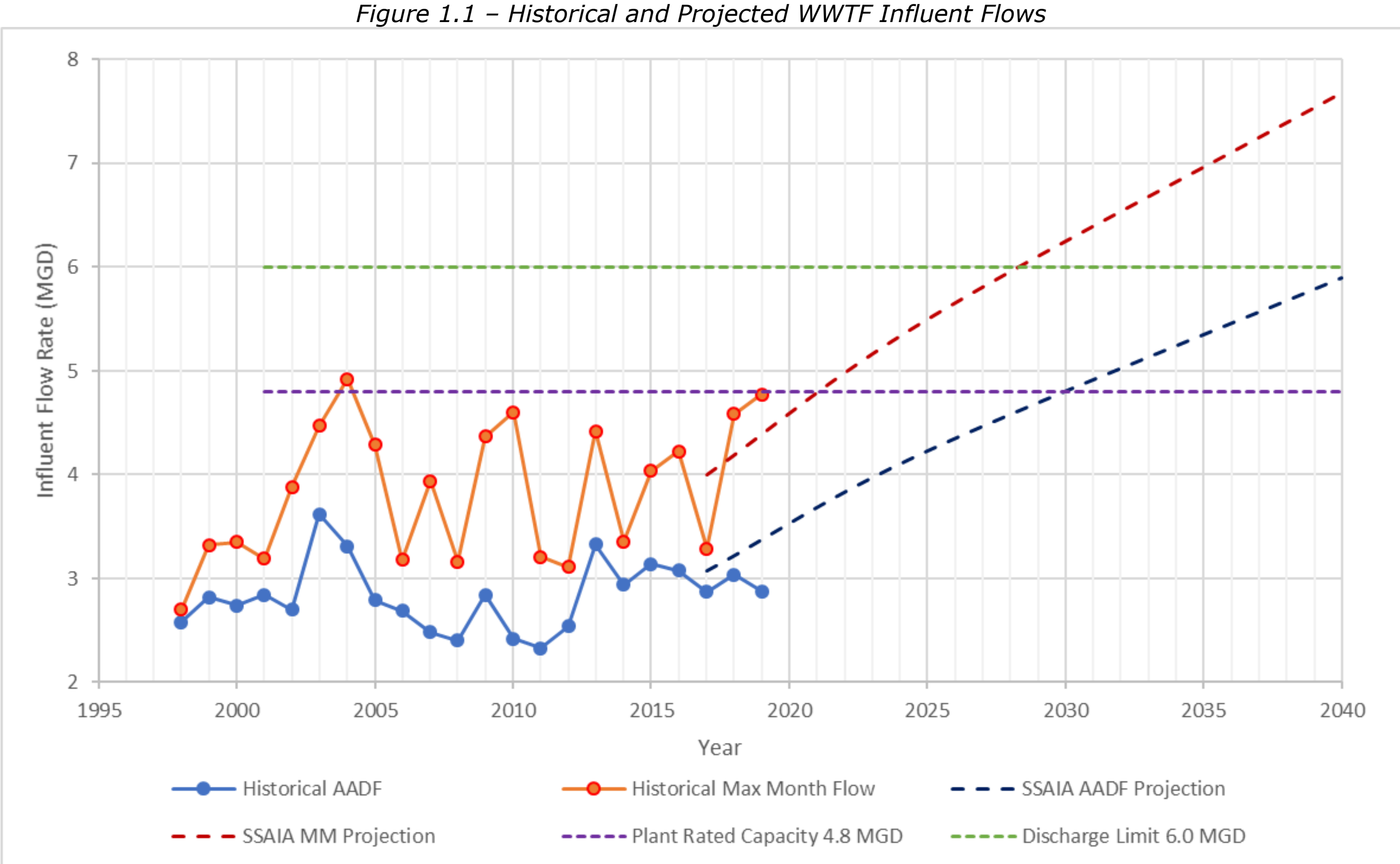
The influent flow and loading projections that served as the basis for all capacity evaluations and alternatives analyses were presented in TM1 to establish the basis of design conditions for the WWTF. The future influent flow projections from the City's Sanitary Sewer Asset Inventory and Assessment (SSAIA) Master Plan Report are shown in **Table 1.3** below and compared to the historical influent flows to the WWTF in **Figure 1.1** below. These future influent flow projections served as the basis for the timing of expansion needs for the City's WWTF.

*Table 1.3 – WWTF Influent Flow Projections from the SSAIA Master Plan Report*

| Flow Condition | Base Year (2017)[1] | 2025 | 2040 |
|---|---|---|---|
| Annual Average Daily Flow (AADF) | 3.07 MGD | 4.23 MGD | 5.90 MGD |
| Maximum Month Average Daily Flow (MMADF)[2] | 4.00 MGD | 5.50 MGD | 7.68 MGD |

*[1]Base year established by SSAIA Master Plan Report*
*[2]Based on 5-year average maximum month peaking factor (PF) of 1.30.*

*Figure 1.1 – Historical and Projected WWTF Influent Flows*



The WWTF's daily influent flow and water quality data from 2014 through 2019 was reviewed to support the process modeling efforts completed as part of this master plan. The basis of design average influent water quality to the WWTF is summarized in **Table 1.4** below.

*Table 1.4 – Average Influent Wastewater Concentrations for Process Modeling*

| Parameter | Units | Value |
|---|---|---|
| $BOD_5$ | mg/L | 219 |
| TSS | mg/L | 223 |
| VSS | mg/L | 156 |
| TKN | mg/L as N | 45 |
| TP | mg/L as P | 7 |

Peak wet weather flow rates were also estimated in the previous SSAIA Master Plan for the base year (2017), 2025, and 2040 for 2-year and 10-year storm events to model the peak flow conditions in the collection system. The peak wet weather flow rates estimated by the City's collection system model are summarized in **Table 1.5** below.

*Table 1.5 – Summary of Peak Wet Weather Flows*

| Year | 2-Year Storm Peak Flow (MGD) | 10-Year Storm Peak Flow (MGD) |
|---|---|---|
| Base (2017)[1] | 17.4 | 22.8 |
| 2025 | 22.5 | 29.4 |
| 2040 | 28.3 | 36.5 |

[1]*Base year established by SSAIA Master Plan report*

## 1.4 Hydraulic Evaluation

A limited survey of existing conditions and critical elevations was performed throughout the City's WWTF to document as-built elevations of weirs, pipe inverts, gate inverts, top of wall elevations, and other items for the primary purpose of updating the facility's existing hydraulic profile. Following collection of the as-built survey information, the existing facility's hydraulic profile was updated for the original design conditions including the permitted capacity of 4.8 MGD at maximum month average daily flow (MMADF) and the peak hourly flow (PHF) of 12.0 MGD. An evaluation of the WWTF hydraulics was also performed at the projected future flow rates to identify hydraulic limitations that may require correction to ensure the facility meets the required level of service per EPA and NCDEQ reliability requirements. A map of the as-built survey information collected, along with an updated existing facility hydraulic profile, is included in Appendix B. The findings of the hydraulic analysis at the projected future flows to the WWTF are summarized as follows:

- The influent Parshall flume in the influent pumping station wet well is limited to a peak flow of 21.39 MGD.
- The Influent Pumping Station is limited to a firm capacity of 12.4 MGD.
- The two 16-inch diameter influent force mains from the Influent Pumping Station are limited to a total capacity of 15 MGD based on limiting the maximum pipeline velocity to approximately 8 ft/s.
- The screening influent channel is limited to a peak flow of 17 MGD with one screen in service to maintain a minimum freeboard of 12-inches.
- Velocities through the influent mechanical bar screens exceed the maximum recommended flow-through velocity of 4 feet per second (fps) without downstream level control.
- The mixed liquor distribution box to the secondary clarifiers is limited to a total flow rate of 22.5 MGD (including both forward flow and RAS) to maintain a minimum freeboard of 12-inches.
- The existing Trojan UV4000 disinfection equipment is limited to 12.0 MGD; flows above 12.0 MGD through the existing UV equipment will submerge the tertiary filter effluent weirs.
- The existing 36-inch diameter WWTF outfall line is limited to a peak flow of 15 MGD during FEMA 100-year flood conditions (flood elevation of 2076.40); however, the lower portion of the cascade reaeration steps wall is submerged by flood waters under these conditions.

## 1.5 Treatment Process Capacity Evaluations

Following the completion of the existing condition assessments, the capacity limitations of all existing treatment processes were evaluated and documented in TM2. The WWTF's existing capacity limitations are summarized in **Table 1.6** below.

*Table 1.6 – Summary of Existing Treatment Process Limitations*

| Unit Process | Existing Capacity | Notes |
|---|---|---|
| Influent Pumping | 12.4 MGD PHF | Firm capacity of the influent pumping station. |
| Screening | 12.0 MGD PHF | Firm capacity with one mechanical screen in service. |
| Grit Removal | 17.7 MGD PHF | Max capacity with both grit chambers in service to maintain an HRT of 3 minutes. |
| Secondary Treatment | 15,300 lbs BOD/day and 3,100 lbs TKN/day at MMADF | Max capacity with both trains in operation. Max capacity corresponds to 8.39 MGD MMADF with influent wastewater characteristics summarized in TM2. |
| Tertiary Filters | 2.40 MGD PHF | Firm capacity with tertiary filter No. 1 out of service. |
| UV Disinfection | 12.0 MGD PHF | Maximum capacity of existing UV4000 system. |
| Cascade Reaeration | 4.23 MGD AADF | Limited by hydraulic loading rate per foot of step width, evaluated at AADF. Monitor effluent DO to identify need for expansion/replacement. |
| Gravity Thickeners | 15,700 lbs TS/day at maximum month | Firm capacity with one gravity thickener in service, with a peak SLR of 8 lbs/day/$ft^2$. Note, gravity thickeners are currently operated as un-aerated sludge holding basins. |
| Belt Filter Presses | 17,100 lbs TS/day at maximum month | Maximum capacity with both 2-meter BFPs operating at a SLR of 1,500 lbs/hr per BFP, 40 hours per week. |

Based on the current capacity evaluations, the existing tertiary filter No. 2 and the existing UV disinfection system do not meet current NCDEQ reliability requirements. Replacement of these systems should be completed prior to the next expansion to a permitted capacity of 6.0 MGD. In addition, the existing influent pumping station, screening, and grit removal systems are undersized for the current and future peak wet weather flows. These unit processes should be expanded or replaced prior to the next expansion to a permitted capacity of 6.0 MGD.

## 1.6 Flow Equalization Preliminary Engineering Evaluations

The need for influent flow equalization at the WWTF to prevent sanitary sewer overflows (SSOs) was identified in the City's SSAIA Master Plan report based on the peak wet weather flows predicted by the City's collection system model. Presently, the City's WWTF is limited to a peak hydraulic capacity of 12 MGD, while the current peak instantaneous wet weather flows predicted by the collection system model exceed 17 MGD. The City is currently forced to surcharge the existing 42-inch diameter gravity outfall line to the WWTF to provide storage capacity during significant wet weather events. The City desires to install influent flow equalization facilities at the WWTF to capture and store the peak wet weather flows to limit the occurrence of SSOs as much as possible. TM3 evaluated multiple alternatives to provide flow equalization facilities at the WWTF and provided the basis of design criteria for the recommended alternative. Inline flow equalization facilities are recommended to be installed at the biosolids handling facility across Balfour Road from the WWTF to limit peak wet weather flows to the WWTF, while also providing flow and loading equalization to the WWTF to maximize process control and effluent quality. The flow equalization facilities basis of design criteria is summarized in **Table 1.7** below.

*Table 1.7 – Flow Equalization Facilities Basis of Design Summary*

| Parameter | Units | Value |
|---|---|---|
| Number of Tanks | - | 1 |
| Diameter | ft | 160 |
| Design Sidewater Depth | ft | 20 |
| Design Freeboard | ft | 1 |
| Design Volume | MG | 3.0 |
| Overflow Capacity | MGD | 19.5 |
| Minimum Working Depth | ft | 3 |
| Design Working Volume | MG | 2.5 |
| Minimum Floor Slope | % | 2 |
| Level Measurement | - | Ultrasonic |
| DO Measurement | - | Luminescent DO Probe |
| Mixing and Aeration Equipment | - | Jet Aeration and Mixing |
| Equalized Flow Measurement | - | Electromagnetic Flow Meter |
| Equalized Flow Rate Control Method | - | Modulating Plug Valves |

## 1.7 Future Needs

Alternatives for future expansion of the WWTF were evaluated to meet the projected 2040 flows and loads presented above and documented in TM1. In addition, expansion needs were identified for an intermediate expansion to 6.0 MGD MMADF based on the effluent limits already established for this design condition in the City's current NPDES permit. A summary of the recommended improvements for each capacity expansion, as documented in TM2 and TM3, is provided in **Table 1.8** below.

*Table 1.8 – WWTF Expansion Summary*

| Process Area | Current Facilities 4.8 MGD MMADF | 2025 Facilities 6.0 MGD MMADF | 2035 Facilities 7.8 MGD MMADF |
|---|---|---|---|
| **Influent Pumping** | | | |
| Firm Capacity (Peak), Total | 12.4 MGD | 22.5 MGD | 28.3 MGD |
| **Screening** | | | |
| Total No. of Screens | 2 | 3 | 4 |
| Capacity (Peak), Each | 12 MGD | 11.25 MGD | 11.25 MGD |
| **Grit Removal** | | | |
| Total No. of Grit Separators | 2 | 2 | 2 |
| Capacity (Peak), Each | 8.85 MGD | 14.15 MGD | 14.15 MGD |
| Total No. of Grit Washers | 0 | 2 | 2 |
| Capacity, Each | 0 | 250 gpm | 250 gpm |
| **Flow Equalization** | | | |
| Total No. of EQ Basins | 0 | 1 | 1 |
| Volume, Total | 0 | 3.0 MG | 3.0 MG |
| **Secondary Treatment** | | | |
| Total No. of Aeration Basins | 2 | 2 | 3 |
| Volume, Total | 4.8 MG | 4.8 MG | 7.2 MG |
| Total No. of Secondary Clarifiers | 2 | 2 | 3 |
| Surface Area, Total | 12,720 $ft^2$ | 12,720 $ft^2$ | 19,080 $ft^2$ |
| **Tertiary Treatment** | | | |
| Total No. of Filters | 2 | 2 | 3 |
| Surface Area, Total | 2,432 $ft^2$ | 3,200 $ft^2$ | 3,950 $ft^2$ |
| Total No. of UV Disinfection Units | 1 | 2 | 2 |
| Capacity (Peak), Each | 12 MGD | 12 MGD (No. 1), 15 MGD (No. 2) | 15 MGD |
| Cascade Reaeration Step Width, Total | 8 ft | 8 ft | 12 ft |
| **Biosolids Handling** | | | |
| Total No. of Gravity Thickeners | 2 | 2 | 2 |
| Surface Area, Total | 3,920 $ft^2$ | 3,920 $ft^2$ | 3,920 $ft^2$ |
| Total No. of Aerated Thickened Sludge Holding Tanks | 0 | 2 | 2 |
| Volume, Total | 0 | 235,000 gal | 235,000 gal |
| Total No. of Belt Filter Presses | 2 | 2 | 2 |
| Capacity, Each | 1,500 lbs/hr | 1,500 lbs/hr | 1,500 lbs/hr |
| Total No. of Thermal Driers | 0 | 1 | 1 |
| Nominal Evaporative Capacity, Total | 0 | 1.67 tons/hr | 1.67 tons/hr |

# 2. CAPITAL IMPROVEMENTS PLAN

As part of the development of the WWTF Master Plan, a capital improvements projects (CIP) list was developed. The CIP list includes repair and replacement (R&R) projects identified during the existing condition assessments documented in TM1, recommended improvements to various process areas identified in the treatment process evaluations documented in TM2, recommended improvements to provide flow equalization facilities at the WWTF as documented in TM3, and projects identified by City staff.

The proposed CIP projects that are anticipated to be required to rehabilitate, improve, and expand the City of Hendersonville WWTF during the planning period through 2040 are listed in **Table 2.1** below. The CIP list includes the project number, project name, a general description of the work, project start year, estimated cost, and total project cost with inflation. The total project cost with inflation was included to account for inflation on projects with a programmed start date beyond 2022. A 3% annual inflation factor was applied, starting in 2023, for projects programmed to start in 2023 or later.

The estimated capital costs presented in the CIP project list were prepared as Class 4 cost estimates per the definitions of AACE International, and all cost estimates are presented in September 2021 dollars. Various factors may combine to result in cost fluctuations within the range of accuracy for Class 4 cost estimates including fluctuations in market conditions, changes in project scope, improved project definition, value engineering, and selection of alternative processes, equipment, or technologies. The design criteria and capital costs of the recommended improvements are recommended to be revisited and updated regularly to capture changes in facility needs and market conditions prior to project conception to allow for budgets to be updated appropriately.

Working with the City, the projects were prioritized and scheduled over the 20-year planning period. The priority and schedule are based on several factors, including criticality, expected remaining equipment and facility life/condition, facility operations and need to maintain continued facility operation and capacity, ability to meet regulatory requirements, correlation with other impacted process improvements, and the need to distribute costs over the planning period. Most capital improvement projects with an estimated capital cost greater than $1,000,000 include a design project in year "N", with the associated construction of that project in year "N+1". Where engineering, legal, and administrative costs are not broken out as a separate design cost, they are included as part of the overall project costs presented. There is also the potential to group several projects into larger projects, related by either process and/or physical location should the City choose to do so.

Many listed projects have been discussed in greater detail in the previous Technical Memoranda and sections of this Master Plan. Several of these projects have been expanded in the CIP list to include replacement of ancillary equipment and facility repairs, primarily due to age or remaining useful life. As a result, the project cost estimates presented in the various technical memoranda of the Master Plan may be less than the cost included in the CIP list.

*Table 2.1 – CIP Projects for the Hendersonville WWTF*

| Project Name | Needed for Expansion to 6.0 MGD? | Project Type[1] | Project Start | Estimated Project Cost | Estimated Project Cost with Inflation | Description of General Work |
|---|---|---|---|---|---|---|
| UV Disinfection Improvements – Design | ✓ | Capacity/Replacement | 2022 | $393,000 | - | Design of new UV disinfection system between existing disinfection channel and utility building with clear-span shelter structure, new filtered effluent electromagnetic flow meter, meter vault, filtered effluent piping modifications, connection to the existing NPW pump wet well, lightning protection system, and other associated equipment and appurtenances. Maintain use of existing UV disinfection system and replace existing fiberglass grating. |
| Clarifier Drive Mechanism Replacement | | Replacement | 2022 | $137,000 | - | Replace secondary clarifier drive mechanisms per Evoqua inspection report and quote. |
| Aeration Basin Rehabilitation – Design | | Rehabilitation | 2022 | $312,000 | - | Perform engineering analysis of boweing/deflection in aeration basin #2 north wall to develop repair recommendations, survey aeration basin #2 north wall to measure and monitor deflection, perform engineering analysis of aeration basins to verify structural integrity and develop repair plans. |
| Biosolids Thermal Drying System – Design | | Risk Reduction | 2022 | $1,757,000 | - | Replace biosolids storage shelter roof, install new protective coatings on structural steel members, and convert portion of storage shelter to new thermal drying facility structure. New dewatered cake conveyor to new thermal drying facility. New thermal drying facility including sludge feed hopper, medium-temperature gas-fired thermal belt dryer, dried biosolids conveyor, dried biosolids product storage silo and truck load-out station, and all associated equipment and appurtenances. |
| Sludge Thickening Rehabilitation and TWAS Storage – Design | | Rehabilitation/Risk Reduction | 2022 | $767,000 | - | Repair cracks in existing gravity thickener #1 and install interior/exterior coating system, equipment manufacturer inspect gravity thickener mechanical and drive mechanisms and provide rehabilitation recommendations, replace belt filter press feed pumps, relocate isolation valves on thickened sludge suction piping, rehabilitate/rebuild existing gravity thickener mechanical and drive mechanisms, install interior coating systems in gravity thickener #2, repair cracks in thickening building and install new steel beams to support roof slab (if required). Construct two new 50 foot diameter aerated thickened WAS storage tanks prior to dewatering. Repurpose existing belt filter press feed pumps to pump TWAS to the new aerated TWAS storage tanks. Modify TWAS suction piping to the existing pumps to allow any of the three existing pumps to withdraw TWAS from either gravity thickener. |
| UV Disinfection Improvements – Construction | ✓ | Capacity/Replacement | 2023 | $2,407,000 | $2,479,210 | Construction of new UV disinfection system between existing disinfection channel and utility building with clear-span shelter structure, new filtered effluent electromagnetic flow meter, meter vault, filtered effluent piping modifications, connection to the existing NPW pump wet well, lightning protection system, and other associated equipment and appurtenances. Maintain use of existing UV disinfection system and replace existing fiberglass grating. |
| Aeration Basin Rehabilitation – Construction | | Rehabilitation | 2023 | $1,685,000 | $1,735,550 | Repair aeration basin #2 north wall bowing/deflection following recommendations of engineering analysis, repair cracks in faces of exterior walls following recommendations of engineering analysis, repair cracks in walkway slabs and top of walls. Repairs to aeration basin #2 north wall expected to include installation of new concrete buttresses and extension to the aeration basin base slab with wooden pile foundation. |

| Project Name | Needed for Expansion to 6.0 MGD? | Project Type[1] | Project Start | Estimated Project Cost | Estimated Project Cost with Inflation | Description of General Work |
|---|---|---|---|---|---|---|
| Headworks Improvements and Flow Equalization – Design | ✓ | Capacity | 2023 | $4,247,000 | $4,374,410 | Repair cracks in exterior top of existing wet well wall, replace influent flow measurement equipment, replace wet well level measurement equipment, repair cracks in walls and slabs, perform ventilation system improvements. Replace the existing influent pumps with larger pumps to accommodate anticipated peak wet weather flows, replace the existing 16-inch force mains with 24-inch diameter force mains, and redirect all influent flow to a new inline flow equalization basin. Expand the existing influent pump station to include new mechanical screening upstream of all pumps, construction of a second, interconnected wet well with additional pumps. Construction of a new mechanically induced vortex grit removal system upstream of new flow equalization facilities. Construction of a new 3.0 MG inline flow equalization basin with associated jet aeration and mixing equipment, and other associated equipment and appurtenances. |
| Septage Receiving Improvements | | Rehabilitation | 2023 | $538,000 | $554,140 | Removal and replacement of existing concrete pavement, curb, and gutter. Removal and replacement of existing asphalt access drive to septage receiving area. Improvements to septage receiving area discharge/drain piping and replacement of existing discharge manhole. New concrete platform for existing septage receiving equipment. |
| Biosolids Thermal Drying System – Construction | | Risk Reduction | 2024 | $9,474,000 | $10,050,967 | Replace biosolids storage shelter roof, install new protective coatings on structural steel members, and convert portion of storage shelter to new thermal drying facility structure. New dewatered cake conveyor to new thermal drying facility. New thermal drying facility including sludge feed hopper, medium-temperature gas-fired thermal belt dryer, dried biosolids conveyor, dried biosolids product storage silo and truck load-out station, and all associated equipment and appurtenances. |
| Sludge Thickening Rehabilitation and TWAS Storage – Construction | | Rehabilitation/Risk Reduction | 2024 | $4,134,000 | $4,385,761 | Repair cracks in existing gravity thickener #1 and install interior/exterior coating system, equipment manufacturer inspect gravity thickener mechanical and drive mechanisms and provide rehabilitation recommendations, replace belt filter press feed pumps, relocate isolation valves on thickened sludge suction piping, rehabilitate/rebuild existing gravity thickener mechanical and drive mechanisms, install interior coating systems in gravity thickener #2, repair cracks in thickening building and install new steel beams to support roof slab (if required). Construct two new 50 foot diameter aerated thickened WAS storage tanks prior to dewatering. Repurpose existing belt filter press feed pumps to pump TWAS to the new aerated TWAS storage tanks. Modify TWAS suction piping to the existing pumps to allow any of the three existing pumps to withdraw TWAS from either gravity thickener. |
| Headworks Improvements and Flow Equalization - Construction | ✓ | Capacity | 2025 | $22,900,000 | $25,023,448 | Repair cracks in exterior top of existing wet well wall, replace influent flow measurement equipment, replace wet well level measurement equipment, repair cracks in walls and slabs, perform ventilation system improvements. Replace the existing influent pumps with larger pumps to accommodate anticipated peak wet weather flows, replace the existing 16-inch force mains with 24-inch diameter force mains, and redirect all influent flow to a new inline flow equalization basin. Expand the existing influent pump station to include new mechanical screening upstream of all pumps, construction of a second, interconnected wet well with additional pumps. Construction of a new mechanically induced vortex grit removal system upstream of new flow equalization facilities. Construction of a new 3.0 MG inline flow equalization basin with associated jet aeration and mixing equipment, and other associated equipment and appurtenances. |

| Project Name | Needed for Expansion to 6.0 MGD? | Project Type[1] | Project Start | Estimated Project Cost | Estimated Project Cost with Inflation | Description of General Work |
|---|---|---|---|---|---|---|
| Tertiary Filter No. 2 Replacement – Design | ✓ | Capacity | 2025 | $246,000 | $268,881 | Replace tertiary filter #2 with a new AquaDiamond filter system, install clear span structure over tertiary filters, repair cracks in north wall. |
| Blower Building Improvements – Design | | Replacement/Rehabilitation | 2025 | $359,000 | $392,289 | Subsurface soils investigation, recoat blower discharge piping, repair/replace sidewalks, pipe supports, access stair framing, columns, footings, and roof framing (if required) following recommendations of subsurface soils investigation. Replace existing Hoffman multistage centrifugal blowers with new turbo blowers and VFDs. Conversion of existing blower building to an enclosed blower room with associated blower intakes, ancillary equipment, and appurtenances. |
| Tertiary Filter No. 2 Replacement – Construction | ✓ | Capacity | 2026 | $1,958,000 | $2,203,746 | Replace tertiary filter #2 with a new AquaDiamond filter system, install clear span structure over tertiary filters, repair cracks in north wall. |
| Sludge Dewatering Cake Conveyor Belt Replacement | | Replacement | 2026 | $845,000 | $951,055 | Replace dewatered cake conveyor belt, motor, drive mechanism, chain, rollers, and bearings. |
| Blower Building Improvements – Construction | | Replacement/Rehabilitation | 2026 | $1,936,000 | $2,178,985 | Subsurface soils investigation, recoat blower discharge piping, repair/replace sidewalks, pipe supports, access stair framing, columns, footings, and roof framing (if required) following recommendations of subsurface soils investigation. Replace existing Hoffman multistage centrifugal blowers with new turbo blowers and VFDs. Conversion of existing blower building to an enclosed blower room with associated blower intakes, ancillary equipment, and appurtenances. |
| Recycle Pumping Station Rehabilitation | | Replacement/Rehabilitation | 2026 | $967,000 | $1,088,367 | Replace RAS pump #2 and WAS pumps, RPS heating and ventilation system improvements, repair cracks in walls, slabs, and exterior top of walls, repair sidewalk settlement on west side of RPS to eliminate trip hazard from valve operating nuts. |
| Dewatering Facility Lightning Protection Improvements | | Risk Reduction | 2026 | $500,000 | $562,754 | Install grounding/lightning protection system on existing dewatering building. |
| Aeration Basin Improvements – Design | | Process Efficiency | 2029 | $264,000 | $324,687 | Conversion of existing extended aeration process to a Modified Ludzack-Ettinger process. Installation of submersible ultra-low-head high-flow type NRCY pumps with associated NRCY pipelines. Installation of a compressed gas mixing system in the first diffuser grid of each aeration basin. |
| Aeration Basin Improvements – Construction | | Process Efficiency | 2030 | $1,424,000 | $1,803,881 | Conversion of existing extended aeration process to a Modified Ludzack-Ettinger process. Installation of submersible ultra-low-head high-flow type NRCY pumps with associated NRCY pipelines. Installation of a compressed gas mixing system in the first diffuser grid of each aeration basin. |
| Belt Filter Press System Replacement – Design | | Replacement | 2030 | $281,000 | $355,962 | Replacement existing belt filter presses with two new 2.0 meter belt filter presses. Replace wash water supply system, polymer makedown skids, and other appurtenances and ancillary equipment. |
| Belt Filter Press System Replacement – Construction | | Replacement | 2031 | $1,514,000 | $1,975,427 | Replacement existing belt filter presses with two new 2.0 meter belt filter presses. Replace wash water supply system, polymer makedown skids, and other appurtenances and ancillary equipment. |
| Secondary Clarifier Rehabilitation – Design | | Rehabilitation | 2031 | $328,000 | $427,966 | Rehabilitate/rebuild existing clarifier mechanical and drive mechanisms, replace clarifier scum boxes, repair cracks in exterior walls. |
| Secondary Clarifier Rehabilitation – Construction | | Rehabilitation | 2032 | $1,767,000 | $2,374,700 | Rehabilitate/rebuild existing clarifier mechanical and drive mechanisms, replace clarifier scum boxes, repair cracks in exterior walls. |

| Project Name | Needed for Expansion to 6.0 MGD? | Project Type[1] | Project Start | Estimated Project Cost | Estimated Project Cost with Inflation | Description of General Work |
|---|---|---|---|---|---|---|
| 7.8 MGD Facility Expansion – Design | | Capacity | 2032 | $4,680,000 | $6,289,529 | Expansion of the existing WWTF to accommodate the predicted 2040 loading conditions. Addition of aeration basin No. 3, blower building No. 2, 90-ft diameter secondary clarifier No. 3, RAS/WAS pump station No. 2, flow distribution boxes, and tertiary filter No. 3. Replacement of the existing UV4000 system in UV Channel No. 1 with a new 15 MGD UV disinfection system to match the proposed equipment in UV Channel No. 2, replacement of the cascade reaeration steps, and replacement/expansion of the effluent outfall. |
| 7.8 MGD Facility Expansion – Construction | | Capacity | 2035 | $25,851,000 | $37,963,065 | Expansion of the existing WWTF to accommodate the predicted 2040 loading conditions. Addition of aeration basin No. 3, blower building No. 2, 90-ft diameter secondary clarifier No. 3, RAS/WAS pump station No. 2, flow distribution boxes, and tertiary filter No. 3. Replacement of the existing UV4000 system in UV Channel No. 1 with a new 15 MGD UV disinfection system to match the proposed equipment in UV Channel No. 2, replacement of the cascade reaeration steps, and replacement/expansion of the effluent outfall. |

*[1]Project types include: Capacity Improvements, Replacement, Rehabilitation, Process Efficiency Improvements, Risk Reduction*

# APPENDIX A:

## TECHNICAL MEMORANDUM No. 1 – PRELIMINARY EVALUATIONS AND CONDITION ASSESSMENTS

# CITY OF HENDERSONVILLE
## WASTEWATER TREATMENT FACILITY MASTER PLAN

## FINAL Technical Memorandum No.1 – Preliminary Evaluations and Condition Assessments

Date: February 23, 2021

**Prepared for:**


City of Hendersonville
305 Williams Street
Hendersonville, NC 28792


**Prepared by:**


McKim & Creed, Inc.
8020 Tower Point Dr.
Charlotte, NC 28227
Firm License No. F-1222


**McKim & Creed Project 06496-0009**

# CONTENTS

# TABLES



# FIGURES

# LIST OF ACRONYMS

| Abbreviation | Definition |
|---|---|
| AADF | Annual Average Daily Flow |
| ATAD | Autothermal Thermophilic Aerobic Digestion |
| ATC | Authorization to Construct |
| BFP | Belt Filter Press |
| $BOD_5$ | Biochemical Oxygen Demand, Five Day |
| CFR | Code of Federal Regulations |
| CIP | Capital Improvement Plan |
| C.I.P. | Cast-In-Place |
| CMMS | Computerized Maintenance Management System |
| CMU | Concrete Masonry Unit |
| CPU | Central Processing Unit |
| DC | Direct Current |
| DI | Ductile Iron |
| DIP | Ductile Iron Pipe |
| DMR | Discharge Monitoring Report |
| DMZ | Demilitarized Zone (computing) |
| DO | Dissolved Oxygen |
| EDI | Environmental Dynamics International |
| EPDM | Ethylene Propylene Diene Monomer |
| EQ | Equalization |
| FOG | Fats, Oils, and Grease |
| HMI | Human Machine Interface |
| HP | Horsepower |
| HVAC | Heating Ventilation and Air Conditioning |
| ICS | Instrumentation and Control System |
| INF | Influent |
| IP | Industrial Protocol |
| IPS | Influent Pumping Station |
| KPI | Key Performance Indicators |
| L | Liters |
| LED | Light-Emitting Diode |
| LIMS | Laboratory Information Management System |
| MCC | Motor Control Center |
| MG | Million Gallons |
| mg | milligram |
| MGD | Million Gallons per Day |
| MLSS | Mixed Liquor Suspended Solids |
| MOV | Motor Operated Valve |
| MPO | Metropolitan Planning Organization |
| NCDEQ | North Carolina Department of Environmental Quality |
| $NH_3$-N | Ammonia as nitrogen |
| $NO_3$-N | Nitrate as nitrogen |
| $NO_2$-N | Nitrite as nitrogen |
| NOAA | National Oceanic and Atmospheric Administration |
| NPDES | National Pollutant Discharge Elimination System |

| | |
|---|---|
| OIT | Operator Interface Terminal |
| ORP | Oxidation-Reduction Potential |
| PF | Peaking Factor |
| PHF | Peak Hourly Flow |
| PIN | Personal Identification Number |
| PLC | Programmable Logic Controller |
| PS | Pump Station |
| PVC | Polyvinyl Chloride |
| RAM | Random-Access Memory |
| RAS | Return Activated Sludge |
| RPM | Revolutions Per Minute |
| RPS | Recycle Pumping Station |
| RRVS | Resistor Reduced Voltage Starter |
| SCADA | Supervisory Control and Data Acquisition |
| SPD | Surge Protective Devices |
| SRT | Solids Retention Time |
| SS | Stainless Steel |
| SSAIA | Sanitary Sewer Asset Inventory and Assessment |
| SSO | Sanitary Sewer Overflow |
| SWD | Side Water Depth |
| TAZ | Traffic Analysis Zone |
| TKN | Total Kjeldahl Nitrogen |
| TN | Total Nitrogen |
| TP | Total Phosphorus |
| TS | Total Solids |
| TSS | Total Suspended Solids |
| USGS | United States Geological Survey |
| UV | Ultra-Violet |
| UVD | Ultra-Violet Disinfection |
| VAC | Volts, Alternating Current |
| VDC | Volts, Direct Current |
| VFD | Variable Frequency Drive |
| VSS | Volatile Suspended Solids |
| WAS | Waste Activated Sludge |
| WWTF | Wastewater Treatment Facility |
| WWTP | Wastewater Treatment Plant |

# 1. INTRODUCTION

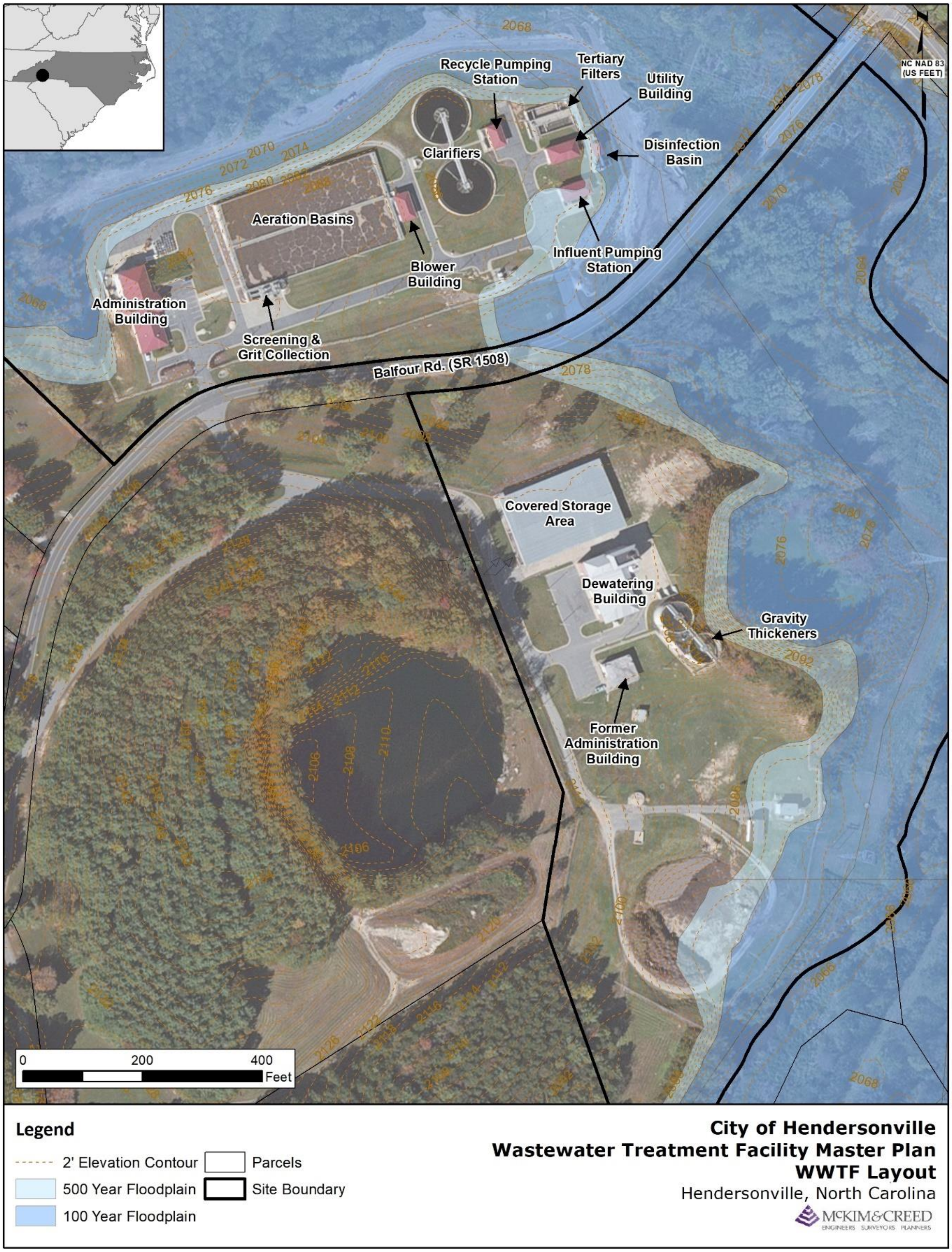
## 1.1 Background

The City of Hendersonville's Wastewater Treatment Facility (WWTF) was originally constructed in 1965 and was replaced in 2001 with a new WWTF located across the street from the original facility. The City's WWTF is located at 99 Balfour Road in Hendersonville, NC. The current WWTF consists of influent pumping with open channel flow measurement, screening and grit removal, an extended aeration activated sludge secondary treatment process, secondary clarification, tertiary cloth media filtration, UV disinfection, gravity thickening of waste activated sludge (WAS), belt filter press sludge dewatering, and dewatered sludge storage. The solids handling processes are located on the opposite side of Balfour Road at the site of the original WWTF. The WWTF layout is shown below in **Figure 1.1**. The WWTF is currently permitted for a design capacity of 4.8 MGD. The WWTF's current NPDES discharge permit also includes provisions for a future permitted capacity of 6.0 MGD upon issuance of an authorization to construct for expansion of the facility.

Since the commissioning of the current WWTF in 2001, there have been no studies conducted to evaluate the entire WWTF's existing conditions, treatment process performance, treatment process capacities, and future treatment capacity expansion needs. Recently, there have been several smaller scale studies and evaluations conducted, however these evaluations have been limited in scope and have not provided a holistic view of the entire facility's operations. Starting in 2017, the City invested in two significant improvement projects at the WWTF. The first project provided for the installation of a 1,500 kW diesel-driven emergency generator. The second project provided for the replacement of one of the two traveling bridge tertiary sand filters with an AquaDiamond cloth media filtration unit. Construction of both projects was completed in 2020, and they are fully operational. In addition to the significant improvement projects, the existing equipment has been regularly maintained and various pieces of equipment replaced on an as-needed basis.

In spite of maintenance efforts, most of the treatment equipment at the WWTF is approximately 20 years old and is approaching the end of the expected 20 to 30-year design life. When this occurs, the major processes and equipment should be evaluated for rehabilitation, replacement, and expansion needs to ensure the facility will continue to operate as intended.

*Figure 1.1 - Existing Wastewater Treatment Facility Layout*



## 1.2 Program Goals and Objectives

The City of Hendersonville contracted services with McKim & Creed to develop an overall WWTF master plan to address the concerns and uncertainty related to the age, condition, and capacity of the existing treatment process. The City of Hendersonville Wastewater Treatment Facility Master Plan is intended to provide a holistic review of the major systems throughout the facility, to provide recommendations for replacement, rehabilitation, upgrades, and treatment capacity expansion.

The overall objectives of the WWTF Master Plan include the following:

1. Review the current condition of existing processes, equipment, and major systems and provide recommendations for rehabilitation, replacement, and upgrades.
2. Compare the recommendations for rehabilitation, replacement, and upgrades to the currently planned capital improvement projects and prioritize needs.
3. Survey critical weir, pipe, and equipment elevations throughout the WWTF and prepare an updated hydraulic profile for the entire facility to be used as a baseline for future improvements.
4. Evaluate existing treatment process capacities and limitations; and evaluate the potential for a capacity re-rating of the existing facility.
5. Review future influent flow and constituent loading conditions through the 2040 design year. Evaluate treatment process modifications, upgrades, and expansion alternatives to increase the facility's treatment capacity to meet anticipated future conditions.
6. Review sizing and recommended locations for a new influent flow equalization basin, based on previous recommendations from the City's *Sanitary Sewer Asset Inventory and Assessment (SSAIA) Master Plan Report*.
7. Prepare a comprehensive plan for facility improvements to enhance current treatment operations, and to serve as a planning tool for future modifications, upgrades, and expansions to meet future wastewater treatment needs through the 2040 design year. Provide capital improvement planning recommendations for the immediate term and planning horizons of 2025, 2030, 2035, and 2040.

To accomplish these goals, this Master Plan has been organized into three (3) separate technical memorandums, that when combined, will form the basis of the comprehensive facility plan. This Technical Memorandum No. 1 is the first step in the comprehensive facility Master Plan, and will provide the following information:

1. A review of the previous WWTF flow projections and process capacity evaluations performed as part of the previous *SSAIA Master Plan Report*.

2. A summary of data collected to support the efforts of this Master Plan, and a summary of data to be used for treatment process modeling efforts in the following phases of the Master Plan.
3. A summary of currently planned capital improvement projects to date.
4. The review and condition assessment of the existing WWTF infrastructure, treatment processes, and major process equipment; and recommendations for repairs, rehabilitation, replacement, and future evaluations.

# 2. SUMMARY OF PREVIOUS ENGINEERING STUDIES

This section provides a summary of the previous *Sanitary Sewer Asset Inventory and Assessment Master Plan Report* and the *Process Capacity Assessment and Plant Expansion Addendum to the SSAIA Master Plan Report*. This review documents the previous influent wastewater flow projections presented in the SSAIA Master Plan, which will serve as the basis for the future design conditions evaluated in this Master Plan. This review also summarizes the findings of the previous process capacity and plant expansion assessment for reference.

## 2.1 City of Hendersonville Sanitary Sewer Asset Inventory and Assessment (SSAIA) – Master Plan Report

The SSAIA Master Plan Report developed flow projections for the base year (2017) and future planning years for 2025 and 2040. Base year flow to the WWTF was determined by analysis of historical flow data recorded at the facility. The future planning year flow projections were developed from the following:

- Historical plant flows.
- Spatially distributed traffic analysis zone (TAZ) polygons from Land of Sky Regional Council that include population and employment projections.
- 2010 and 2040 French Broad River MPO (FBRMPO) TAZ projection data from Land of Sky Regional Council.
- City of Hendersonville 2017 Water System Master Plan Report.
- Areas of historically failing septic systems provided by Seth Swift, Environmental Health Supervisor with the Henderson County Board of Health.
- Private wastewater treatment plant (WWTP) flows: http://www.epa.gov/enviro/facts/pcs-icis/search.html.
- Industrial and commercial development areas provided by the City partnership.
- Historical precipitation data from the United States Geological Survey (USGS).
- Historical precipitation data from the National Oceanic and Atmospheric Administration (NOAA).
- Henderson County 2020 Comprehensive Plan.
- The City of Hendersonville's 2030 Comprehensive Plan.

- Stakeholder meeting on April 27, 2018 that included Town of Laurel Park, Henderson Co. Partnership for Economic Development, and Henderson County Schools.

The historical wastewater flow rates were combined with data from the City of Hendersonville's 2017 *Water System Master Plan Report*, data from other City of Hendersonville and Henderson County planning studies (listed above), historical precipitation data to account for inflow and infiltration (I/I), and input received from community stakeholders served by the sewer system. Traffic analysis zone (TAZ) data was used to determine potential flow increases from population and employee growth, while additional future flows were considered from the elimination of private wastewater treatment plants, conversions of failing and hazardous septic systems to public sewers, and the addition of future industrial customers.

The base year (2017) flow was determined to be 3.07 million gallons per day (MGD) based on a five (5) year average of average annual daily flows from 2013 to 2017. Because the WWTF permitted capacity is based on maximum monthly flow, a 30-day rolling average for the same five (5) year period was used to determine the maximum monthly flow resulting in a maximum monthly peaking factor of 1.30. This peaking factor was used for the base year and the future planning years.

The projected flow increases for the future planning years (2025 and 2040) were added to the base flow to determine the future average annual flows to the WWTF. Flows were projected to increase linearly. **Table 2.1** below illustrates the flow projections made in the master plan report.

*Table 2.1 – Master Plan Flow Projections*

| COH Sewer Service Area | 2017 | 2025 | 2040 |
|---|---|---|---|
| Average Annual Flow Projections (MGD) | 3.07 | 4.23 | 5.90 |
| Maximum Month Total Flow Projections (MGD)[1] | 4.00 | 5.50 | 7.68 |

*[1]Based on 5 year average maximum month peaking factor (PF) of 1.30.*

These flow projections, as made by the City of Hendersonville SSAIA – Master Plan Report, shall be used throughout this report as the basis for discussing flows to the WWTF.

## 2.2 City of Hendersonville Process Capacity Assessment and Plant Expansion Addendum to the SSAIA Master Plan Report

The *Process Capacity Assessment and Plant Expansion Addendum to the SSAIA Master Plan Report* evaluated the City of Hendersonville WWTF's current influent flows, loads, treatment performance, secondary treatment process capacity, and recommended operational changes and further investigations. The assessment also discussed plant expansion alternatives and recommendations for flow equalization. A summary of the primary findings of the previous assessment is presented below.

- Noted City has limited peak flows to the facility to 6.5 MGD to avoid solids washout from clarifiers.

- Recommended adding TKN to the influent monitoring program to determine the total nitrogen load to the facility.
- Noted sludge blankets averaged approximately 2 ft, with some instances above 3 ft, and up to as high as 8ft. Recommended sludge blankets be kept below 1 ft on average, and 1.5 ft at peak flows to prevent solids washout.
- Recommended reducing MLSS concentration in the aeration basins based on the required SRT to maintain nitrifying conditions. Modeled results indicated a 9.6 day aerobic SRT and minimum MLSS of 1,920 mg/L to maintain nitrifying conditions at a liquid temperature of 20 degrees Celsius.
- Prepared state point analysis of existing clarifiers based on design conditions with MLSS concentration of 1,920 mg/L per the modeling, and 4,300 mg/L per current operations.
    - Determined that the existing solids loading rate on the clarifiers is too high and is causing solids washout at flows above the average design flow rate.
- Provided several alternatives for future plant expansion to 7.7 MGD design capacity based on the projected 2040 maximum month flow.
    - Alternative 1 – Addition of primary clarifiers
    - Alternative 2 – Addition of conventional process treatment train
    - Alternative 3 – Process intensification
    - Alternative 4 – New granular activated sludge treatment train.

The previous equalization basin sizing was based on the modeled impacts of a 2-year design storm on the City's collection system. The 2-year design storm was used as the basis for the recommendations included in the SSAIA Master Plan Report. Within the model, the existing WWTF's hydraulic capacity of 12 MGD was exceeded by the 17.4 MGD (theoretical) instantaneous peak flow from the 2-year storm, which results in an estimated volume of 0.95 MGD that exceeded the facility's existing capacity. A 2-year storm scenario for the future planning year of 2040, was also considered in the model assuming a permitted facility capacity of 9 MGD.

The assessment recommended the addition of an equalization (EQ) tank with a minimum volume of 1.0 MG to address current needs. It also suggested that a 6 MG EQ tank be constructed to keep the peaking factor at 2.5 based on the design maximum flow of 7.68 for the planning year 2040. However, the assessment concluded that the need for additional storm flow EQ should be re-assessed during the design of the next facility expansion. **Table 2.2** below illustrates the modeled 2-year storm scenarios and the estimated requisite storm EQ volume for the WWTF.

*Table 2.2 - Equalization Basin Sizing Calculations from Process Capacity Assessment*

| Year | 2-Year Storm Peak Flow (MGD) | Permitted Treatment Plant Flow (MGD) | Plant Hydraulic Capacity (MGD) (PF=2.5) | Storm EQ Volume Needed (MG) |
|---|---|---|---|---|
| 2017 | 17.4 | 4.8 | 12 | 0.95 |
| 2040 | 39.4 | 9 | 22.5 | 5.74 |

The values shown in **Table 2.2** shall be used as the basis for determining the equalization volume required for the WWTF. However, ultimate design recommendations may change due to differing assumptions and/or the capacity effects of recommended facility expansion alternatives.

# 3. DATA COLLECTION

Data was assessed for wastewater influent and effluent characteristics, operational data, and effluent requirements based on the current facility permit. Previously reported facility data and projections for future facility influent loading for the 20-year planning period were confirmed or amended.

## 3.1 Operational Permits

The City WWTF maintains the following operational permits outlined in **Table 3.1:**

*Table 3.1 – Operational Permits*

| Permit | Permit # | Issued | Expires |
|---|---|---|---|
| National Pollutant Discharge Elimination System (NPDES) | NC0025534 | 12/11/2017 | 12/31/2022 |
| Residuals Class A Distribution | WQ0011381 | 2/1/2020 | 9/30/2025 |

Under the current NPDES permit, the WWTF has a permitted capacity of 4.8 MGD with the ability to upgrade the facility to 6.0 MGD upon issuance of an Authorization to Construct (ATC) for plant expansion. The permit imposes limits on the effluent characteristics. These limits include 5-day biochemical oxygen demand ($BOD_5$), total suspended solids (TSS), ammonia as nitrogen ($NH_3$-N), dissolved oxygen (DO), fecal coliform, total copper, and chronic toxicity. The permit also requires that the City monitor and report other effluent characteristics including pH, nutrient concentrations, dissolved metals concentrations, temperature, and total hardness. The nutrients include total phosphorus (TP) and total nitrogen (TN), and the dissolved metals include nickel, silver, and copper; all of which are sampled quarterly. No mercury limitation is required by the permit. The NDPES permitted effluent limits are listed in **Table 3.2** below.

*Table 3.2 - NDPES Permit Effluent Limits*

| | Limits – 4.8 MGD | | Limits - 6.0 MGD | |
|---|---|---|---|---|
| Effluent Characteristics | Monthly Average | Weekly Average | Monthly Average | Weekly Average |
| $BOD_5$, (Summer: between April 1 and October 31) | 10.0 mg/L | 15.0 mg/L | 10.0 mg/L | 15.0 mg/L |
| $BOD_5$, (Winter: between November 1 and March 31) | 20.0 mg/L | 30.0 mg/L | 20.0 mg/L | 30.0 mg/L |
| TSS | 30.0 mg/L | 45.0 mg/L | 30.0 mg/L | 45.0 mg/L |
| $NH_3$-N, (Summer: between April 1 and October 31) | 2.0 mg/L | 6.0 mg/L | 2.0 mg/L | 6.0 mg/L |
| $NH_3$-N, (Winter: between November 1 and March 31) | 4.0 mg/L | 12.0 mg/L | 4.0 mg/L | 12.0 mg/L |

| | | | | |
|---|---|---|---|---|
| Dissolved Oxygen (DO) | Daily Average ≥ 5.0 mg/L | | Daily Average ≥ 5.0 mg/L | |
| Fecal Coliform (geometric mean) | 200/100 mL | 400/100 mL | 200/100 mL | 400/100 mL |
| Total Copper (µg/L) | Monitor and Report | | 37.5 µg/L | 42.8 µg/L |
| Chronic Toxicity | *Ceriodaphnia dubia* Pass/Fail at 18% influent | | *Ceriodaphnia dubia* Pass/Fail at 21% influent | |

The Residuals Class A Distribution permit has been modified such that permitted tonnage distribution of Class A residuals is zero (0) dry tons per year. The City renewed this permit to prepare for future solids management improvements at the WWTF that would allow them to produce Class A residuals. The City previously produced Class A lime stabilized biosolids at the WWTF, however, according to facility staff this process was discontinued due to a lack of interest in the product by local farmers, and excessive odor complaints from nearby property owners. Once plans to produce Class A biosolids at the WWTF are finalized, the City may apply for a major permit modification to add the new treatment processes and increase the facility's permitted dry tonnage.

## 3.2 Influent Flow Data

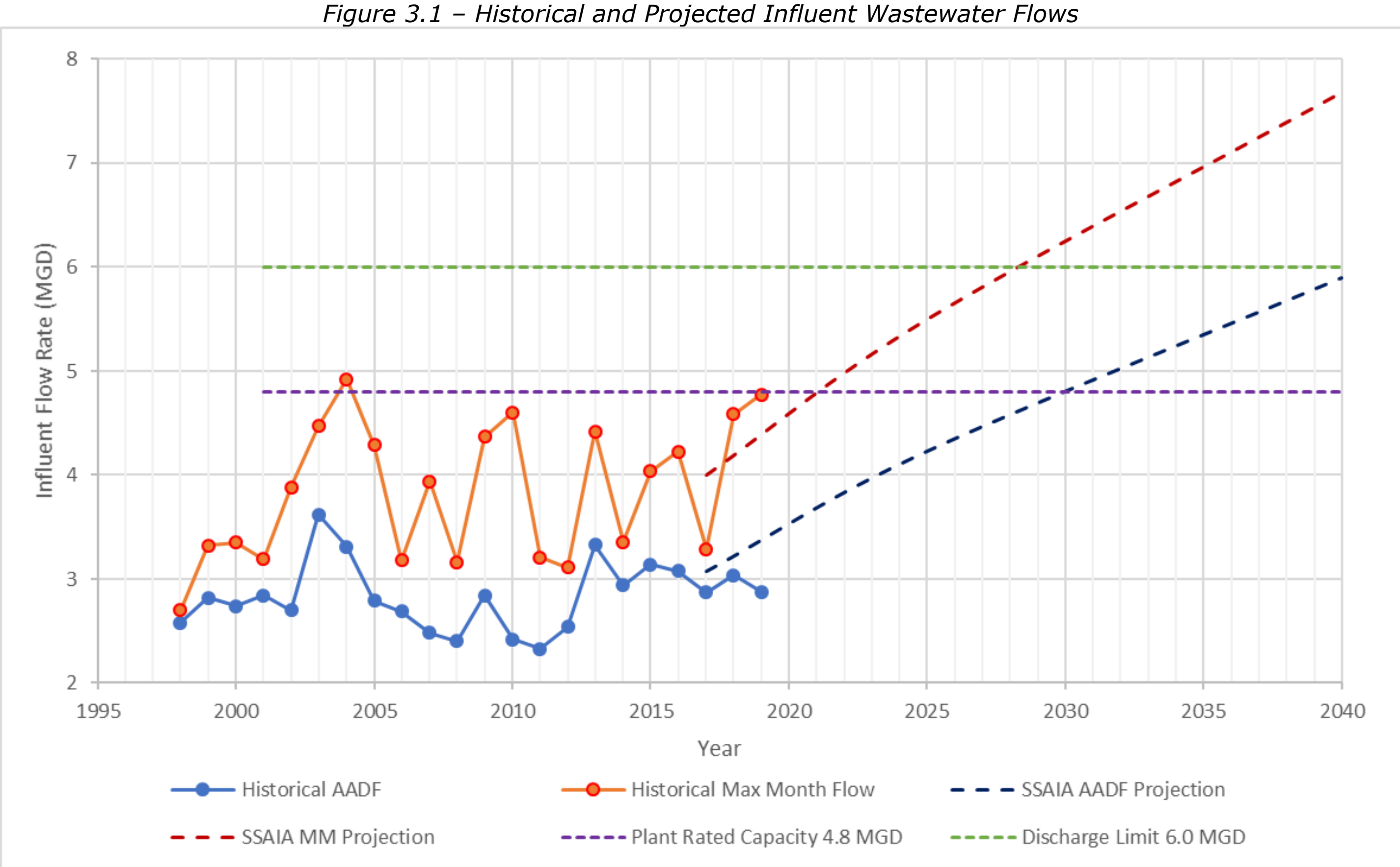
Daily influent flow data was analyzed over a five (5) year period from January 2014 to December 2019. The influent flow data is presented below in **Table 3.3**.

*Table 3.3 – Average Influent Flow Data*

| Parameter | Units | Average |
|---|---|---|
| Average Influent Flow | MGD | 2.99 |
| Summer Average Influent Flow | MGD | 2.82 |
| Winter Average Influent Flow | MGD | 3.29 |
| Maximum Month Flow | MGD | 4.78 |
| Maximum Month PF | - | 1.60 |
| Maximum Day Flow | MGD | 6.28 |
| Maximum Day PF | - | 2.10 |
| 2014 AADF | MGD | 2.94 |
| 2015 AADF | MGD | 3.14 |
| 2016 AADF | MGD | 3.08 |
| 2017 AADF | MGD | 2.87 |
| 2018 AADF | MGD | 3.04 |
| 2019 AADF | MGD | 2.88 |

From the review of the influent flow data it is observed that Annual Average Daily Flows (AADF) over the recent five (5) year period from 2014 to 2019 followed a declining trend. An overall increasing trend was observed with respect to the maximum month flows over this same five (5) year period from 2014 to 2019. The historical influent flow data from 1998 to 2019 was compared to the influent flow projections from the SSAIA Master Plan Report as shown in **Figure 3.1**. Historical flow data from 1998 to 2013 shown in the figure below was gathered from the SSAIA Master Plan Report for comparison of historical trends to the projected influent flows. As seen below, influent flows have historically been relatively constant, with very little change in AADF over the past 20 years. Historical maximum month flows have varied significantly year over year, as is expected based on the correlation to yearly rainfall totals. However, the average trend for maximum month flows over the last 20 years has also been relatively constant.

*Figure 3.1 – Historical and Projected Influent Wastewater Flows*




The influent flow projections presented in the SSAIA Master Plan report were prepared using traditional methods and appear to have included a comprehensive analysis of potential impacts on future influent flows. It was noted that the influent flow projections were most influenced by future population and industrial growth projections. The historical influent flow trends for both AADF and maximum month flows have increased at a much slower rate than what has been projected from 2017 through 2040. This indicates a potential significant difference between historical and projected population and industrial growth rates

The flow projections provided in the SSAIA Master Plan Report will continue to be used throughout this Master Plan. However, the difference between actual and projected growth rates seen above may have a significant impact on the timing of future facility expansion needs. As such, future facility upgrades and expansion needs identified through this Master Plan should reference both the future planning year and expected influent flow rate at which they are needed. This approach will provide the City the flexibility to adapt and revise the resulting WWTF Capital Improvement Plan based on the actual growth rate of influent flows to the facility.

## 3.3 Influent and Effluent Water Quality Data

Influent and effluent water quality data was analyzed from January 2014 to December 2019. This data was gathered from monthly NPDES permit Discharge Monitoring Reports (DMRs) and analyzed to establish the following:

- Determine average historical influent constituent concentrations and loading rates
- Gauge historical treatment performance based on effluent water quality
- Establish future influent constituent concentrations and loading rates to be used over the 20-year planning period.

Per the facility's NPDES permit, the City is required to sample and record the major water quality constituents shown below in **Table 3.4**. It is important to note that the NPDES permit does not require the City to monitor influent concentrations of TKN, $NH_3$-N, or TP. These influent parameters are critical for the development of a representative wastewater process model.

*Table 3.4 – NPDES Permit Sampling Requirements*

| Parameter | Sample Location | Reported Units | Sampling Frequency | Sample Type |
|---|---|---|---|---|
| $BOD_5$ | Influent | mg/L | 5x/week | Composite |
| TSS | Influent | mg/L | 5x/week | Composite |
| Temperature | Effluent | °C | 5x/week | Grab |
| pH | Effluent | s.u. | 5x/week | Grab |
| $BOD_5$ | Effluent | mg/L | 5x/week | Composite |
| TSS | Effluent | mg/L | 5x/week | Composite |
| $NH_3$-N | Effluent | mg/L | 5x/week | Composite |
| Fecal Coliform | Effluent | #/100mL | 5x/week | Grab |
| DO | Effluent | mg/L | 5x/week | Grab |
| TN | Effluent | mg/L | Quarterly | Calculated from composite samples for TKN, $NO_3$-N, and $NO_2$-N |

| | | | | |
|---|---|---|---|---|
| TP | Effluent | mg/L | Quarterly | Composite |

### 3.3.1 Influent Wastewater Characteristics

The average influent $BOD_5$ and TSS data is presented in **Table 3.5** below. Summer and winter averages were calculated in addition to the yearly averages due to the differing summer and winter effluent limits for $BOD_5$ and $NH_3$-N. As shown below, summer averages for influent $BOD_5$ and TSS were 10.3% and 16.5% higher than winter averages. The average influent $BOD_5$ and TSS concentrations indicate that the wastewater collected within the City of Hendersonville's service area is primarily average strength domestic wastewater. However, data for nutrient concentrations such as TKN, $NH_3$-N, and TP have not been historically monitored, therefore, an assessment of influent nutrient loading conditions was not possible. It is recommended that additional monitoring be performed at this time, and as part of regular operations going forward, to quantify influent nutrient loading conditions by regularly monitoring influent TKN, $NH_3$-N, and TP.

*Table 3.5 – Average Influent Water Quality Data 2014-2019*

| Parameter | Units | Value |
|---|---|---|
| Average $BOD_5$ | mg/L | 219 |
| Average TSS | mg/L | 223 |
| Summer Average $BOD_5$ | mg/L | 224 |
| Summer Average TSS | mg/L | 233 |
| Winter Average $BOD_5$ | mg/L | 203 |
| Winter Average TSS | mg/L | 200 |

### 3.3.2 Effluent Wastewater Characteristics

As shown above, the facility's NPDES permit requires monitoring data for effluent water quality, including most major nutrient parameters, physical parameters, metals concentrations, and organic loading. A summary of the major constituents of concern for the effluent water quality is shown below in **Table 3.6**. It is important to note that the NPDES permit does not currently impose effluent limits for total copper. However, the permit does include total copper limits of 37.5 μg/L (monthly average) and 42.8 μg/L (weekly average) for the future 6.0 MGD design capacity. Historical quarterly monitoring data for total copper has consistently averaged well below 20 μg/L. Therefore, this future permit limit is not expected to cause any compliance issues.

*Table 3.6 – Average Effluent Water Quality Data 2014-2019*

| Parameter | Units | Average | Summer | Winter |
|---|---|---|---|---|
| $BOD_5$ | mg/L | 5.31 | 5.40 | 5.12 |
| TSS | mg/L | 5.90 | 5.95 | 5.79 |

| | | | | |
|---|---|---|---|---|
| Temperature | °C | 18.0 | 20.7 | 14.2 |
| pH | - | 6.88 | 6.94 | 6.59 |
| Ammonia as N ($NH_3$-N) | mg/L | 0.56 | 0.56 | 0.55 |
| Dissolved Oxygen (DO) | mg/L | 7.20 | 6.74 | 7.84 |
| Total Nitrogen (TN)[1] | mg/L | 6.94 | 5.66 | 7.98 |
| Total Phosphorus (TP)[1] | mg/L | 2.29 | 2.38 | 2.18 |

*[1]Data collected from 2/6/2017 to 11/18/2019.*

### 3.3.3 Influent Concentrations for Process Modeling

As indicated earlier in this section, data for influent TKN, $NH_3$-N, TN, and TP was not available from the NPDES permit DMRs. Additional quarterly laboratory data was collected from the City to establish a better understanding of the potential influent nutrient concentrations. Laboratory analysis data for influent $NH_3$-N and TP was provided by the City. This information is summarized in **Table 3.7**, below. It is important to note that influent TKN concentrations were not reported in the quarterly laboratory analysis data. TKN is the sum of organic nitrogen and $NH_3$-N. Therefore, the influent wastewater TKN concentration can be estimated using a typical assumption for the percentage of $NH_3$-N that makes up TKN. Raw domestic wastewater's $NH_3$-N content is typically 50 to 60% of the wastewater's TKN content.

*Table 3.7 – Quarterly Influent $NH_3$-N and TP Data*

| Influent $NH_3$-N mg/L | Influent TP mg/L | Date Collected |
|---|---|---|
| 12.5 | - | 03/18/2019 |
| - | 4.4 | 03/19/2019 |
| 13.6 | - | 05/13/2019 |
| - | 3.8 | 05/14/2019 |
| 14.0 | 4.7 | 08/13/2017 |
| 17.0 | 6.8 | 11/19/2019 |
| 17.8 | 7.7 | 02/25/2020 |
| 15.2 | 4.1 | 05/05/2020 |

Average influent wastewater concentrations for key constituents are established in **Table 3.8** below for use in the development of the BioWin (EnviroSim Associates Ltd., Canada) wastewater process model. It is noted that key assumptions have been made regarding influent TKN and TP concentrations due to the small number of samples collected from the quarterly laboratory data. The assumptions made for TKN and TP concentrations are based on typical wastewater profiles observed at other treatment facilities treating average strength domestic wastewater. Current influent $NH_3$-N concentrations appear to be quite low and not consistent with typically observed values for domestic wastewater. Therefore, it is recommended that future planning be performed based on influent TKN, $NH_3$-N, and TP values typical of average strength domestic wastewater. The information presented below will be used as the basis for influent wastewater characterization in the BioWin wastewater process model throughout the 20 year planning period.

*Table 3.8 – Average Influent Wastewater Concentrations for Process Modeling*

| Parameter | Units | Value |
|---|---|---|
| $BOD_5$ | mg/L | 219 |
| TSS | mg/L | 223 |
| VSS | mg/L | 156 |
| TKN | mg/L | 45 |
| TP | mg/L | 7 |

## 3.4 Process Data

WWTF process data from the secondary biological treatment process was analyzed from January 2018 to December 2019. The data is presented in **Table 3.9.**

*Table 3.9 – Process Data*

| Parameter | Units | North Average | South Average |
|---|---|---|---|
| TSS – Clarifier Effluent | mg/L | 7.32 | 7.57 |
| MLSS – Aeration Basins | mg/L | 4,318 | 4,274 |
| VSS – Aeration Basins | mg/L | 3,056 | 2,988 |
| SVI[1] | - | 63.70 | 63.51 |
| Alkalinity[1] | mg/L as $CaCO_3$ | 72.74 | 74.62 |
| Clarifier Depth of Sludge Blanket | ft | 2.10 | 2.12 |

*[1]Measurement taken in aeration basins.*

## 3.5 Sludge Production

WWTF sludge production estimates were updated in the City of Hendersonville Solids Management Plan Evaluation Report. The assumptions used to complete the WWTF mass balance include a gravity thickener solids capture rate of 90%, a dewatering belt filter press capture rate of 95%, dosing polymer at a rate of 20 lbs per dry ton of sludge, and a sludge solids concentration (%TS) of 0.8% from the clarifiers which is used to estimate waste activated sludge (WAS). This solids concentration was estimated assuming both clarifiers are typically in service, an aeration basin MLSS concentration of approximately 4,000 mg/L, an average influent flow rate of approximately 3.2 MGD, and a return activated sludge (RAS) flow rate of 3.2 MGD. Sludge production data is shown in **Table 3.10**.

*Table 3.10 – Sludge Production for 2020 AADF Conditions*

| Flow Stream | Solids Concentration (%) | Flow (gpd) | Mass (Dry lb/d) |
|---|---|---|---|
| WAS from Clarifiers | 0.80% | 85,400 | 5,700 |

| | | | |
|---|---|---|---|
| Thickened Sludge from Gravity Thickeners | 3.36% | 18,200 | 5,200 |
| Supernatant from Gravity Thickeners | 0.1% | 67,200 | 500 |
| Polymer to Belt Filter Press Feed Sludge | 10% | 62 | 52 |
| Dewatered Sludge from Belt Filter Press | 17% | 3,500 | 5,000 |
| Filtrate from Belt Filter Press | 0.2% | 14,762 | 252 |

## 3.6 Electrical Usage

Duke Energy provides electricity to the WWTF based on the Optional Power Service, Time of Use with Voltage Differential (OPT-V) rate schedule. The electricity bill is broken out into a base facility charge, a demand charge per kW, and an energy charge per kWh. The rates for energy charges ($/kWh) change based on when the facility uses electricity during the week. During the summer (June 1 – Sept. 30), on-peak hours are from 1:00 pm – 9:00 pm Monday through Friday; during the winter (Oct. 1 – May 31), they are from 6:00 am – 1:00 pm, Monday through Friday. All other hours are classified as off-peak hours, which have an energy charge rate that's roughly half the on-peak energy charge rate. The on-peak rates for the demand charge ($/kW) in the winter are roughly half the rates for the summer.

Thirty-five (35) months of electrical usage data for the WWTF was gathered from May 2017 to March 2020 with an average annual electricity bill of $266,877. Monthly energy usage, cost, and cost per kWh are illustrated in **Table 3.11** below.

*Table 3.11 – WWTF Monthly Electrical Usage and Cost*

| Basis | Energy Usage (kWh / Month) | Cost ($ / Month) | Charge Rate ($ / kWh) |
|---|---|---|---|
| Annually | 370,886 | $22,240 | $0.060 |
| Summer (June 1 – Sept 30) | 377,629 | $25,168 | $0.067 |
| Winter (Oct. 1 – May 31) | 367,368 | $20,715 | $0.056 |

## 3.7 Chemical Usage

The WWTF currently uses only one chemical throughout all its processes. Polymer is added to the waste activated sludge (WAS) just upstream of the belt filter presses (BFP) to improve dewaterability. The polymer product, Clarifloc C-6265, provided by Polydyne Inc., is added in accordance with the solids processing schedule, which currently consists of running the BFP approximately two (2) to seven (7) days per month, typically operating 12 hours per day. Although historical polymer use records are not available, recent data show that polymer is applied at rates near 15 lbs per dry ton of gravity thickened WAS fed to the BFPs. Chemical cost and usage information are illustrated in **Table 3.12** below.

Disinfection at the WWTF is achieved by ultra-violet disinfection (UVD), therefore the facility does not perform chlorination and dechlorination or use any of the associated chemicals.

*Table 3.12 – Chemical Usage*

| Chemical | Process | Product Name | Usage Rate | Unit Cost |
|---|---|---|---|---|
| Polymer | Belt Filter Press | Clarifloc C-6265 by Polydine, Inc. | 15 lb/dry ton sludge[1] | $1.24 / lb |

*[1]Polymer is delivered in liquid form. Lb refers to a lb of liquid mixture. Historical polymer usage data is not available. The usage rate is based on minimal data and should should only be used as an approximate value.*

## 3.8 Equipment Asset Management Data

Data was collected for major process equipment throughout the Wastewater Treatment Facility (WWTF) for incorporation into the City of Hendersonville's asset management system (Cityworks®). The major equipment data attributes for collection include the process area, equipment type, manufacturer, model, head and design flow (if applicable), horsepower, voltage, RPM, drive, installation or replacement year, and expected useful life remaining. This data was gathered while on-site at the WWTF as well as from available equipment cutsheets, capacity curves, and operation and maintenance (O&M) manuals provided by the City.

Equipment data was collected for each process area, including the Influent Pumping Station, screening and grit collection, aeration basins, secondary clarifiers, recycle pumping station, effluent filters, UV disinfection system, the plant Utility Building, gravity thickeners, and the dewatering building. Refer to Appendix A for a complete list of WWTF equipment and all existing available equipment datasheets. If equipment data is unavailable, the equipment list includes an explanatory note.

# 4. CURRENT CAPITAL IMPROVEMENT PLAN

The City's current Capital Improvement Plan (CIP) was reviewed to identify currently planned projects at the WWTF and determine the purpose of each project. The information gathered regarding currently planned capital improvement projects will be compared to the findings of the site condition assessments to verify the need for each project. The currently planned capital improvement projects will be prioritized along with any recommended improvements identified during the site condition assessments. This prioritization will provide a baseline understanding of current facility needs for consideration during evaluation of proposed facility upgrade and expansion alternatives in later phases of this Master Plan. The currently planned future capital improvement projects for the City WWTF are summarized in **Table 4.1**.

*Table 4.1 – Planned Capital Improvement Projects*

| Project | Description | Year | City Allocated Funding |
|---|---|---|---|
| WWTF Aeration Basin #1 Diffusers Replacement | Replace aeration diffuser membranes in Aeration Basin #1 | 2020 | $43,170* |
| WWTF Renovation Project | Various rehabilitation projects | 2021 | $1,370,000 |
| WWTF UV Disinfection System | Replace existing UV Disinfection system | 2023 | $1,794,000 |
| WWTF Sludge Drying System | Reduce landfill costs and increase sludge disposal options | 2024 | $4,109,000 |
| WWTF EQ Basin | 6.0 MG EQ Basin | 2024 | $6,090,000 |
| WWTF 6.0 MGD Expansion | Expand WWTF capacity to 6.0 MGD | 2025 | $5,000,000 |

Note: *Project is completed

## 4.1 WWTF Aeration Basin #1 Diffusers Replacement

The aeration diffuser membranes in Aeration Basin #1 were approximately 20 years old and were in need of replacement due to excessive wear and poor performance. The City contracted with EDI Aeration Works to replace the diffuser membranes in Aeration Basin #2 with new EDI FlexAir® 84P EPDM diffusers, and to inspect the supports, piping, and joints for the aeration equipment in March 2020. The diffuser membranes in Aeration Basin #1 were replaced as planned in November 2020. Each basin includes 450 tubular fine bubble diffusers, with five (5) separate diffuser grids per basin, and 90 diffusers per grid. Each diffuser consists of a PVC membrane support tube, and an EPDM tubular membrane installed over the PVC support tube with stainless steel pipe clamps at each end of the tube to secure the membrane.

## 4.2 WWTF Renovation Project

The WWTF Renovation Project involves rehabilitating various systems and replacing old and deficient equipment. The project extends the useful life of existing systems and addresses safety and compliance issues. The project includes:

- Replacing two clarifier drives that are over 20-years old

- Rehabilitating two grit collectors
- Rehabilitating grit collector spare parts
- Replacing the grit auger and drive unit that are over 20-years old
- Rehabilitating the Influent Pumping Station external stairwell to address safety and access issues
- Replacing the mechanical bar screen that is over 20-years old
- Upgrading the Influent Pumping Station ventilation systems to address safety and compliance issues
- Extending the useful life of the sludge drying shed with rehabilitation and painting
- Extending the useful life of the administration building by rehabilitating the roof
- Extending the useful life of the administration building by replacing the HVAC system that is over 20-years old

## 4.3 WWTF UV Disinfection System

The existing UV disinfection system is slated for replacement due to aging equipment and treatment limitations. The City has experienced frequent component failures, mostly lamp ballast failures, and operational issues due to the age of the existing Trojan UV4000 system. The equipment manufacturer has discontinued production of this equipment, and parts availability is limited and expensive. The existing UV disinfection equipment is also much less efficient compared to newer UV disinfection technologies. A previous evaluation performed as part of the tertiary filter replacement project discovered that the existing UV disinfection equipment is hydraulically limited to a peak hourly flow rate of 12 MGD. Future flows above 12 MGD will result in excessive headloss through the existing UV disinfection equipment. The excessive headloss at these flow rates will submerge the effluent weirs of the tertiary filters and prevent adequate flow through the tertiary filters. The previous evaluation recommended replacement of the existing UV disinfection system with construction of a new UV disinfection channel. This project is expected to include the construction of the new disinfection channel between the existing channel and the existing Utility Building. The new UV disinfection system is expected to be the Trojan UVSigna system, or equal.

## 4.4 WWTF Sludge Drying System

Construction of a new biosolids thermal drying facility is planned for FY 2023 based on the recommendations of the *Solids Management Plan Evaluation Report*. The City currently produces unstabilized biosolids that do not meet the requirements of the 40 CFR Part 503 regulations for land application of biosolids and have no value for beneficial reuse. The WWTF sludge is currently processed by gravity thickening and belt filter press dewatering to produce a dewatered biosolids cake with approximately 16% TS content by mass. The dewatered biosolids are currently hauled from the WWTF to the White Oak Landfill in Haywood County, NC at a rate of $56/wet ton.

Landfill disposal of the City's biosolids has become unreliable in recent years. Several landfills have abruptly refused to accept any cake solids moving forward, while others have significantly increased tipping fees. The increases in tipping fees seen in the western NC region follows the national trend of increasing tipping fees, as published by the Environmental Research & Education Foundation. The national average tipping fee per ton for municipal solid waste landfills has increased from $48.27/ton in 2016 to $55.36/ton in 2019.

The *Solids Management Plan Evaluation Report* recommended that the City construct a new thermal drying facility at the WWTF to produce Class A-EQ biosolids at approximately 90% TS following thermal drying. The construction of the new thermal drying facility is recommended to include the following:

- Partial conversion of the existing covered storage area to a new thermal drying structure
- Installation of dewatered cake conveyors and live bottom hopper for dryer feed storage
- Installation of a medium-temperature belt dryer
- Installation of a dried product conveyance system to storage (belt and/or screw conveyors)
- Installation of dried product storage silos (or hoppers) and truck load-out station

## 4.5 WWTF Equalization Basin

According to the *Process Capacity Assessment and Plant Expansion Addendum to the SSAIA Master Plan Report*, a 2-yr storm EQ volume of 0.95 MG is required for current system flows. The required 2-yr storm EQ volume will increase to 5.74 MG by 2040 according to the model cited in the report. Installation of a 6.0 MG EQ tank would reduce the 2040 peak flow to the plant to a 2.5 peaking factor based on the modeled 2-year storm event and prevent sanitary sewer overflows (SSO's). Under current operations, facility operating staff must occasionally limit the influent pumping rate during wet weather conditions due to treatment process limitations. This forces influent wastewater to surcharge in the wastewater collection system. This operation may result in future SSO's in the collection system. Installation of flow equalization facilities was previously proposed to correct this issue.

## 4.6 WWTF 6.0 MGD Expansion

The WWTF currently has a permitted capacity of 4.8 MGD. The current NPDES permit has also already established permitted effluent limits for a future 6.0 MGD design capacity, upon issuance of an Authorization to Construct (ATC) for facility expansion plans. According to the *Process Capacity*

*Assessment and Plant Expansion Addendum to the SSAIA Master Plan Report*, the maximum month flow is likely to surpass the current plant capacity by 2021. The projected maximum month flow for the future planning year (2040) is 7.7 MGD. The report recommended four alternatives to expand the WWTF capacity from 4.8 MGD to 7.7 MGD including:

- Addition of primary clarifiers
- Addition of conventional process treatment train
- Process intensification
- Addition of new treatment train with aerobic granular sludge

# 5. EXISTING CONDITION ASSESSMENTS

Condition assessments of existing major processes, process equipment, structures, electrical equipment and systems, and instrumentation and control systems were performed at the wastewater treatment facility. Three (3) one-day facility walk-throughs were conducted with lead engineers from civil, process, structural, electrical, and instrumentation and controls disciplines to evaluate and document existing conditions. The purpose of these existing condition assessments is to identify repair, rehabilitation, and replacement needs for the WWTF to help the City plan and budget for these needs. The needs identified from these assessments will be considered when evaluating future facility upgrades and expansions in later phases of this Master Plan. The age, reliability, redundancy status, and condition of existing equipment, processes, systems, and structures were reviewed based on the findings from the facility walk-throughs. Recommendations for repairs, rehabilitation, or replacement are described in the sections below and are summarized in **Section 6**.

## 5.1 Administration Building

### 5.1.1 Purpose and Description

The administration building at the City of Hendersonville WWTF serves multiple functions and is the main hub for most activities at the facility. The administration building houses the facility laboratory, office space for operator and laboratory staff, control/training room, maintenance shop, shop storage, locker rooms, restroom facilities, breakroom, and HVAC/mechanical equipment room. The laboratory space, offices, control/training room, locker rooms, restroom facilities, and break room are all located on the first floor of the administration building. The maintenance shop, shop storage, and HVAC/mechanical equipment room are located in the basement of the administration building, with garage door access on the north side of the building. All on-site laboratory analyses for regulatory compliance and process monitoring analyses are performed within the administration building laboratory by City staff. The control room within the administration building houses the WWTF's supervisory control and data acquisition (SCADA) system server and desktop operator interface. The WWTF operations staff are able to monitor all process controls for the liquid stream processes from the control room within the administration building.

### 5.1.2 Structural Condition Assessment

The existing Administration Building is a single-story structure with a "T"-shaped footprint and partial basement area below the center and north end of the building. The building construction includes cast-in-place (C.I.P.) reinforced concrete, concrete masonry units (CMU), structural steel and timber framing elements. Note the following:

- C.I.P. reinforced concrete construction observed included exterior basement walls, foundation slabs-on-grade, piers, columns, beams and elevated slabs supporting the first floor areas over the basement.
- Record drawings reviewed indicated the building foundation includes C.I.P. reinforced concrete footings and pile caps and grade beams supported by timber piles.
- Record drawings reviewed, and CMU construction observed included basement interior walls and first floor exterior and interior walls.
- Exterior and interior steel hollow shaped section trusses were observed at the main entry canopy and the foyer area. Record drawings reviewed indicated structural steel to be secondary framing items consisting of lintel beam assemblies over door and window exterior wall masonry openings.
- Record drawings reviewed indicated timber elements included timber trusses to support the standing seam metal roofing.

Regarding the C.I.P. construction observed note the following:

- Vertical surfaces of walls, columns and beams and horizontal bottom surfaces of beams appeared to be in good condition and were plumb, level and primarily free of surface defects.
- Top surfaces of basement area exposed slabs-on-grade appeared to be in good condition. Top surfaces appeared to be level and plumb except where sloped to drain as intended. However, linear cracks were observed in top surfaces of the concrete floor slab-on-grade at two locations; one in the mechanical room and one in the shop area. These cracks appeared to be hairline in width and varied in length. It did not appear the slab surfaces at each side of these cracks had deflected or settled differentially vertically (ref. **Figure 5.1** and **Figure 5.2**).
- Top surfaces of the elevated floor slab over the basement area appeared to be in good condition. Top surfaces appeared to be level and plumb and primarily free of surface defects. The floor slab finishes consisted of exposed concrete, vinyl composition tile or ceramic tiles. In rooms with tile floor finish, no defects were observed in the finishes.
- Top surfaces of First Floor exposed slabs-on-grade appeared to be in good condition with top surfaces appearing to be level and plumb. However, a linear crack approximately 1/8” wide was observed at the south side of the interior door to the Pretreatment Room. In addition, linear cracks were observed in top surfaces of the concrete floor slab tile finishes at two locations. An approximate 2’-0” long crack was observed at the north door to the Lab and an approximate 3’-0” long crack was observed at the south entryway to the Lab. Both cracks were oriented in the

east/west direction parallel to the width of the opening and both were approximately 1/16” in width (ref. **Figure 5.3** and **Figure 5.4**).

*Figure 5.1 – Mechanical Room Cracks in Slab-on-Grade*

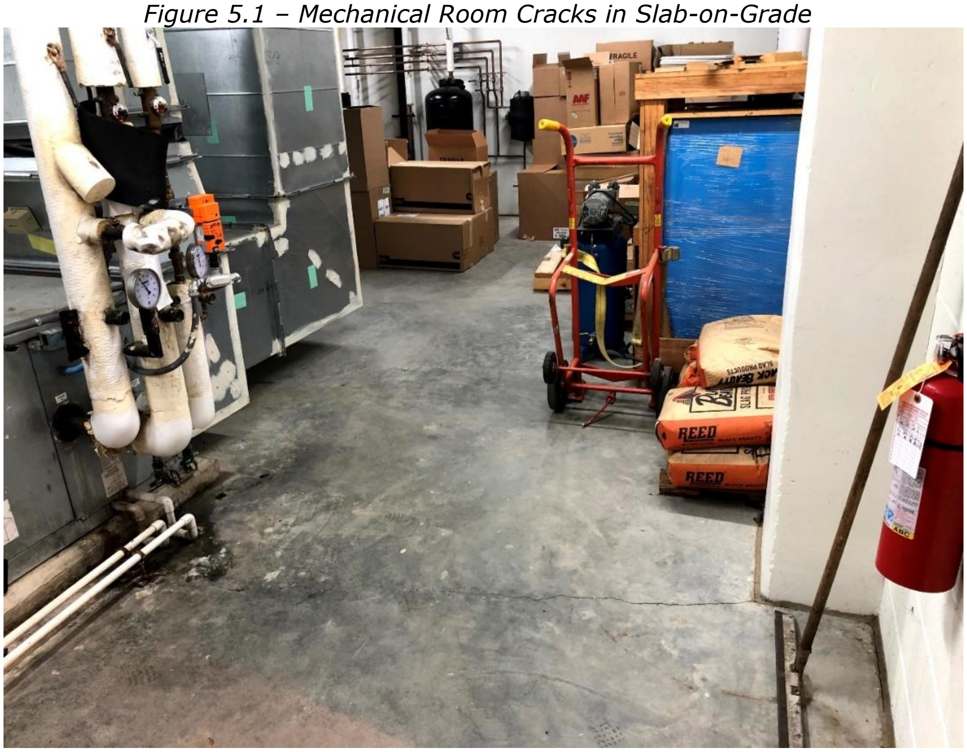


*Figure 5.2 – Shop Area Cracks in Slab-on-Grade*

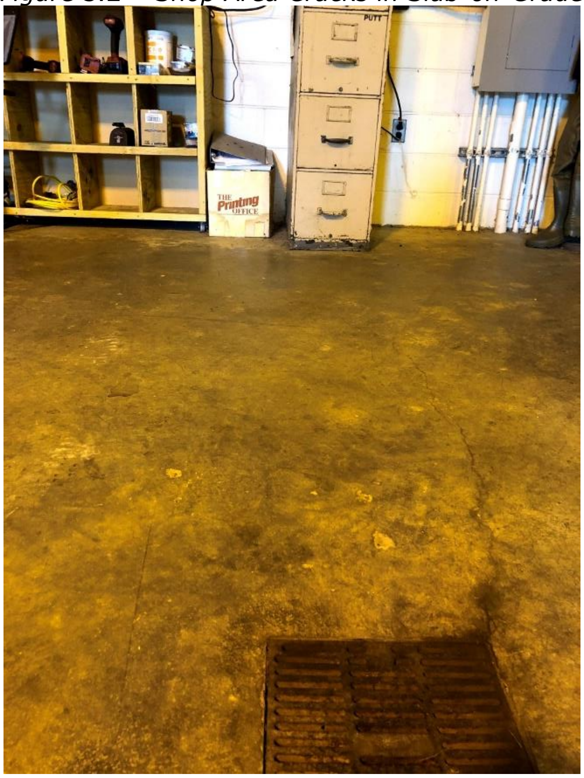


*Figure 5.3 – Laboratory North Entry Crack in Floor Tiles*

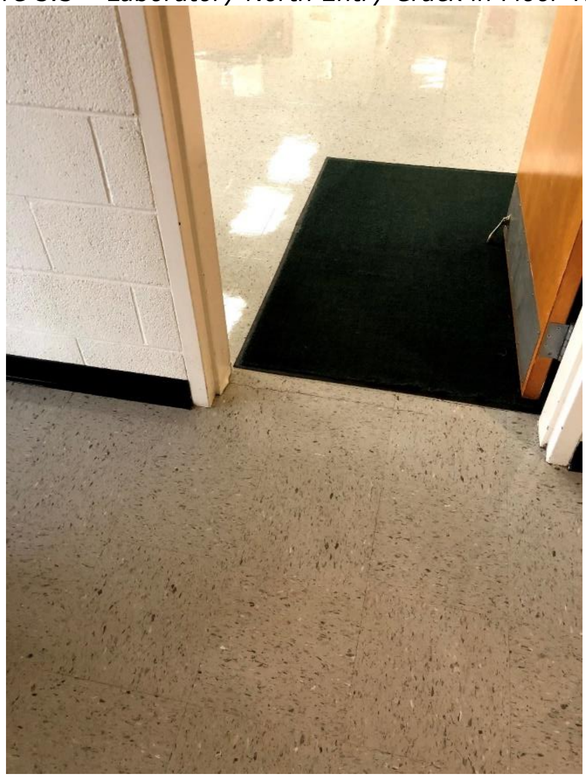


*Figure 5.4 – Laboratory South Entry Crack in Floor Tiles*

Regarding the CMU wall construction observed the walls were in poor condition. Note the following:

- Wall step crack patterns along mortar joint lines were observed at multiple exterior and interior wall locations. Many of the cracks observed were through wall cracks, meaning the cracks could be observed at both sides of the wall. However, at exterior walls existing veneer prevented verification of this condition. Note the following locations:
    - Basement - interior wall common to the Stair and Shop areas (ref. **Figure 5.5**).
    - Basement – east end of interior wall common to the Shop Storage and Shop areas (ref. **Figure 5.6**).
    - Basement - east face of west wall near mid-span of the wall at the bottom of the wall in the Shop Storage Room (ref. **Figure 5.7**).
    - First Floor – north face of south wall, east end and adjacent to top of the interior window in the Program Coordinator Room (ref. **Figure 5.8**).
    - First Floor – west end of north wall and above door to the Janitor's Room (ref. **Figure 5.9**).
    - First Floor – east wall and above door to the Men's Room.
    - First Floor – north wall and above urinal in the Men's Room (ref. **Figure 5.10**).
    - First Floor – west (interior) face of east wall, north end and adjacent to north side of window in the Lab.
    - First Floor – north (interior) face of south wall, west end at top of wall and adjacent to interior wall common to Corridor in the Lab Storage Room / Office (ref. **Figure 5.11**).
    - First Floor – east (interior) face of west wall, north end at top of wall and adjacent to interior wall common to Lab in the Pretreatment Room (ref. **Figure 5.12**).

*Figure 5.5 – Stairs/Shop Area Common Wall*

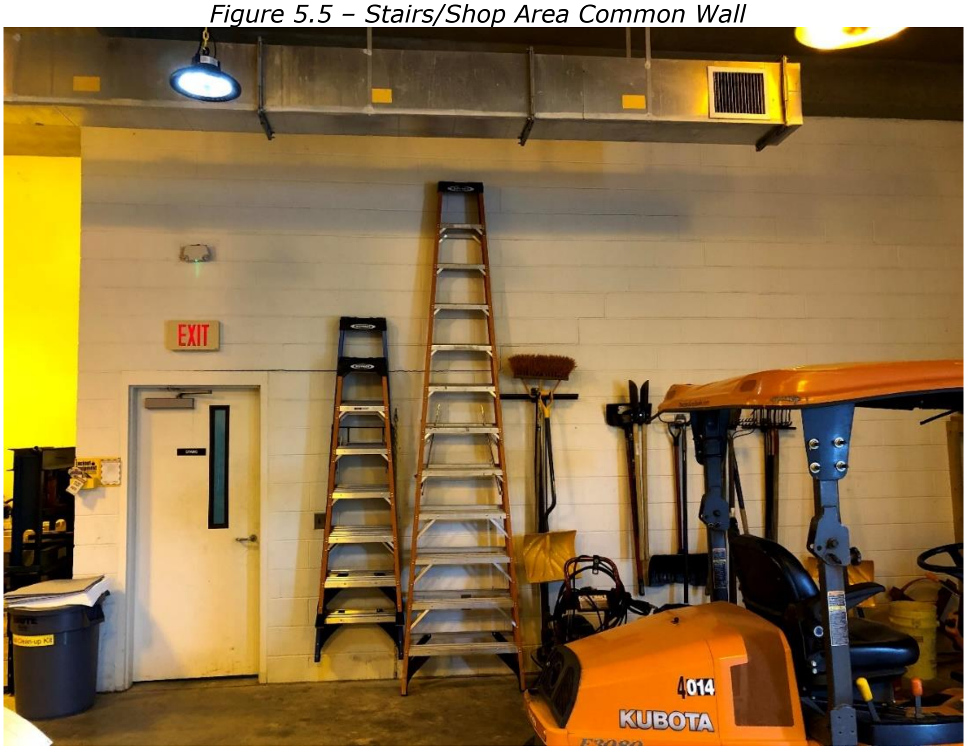



*Figure 5.6 – Shop/Storage Area Common Wall*

*Figure 5.7 – Shop Storage Area Crack in CMU Wall*

*Figure 5.8 – Program Coordinator Room Cracks in CMU Wall*

Figure 5.9 – Janitors Room Cracks in CMU Wall

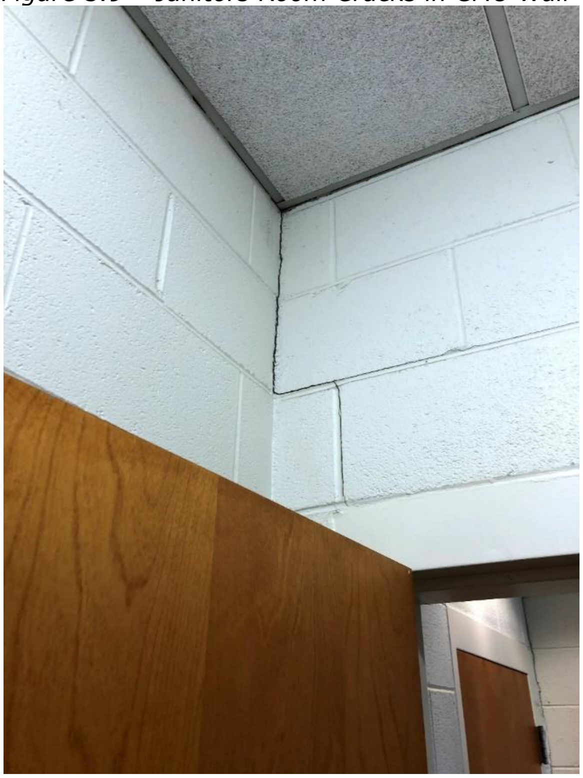


Figure 5.10 – Men's Bathroom Cracks in CMU Wall

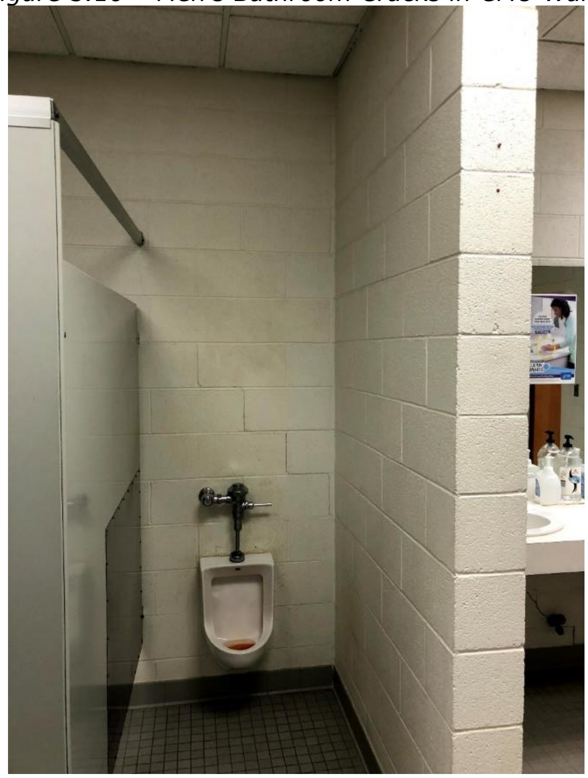


Figure 5.11 – Laboratory Office Cracks in CMU Wall

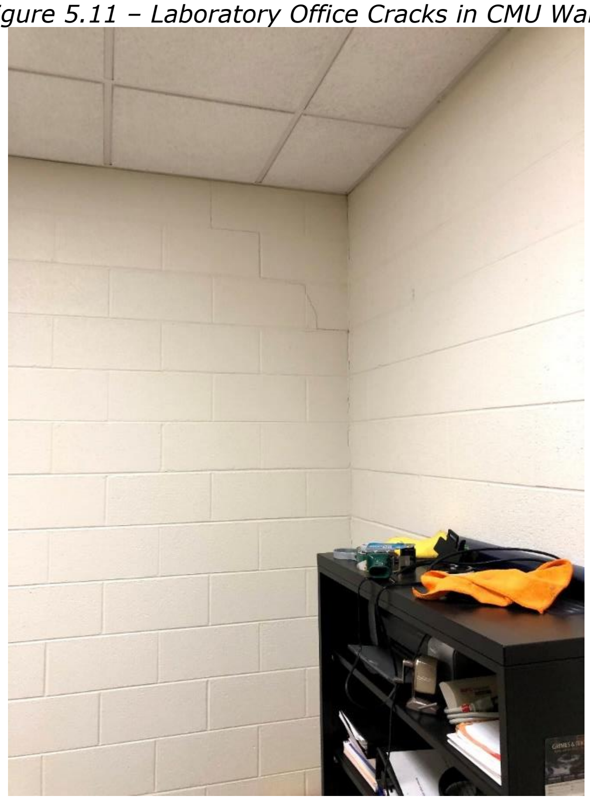
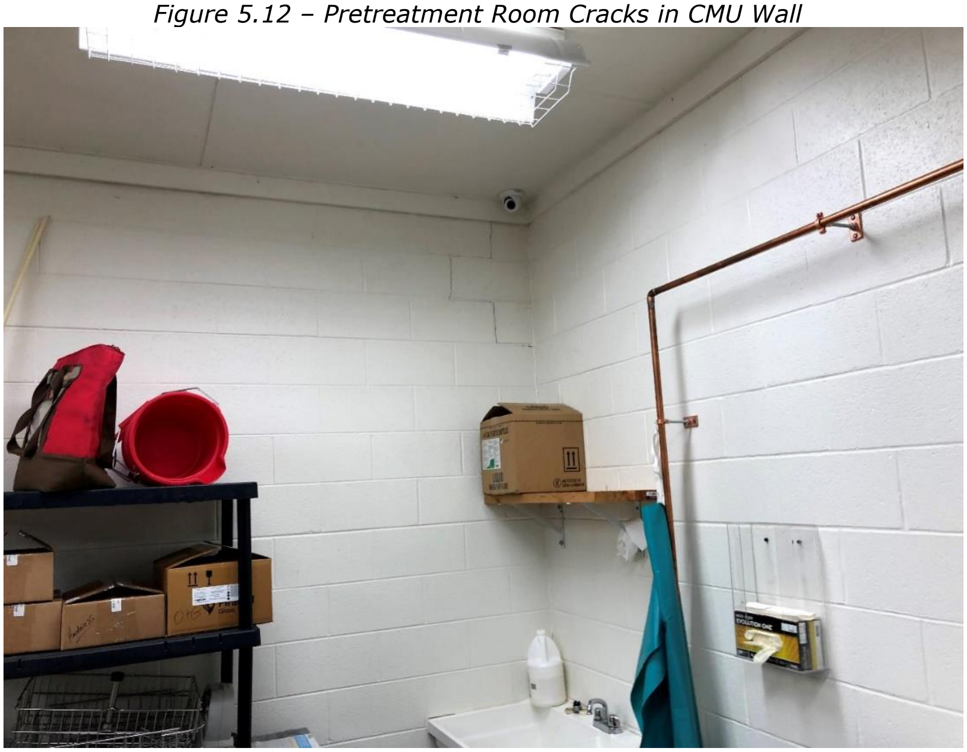


*Figure 5.12 – Pretreatment Room Cracks in CMU Wall*



- Vertical and/or horizontal crack patterns were observed at multiple exterior and interior wall locations, and some of these locations resulted in separation from adjacent walls and/or slabs-on-grade. Many of the interior cracks observed were through wall cracks, and similar to the previous section, the exterior wall existing veneers prevented verification of this condition. Note the following locations:
  - Basement - interior wall intersects for each corner of each wall of the Toilet Room and where the CMU wall abuts concrete column "B-2" (ref. **Figure 5.13**).
  - Basement – west exterior wall and separation from the floor slab-on-grade.
  - Basement – north end of the interior wall common to the Stair and Shop areas (ref. **Figure 5.14**).
  - Basement – high and low sections of the interior wall common to the Stair and Shop areas and the joint line with the existing "B6" concrete beam (ref. **Figure 5.15**).
  - First Floor – west end of the south wall intersection with the west wall in the upper landing of the Stair area.

- First Floor – east and west ends of the north wall intersection with the east and west walls in the Coat/Equipment Room (ref. **Figure 5.16**).
- First Floor – west end of the north wall intersection with the west wall in the Control Room (ref. **Figure 5.17**).
- First Floor – west end of the south wall intersection with the west wall in the Snack Room.
- First Floor – north and south ends of the east wall intersections and adjacent to the top corners of the window in the Chief Operator Room.
- First Floor – north end of the east wall intersection with the north wall in the Women's Room (ref. **Figure 5.18**).
- First Floor – west wall above the door to the Women's Locker Room in the Women's Room.
- First Floor – north wall of the Women's Room approximately 2'-10" east of the window (ref. **Figure 5.19**).
- First Floor – east end of the south wall intersection of the east wall of the Women's Locker Room.
- First Floor – east end of the south wall intersection with the south wall in the Janitor's Room (ref. **Figure 5.20**).
- First Floor – south end of the west wall above the entryway opening to the Men's Room in the Foyer.
- First Floor – south end of west wall of the Men's Room and north of and above the entry door to the Men's Locker's room (ref. **Figure 5.21**).
- First Floor – north end of the short partition wall west of the entry door from the Foyer to the Lab. The separation gap observed was approximately ¾" wide at the top of the wall (ref. **Figure 5.22**).
- First Floor – south end of the east wall of the Pretreatment Room.

*Figure 5.13 – Shop Restroom Vertical Crack*

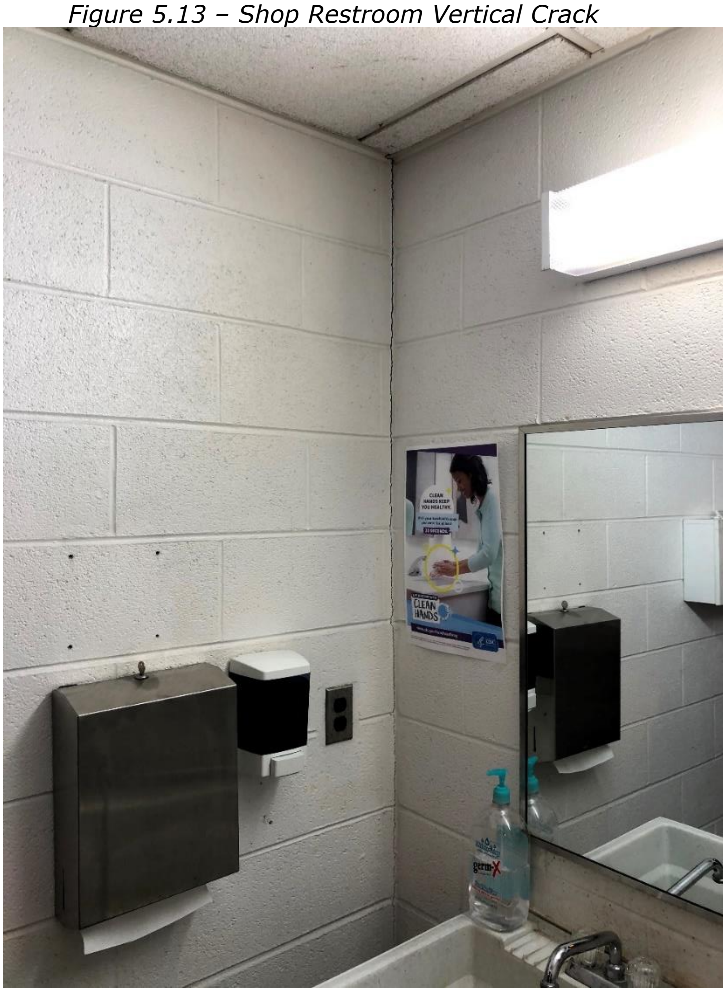



*Figure 5.14 – Stairwell Vertical and Horizontal Cracks*

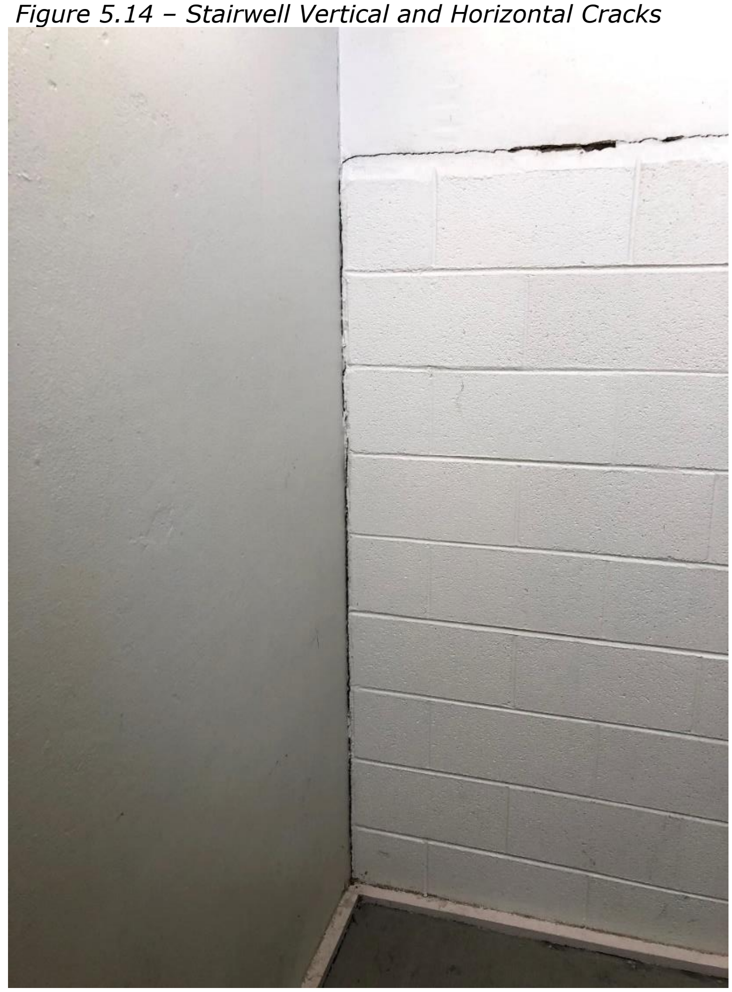


*Figure 5.15 – Stairwell Horizontal Crack Along Concrete Beam*

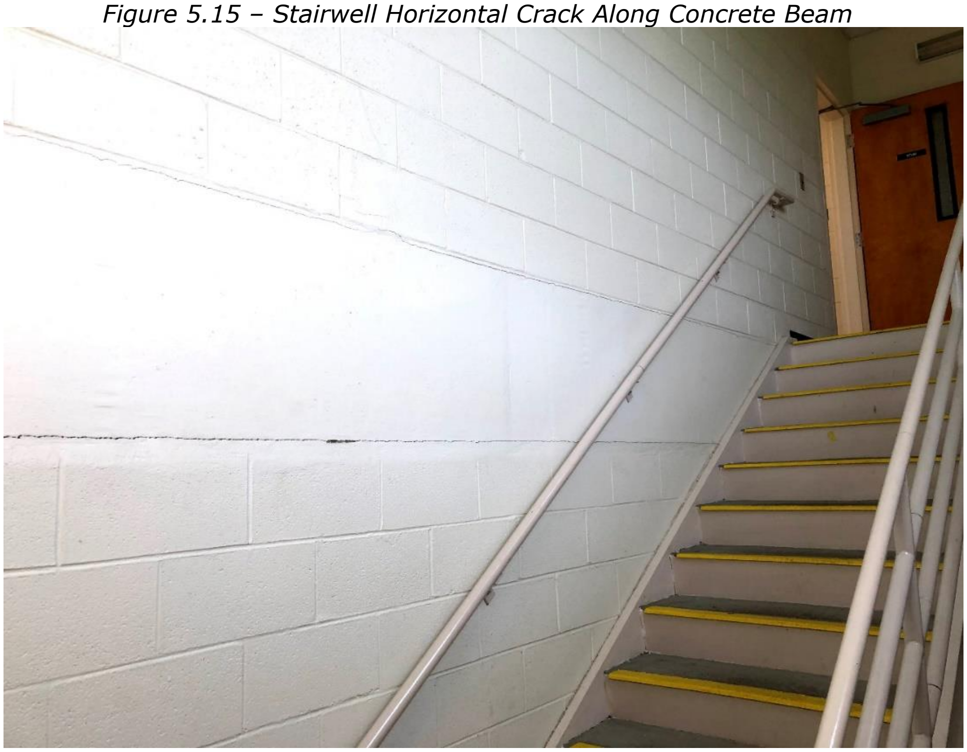


*Figure 5.16 – Coat/Equipment Room Vertical Crack*

*Figure 5.17 – Control Room Vertical Crack*

*Figure 5.18 – Women's Room Vertical Crack*

*Figure 5.19 – Women's Room Vertical Crack*

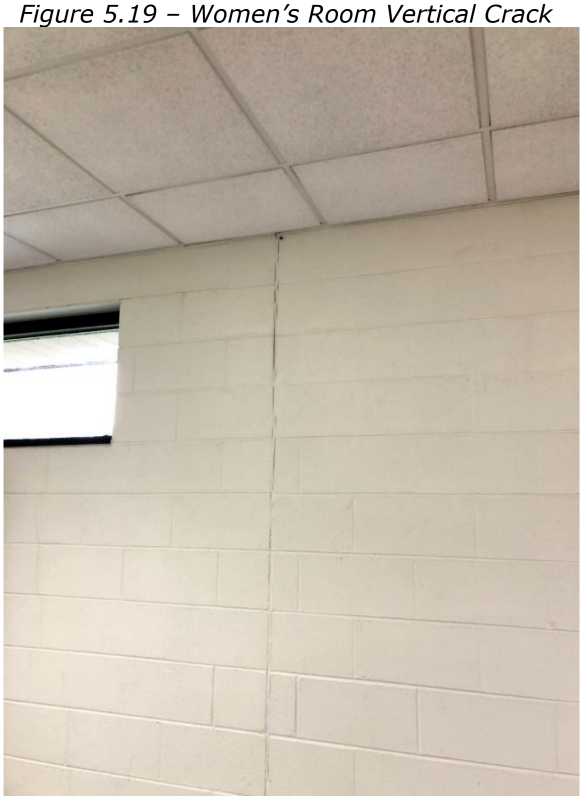


*Figure 5.20 – Janitors Room Vertical Crack*

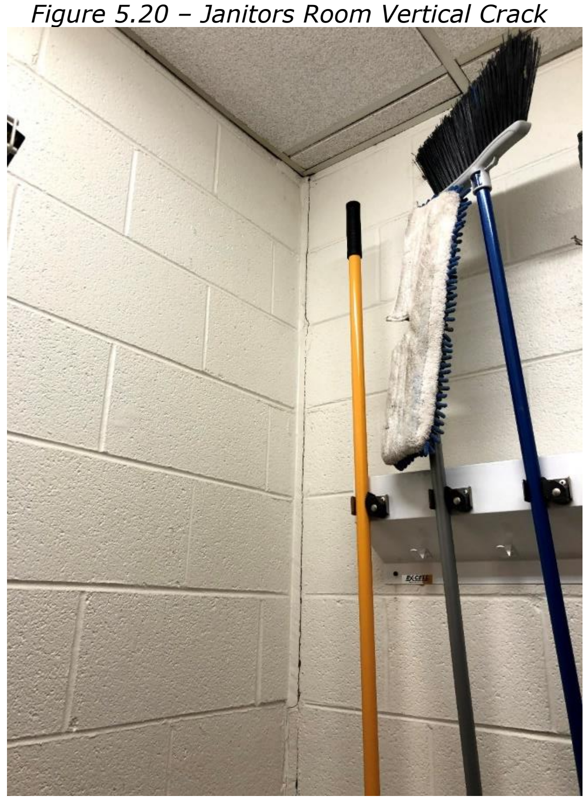


*Figure 5.21 – Men's Room Horizontal Crack*

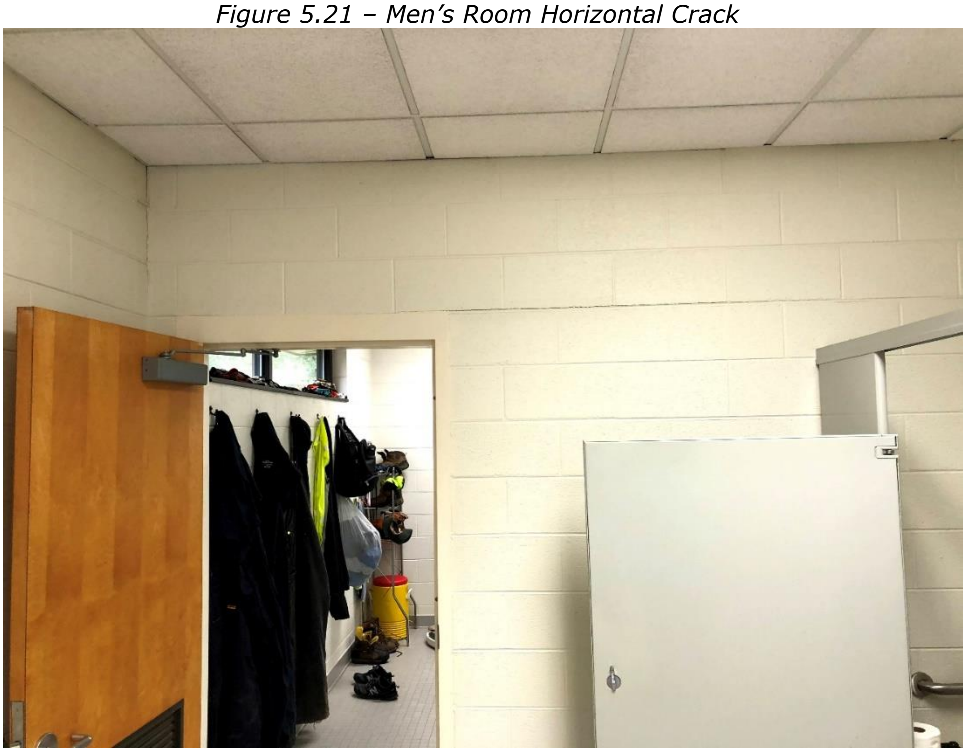


*Figure 5.22 – Laboratory Vertical Crack and Wall Separation*


- Vertical crack patterns propagating though CMU blocks were observed at a few exterior and interior wall locations. These types of cracks can indicate higher stress levels in walls and can be more severe than previous noted step cracks and vertical and horizontal cracks that propagated along the mortar joint lines of the CMU assemblies. As previously noted existing veneers prevented verification of these cracks being through wall crack conditions. Note the following locations:
    - First Floor – east wall below the exterior window sill in the Lab (ref. **Figure 5.23**).
    - First Floor – west wall between the upper and lower cabinets in the Lab (ref. **Figure 5.24**).
    - First Floor – interior wall at the base of the wall in the Lab Corridor / Office areas.

*Figure 5.23 – Laboratory Vertical Crack Below Window*

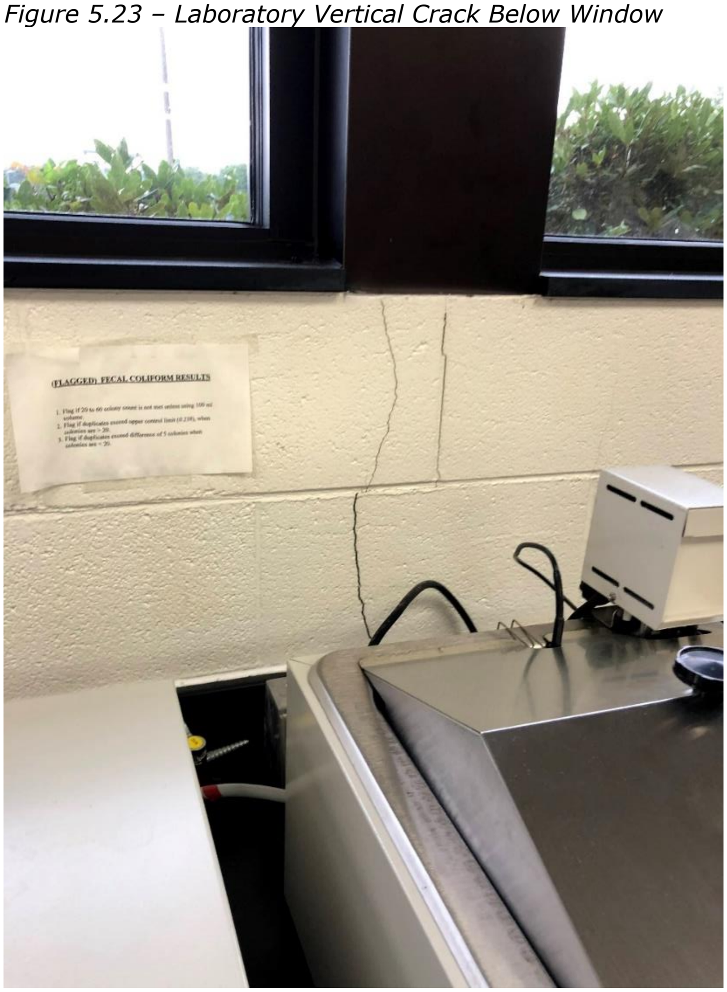
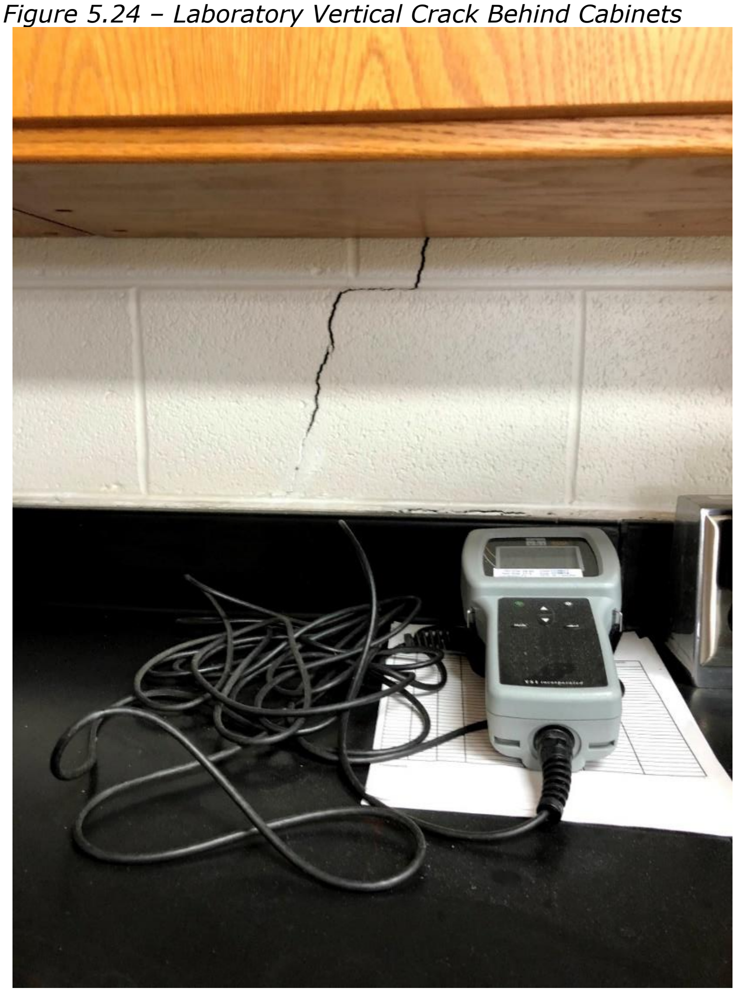



*Figure 5.24 – Laboratory Vertical Crack Behind Cabinets*



- Miscellaneous item to note regarding the interior partition wall in the Foyer. It was noted the top of the wall had separated from the bottom chord of the interior steel truss.

Regarding the structural steel elements, the wide flange lintel beams could not be observed due to the CMU construction in place. Although a majority of the CMU assemblies observed are in poor condition it does not appear the steel lintel beams are structurally inadequate. The steel truss assemblies observed appeared to be in good condition.

Regarding the timber elements, the truss assemblies could not be observed due to the roof panels and ceiling tile assemblies in place. No irregularities in the roof or exterior eave and soffit lines were observed, however note the following:

- Water spots were observed in interior ceiling tiles in the Men's Room and the Men's Lockers room.

- Water staining was observed in the exterior underside wood decking of the south end of the main entry canopy (ref. **Figure 5.25**).

*Figure 5.25 – Main Entry Canopy Water Staining*

Recommendations for this building include the following:

- It appears from the extensive amount of cracking observed in the masonry wall assemblies, that the building could be potentially experiencing differential settling movements. The building foundation includes timber piles which should have prevented this condition. Not knowing if pile assemblies were designed as friction or bearing piles makes it difficult to determine if pile failure could be the root cause of the settling. In addition, it is possible that pile rot or degradation may be contributing to, or the cause of possible pile failures. Unfortunately, it is estimated pile / foundation modifications would be expensive. Fortunately, this condition does not currently appear to be resulting in an unsafe structure for plant personnel. Therefore, future proposed modifications could be scheduled to work within annual budget planning and constraints. However, areas noted where CMU cracks or gaps have occurred should be monitored periodically to verify new areas have not developed and existing conditions have not worsened, to an unsafe condition.

- Future CMU wall repairs should not be made until after it has been established that foundation repairs have essentially stopped potential foundation settling.
- Review areas where water stains were observed in ceiling tiles and verify that these are not the result of roof leaks.

### 5.1.3 Electrical Condition Assessment

All electrical equipment within the Administration Building appeared to be in good working condition. No changes to the existing electrical equipment are recommended at this time.

## 5.2 Power Distribution and Emergency Generator

### 5.2.1 Purpose and Description

Utility power for the WWTF is provided by Duke Energy and is fed to the facility through an existing utility pad-mounted transformer to step down to 277/480V 3-phase service. Two duct banks feed power from the utility pad-mounted transformer to two main switchboards, 'SB-1' and 'SB-2'. Power is then distributed from existing switchboards SB-1 and SB-2 through a series of distribution feeder ductbanks to various main power panels at each major process area throughout the facility. Existing switchboard SB-1 feeds power to the Administration Building, Utility Building, Recycle Pump Station, and the Influent Pumping Station. Power to the Utility Building from SB-1 is then also distributed to the tertiary filters, UV disinfection system, and plant non-potable water pumps. Existing switchboard SB-2 feeds power to the Aeration Basin Blower Control Building and the Dewatering Building.

Emergency back-up power for the WWTF is provided by a new 1500 kW diesel-driven generator set 'GEN-1' which was installed in 2019. Emergency power is distributed from the generator set to existing switchboards SB-1 and SB-2 by a new main switchgear 'SWB-1' which was installed with the new generator set in 2019.

### 5.2.2 Electrical Condition Assessment

A walk-through of the wastewater treatment facility was conducted to observe and note the present condition of the electrical power distribution and utilization equipment, such as switchgear, back-up power generation, switchboards, panelboards, motor control centers, stand-alone motor controllers, control panels, etc. The walk-through started with the main electrical service entrance equipment that provides electrical power to the entire facility.

Recently, a new diesel-driven generator set, 'GEN-1' was installed to provide back-up power for the entire wastewater treatment facility, in the event utility power is not available or not within power quality standards. Additionally, a new walk-in main switchgear enclosure, 'SWB-1' was installed which houses several power circuit breakers and programmable relays to monitor and control utility and back-up power as well as distributing electrical power to two (2) existing switchboards, SB-1' and 'SB-2' that further distribute power to the facility. Since, SWB-1 and GEN-1 is new, this equipment should provide 20 to 30 years of service life with proper maintenance. It was noted that closed transition switching is desirable as well as power quality monitoring/recording. SB-1 and SB-2 were installed around year 2000 and are approximately 20 years into the typical 30 year service life for equipment of this type. Therefore, it is recommended that SB-1 and SB-2 be replaced with new switchboards with increased ampacity ratings in the short term, in conjunction with locating these within a weather-resistant enclosure for increased protection against inclement weather.

## 5.3 Septage Receiving

### 5.3.1 Purpose and Description

Waste from septage haulers is accepted at the City's WWTF at a septage receiving station located on the old treatment plant site, south of the existing thickeners and dewatering building. Flow from the septage receiving station is directed to the existing 42-inch outfall sewer that discharges to the WWTF Influent Pumping Station. The City recently installed a new ScreenCO Systems LLC Mega Screen 800 septage receiving station at the old treatment plant site to improve septage receiving. The new station allows septage haulers to discharge their waste to the Mega Screen 800, which utilizes 3/8-inch gapped manual bar screens to remove trash, rags, and other debris from the septage waste stream prior to reaching the Influent Pumping Station. Trash and other debris captured by the screen is manually raked to a drain tray where it is allowed to drain further prior to disposal in a dumpster. Liquid from the drain tray is also directed to the existing outfall sewer by gravity. Trash and debris removed from the septage receiving screen is hauled off for disposal at a municipal solid waste landfill.

The City currently bills for usage of septage receiving at a rate of $60 per 1,000 gallons. Septage haulers are currently required to check-in at the WWTF Administration Building and manually report the volumes of waste discharged prior to dumping at the septage receiving station. The controlled access gate at the solids handling facility prevents unwanted access to the septage receiving station. The volumes of waste discharged at the septage receiving station are generally estimated by the driver. No systems are currently in place to verify the volume of wastes discharged.

### 5.3.2 Equipment and Process Condition Assessment

The City's septage receiving was just upgraded with a new Mega Screen 800. There are no recommended improvements to the septage receiving equipment since it is new and its operation is completely manual by nature. Discussions with City staff indicate they are very happy with the performance of the new septage receiving station, and that operation is very simple. City staff have also noted the new Mega

Screen 800 has significantly reduced ragging issues at the influent pumps, downstream of septage receiving.

Minor improvements may be made to septage receiving to improve septage receiving record keeping and to prevent freezing in colder weather. The following improvements are suggested; however, they are considered secondary and not requisite to the successful operation of the process.

- Recommend installation of weigh scales or septage receiving flow meter to track volume of septage disposed. The City has noted that they are currently evaluating options for a kiosk system to improve septage receiving and bulk water sales tracking.
- Recommend providing heat tracing at septage receiving station to prevent freezing and clogging issues during colder weather.

## 5.4 Influent Pumping Station

### 5.4.1 Purpose and Description

Influent wastewater from the existing 42” diameter outfall sewer enters the City’s WWTF at the Influent Pumping Station where the influent flow rate is measured, and then pumped up to the screening and grit collection channel. The Influent Pumping Station is a dry pit style pump station with an external influent channel and wet well, and an internal dry pit below grade to house the pumps, motors, and piping for ease of access and maintenance, as seen below in **Figure 5.26**.

*Figure 5.26 – Influent Pumping Station Dry Pit*

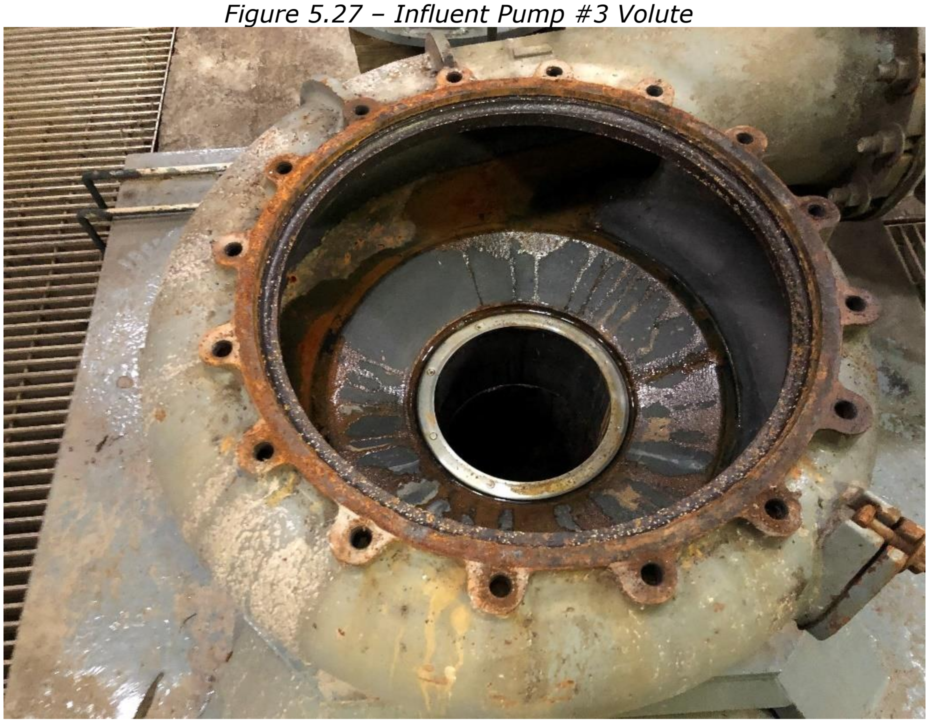
Influent wastewater from the 42-inch outfall enters the influent channel in the wet well and flows through a trash rack to remove large trash and debris. The trash rack is located approximately 28-feet below grade in the wet well and is constructed of steel bars spaced 2 ½-inches apart. The trash rack is cleaned manually by an operator and screenings are hoisted out of the wet well by a ¼-ton jib crane mounted to the top of the wet well wall. The influent wastewater then flows through a cast-in Parshall flume insert for open channel flow measurement. Water level in the Parshall flume is measured by an ultrasonic level transducer, correlated to the associated flow rate per the flume rating curve, and is reported back to SCADA as the total plant influent flow. The influent flow then discharges to the wet well below where it is pumped up to the screening and grit collection channels by four centrifugal dry pit pumps. The plant wide sanitary sewer system discharges waste flows from within the WWTF to the Influent Pumping Station wet well after the Parshall flume via a 24-inch diameter DIP gravity sewer from manhole #1.

Influent pumps #1 and #2 are each 75 HP pumps with 8-inch diameter suction and discharge connections. Influent pumps #3 and #4 are each 125 HP pumps with 10-inch diameter suction and discharge connections. All four pump motors are controlled by variable frequency drives to allow influent pumping to match influent flow rates. All influent pump variable frequency drives have been replaced within the last year. Two interconnected 16-inch diameter discharge force mains direct the pumped flow up to the screening and grit collection channel. Pumps #1 and #3 are connected to one of the 16-inch force mains, while pumps #2 and #4 are connected to the other. The two force mains may be isolated from each other or operated in parallel by opening or closing a 16-inch plug valve within the dry pit of the Influent Pumping Station.

### 5.4.2 Equipment and Process Condition Assessment

The existing dry pit submersible pumps were installed in 2001 and are approaching 20 years old. In general, municipal pump station equipment which is properly installed, operated and maintained will have a reasonable life expectancy within the range of 20 to 30 years. Due to the high grit loading to the influent PS, excessive wear has been indicated on the existing pumps, requiring higher than typical rates of repair. Therefore, it is anticipated that the existing influent pumps are approaching the limits of their anticipated service life and will likely require replacement in the next 5 to 10 years. During the site visit, 125 HP pump #3 was out of service for repairs to the impeller, and the volute of the pump was open for inspection as shown in **Figure 5.27**. The pump casing appears to be in relatively good condition given its age, however the need for impeller repairs supports the assessment of a limited remaining service life.

*Figure 5.27 – Influent Pump #3 Volute*



Per discussions with the City's operations and maintenance staff, the influent pumps have historically required frequent de-ragging. The City noted that the 75 HP influent pumps #1 and #2 have historically been removed from service for de-ragging as much as twice a week prior to the installation of the new septage receiving station. Since the installation of the new septage receiving station, facility staff indicated the pumps now require de-ragging approximately one to two times per month. It is recommended to continue to maintain the new septage receiving pretreatment equipment to continue to protect the influent pumps.

As mentioned above in **Section 2.2**, and from past discussions with City staff, it has been noted that the Influent Pumping Station wet well and influent flow measuring Parshall flume are frequently flooded during wet weather events. The WWTF experiences high flows during wet weather events due to rainfall derived inflow and infiltration. During these events, the facility staff are often forced to limit the influent pumping rate to prevent solids washout from the secondary clarifiers. This causes the Influent Pumping Station wet well to flood and the Parshall flume to become submerged, resulting in the loss of an accurate flow signal, and the 42" diameter outfall sewer to the facility to surcharge. Original design documentation for the current WWTF provided by the City indicates that it was the original design engineer's intent to allow the 42" diameter outfall sewer to the facility to surcharge during power outages to provide approximately 4 hours of flow storage at average daily design conditions (i.e. 4.8 MGD). It is recommended that future improvements be made to provide an alternate means of influent flow measurement that will not be impacted by wet well flooding and surcharge conditions in the 42" diameter outfall sewer to the facility. It is recommended that future improvements be made as part of facility upgrades to handle future hydraulic loading conditions at the time of plant expansion or construction of flow equalization facilities.

Issues with FOG build-up in the Influent Pumping Station wet well were noted by facility staff. Previous site visits to the facility and discussions with facility staff have indicated that a hardened FOG layer has developed within the wet well. City staff have made multiple attempts to remove the hardened FOG layer, including manual removal and the use of City jet-vac trucks. FOG build-up in the wet well is expected to have caused reduced working volume and odor issues, and it is not feasible to continually remove FOG manually after it has accumulated. It is recommended that future improvements be made to remove FOG upstream of the Influent Pumping Station or fully disperse and emulsify FOG within the wet well to prevent accumulation.

The existing bubbler system for wet well level measurement was noted to experience frequent operational issues causing it to be unreliable for influent pump operation. These issues are common with bubbler type level measurement systems in wastewater environments due to orifice fouling. It is recommended to replace the existing bubbler system with level sensor equipment, ultrasonic level measurement system, or radar level measurement system to reduce operational issues and improve pump control reliability.

Facility staff also noted a lack of adequate ventilation within the Influent Pumping Station dry pit. In general, a minimum of six (6) continuous air changes per hour should be provided in the pump station building and dry pit. In discussions with the City, it was noted that the City would prefer that facility staff not be required to enter the wet well to clean the trash rack and maintain equipment due to safety concerns. It is recommended to evaluate the existing ventilation equipment capacity and provide ventilation system improvements as necessary in the short term. Long term objectives may include equipment replacement or modifications to the wet well to allow operation and maintenance from the top surface of the wet well.

### 5.4.3 Structural Condition Assessment

The existing Influent Pump Station facility is a below grade C.I.P. reinforced concrete liquid containment structure with building assembly above grade and the facility includes a rectangular shaped footprint. The facility / building construction includes C.I.P. reinforced concrete, concrete masonry units (CMU), structural steel and timber framing elements. Note the following:

- C.I.P. reinforced concrete construction observed included exterior and interior concrete walls, a foundation base slab on grade, beams and elevated slabs supporting the outside grade floor areas. Record drawings reviewed did not indicate the facility is supported on timber piles.
- CMU construction observed included building exterior walls.
- Structural steel observed included wide flange sections supporting the monorail hoist beam assembly in the building.
- Record drawings reviewed indicated timber elements included timber trusses to support the standing seam metal roofing for the building.

Regarding the C.I.P. construction observed note the following:

- Vertical surfaces of walls and beams and horizontal bottom surfaces of beams appeared to be in good condition and were plumb, level and primarily free of surface defects excepts for vertical or horizontal interior cracks observed at intermittent locations of the north, south, east and west walls of the facility (ref. **Figure 5.28**, **Figure 5.29**, & **Figure 5.30**).
- Horizontal surfaces of exposed exterior walls were level, however a continuous approximate ¼" wide linear crack was observed in the wet well south wall and alligator pattern cracking was observed at the east wall (ref. **Figure 5.31** & **Figure 5.32**).
- Top surface of the exposed areas of the base slab appeared to be in good condition. Top surfaces appeared to be level and plumb except where sloped to drain as intended. However linear cracks were observed in top surfaces of the concrete floor slab between the pumps. These cracks

appeared to be hairline in width and varied in length. It did not appear the slab surfaces each side of these cracks had deflected or settled differentially vertically (ref. **Figure 5.33**).

- Top and bottom surfaces of the elevated floor slab area appeared to be in good condition. Both top and surfaces appeared to be level and plumb and primarily free of surface defects except for single linear or diagonal cracks observed in slab panels. Cracks appeared to be hairline in width and to span from beam to beam across the width of the slab (ref. **Figure 5.34** & **Figure 5.35**).
- For the interior wall and bottom of elevated slab cracks efflorescence was observed.

*Figure 5.28 – Dry Pit Vertical and Horizontal Cracks in CIP Wall*

*Figure 5.29 –Dry Pit Horizontal Cracks in CIP Wall Below Stairwell*

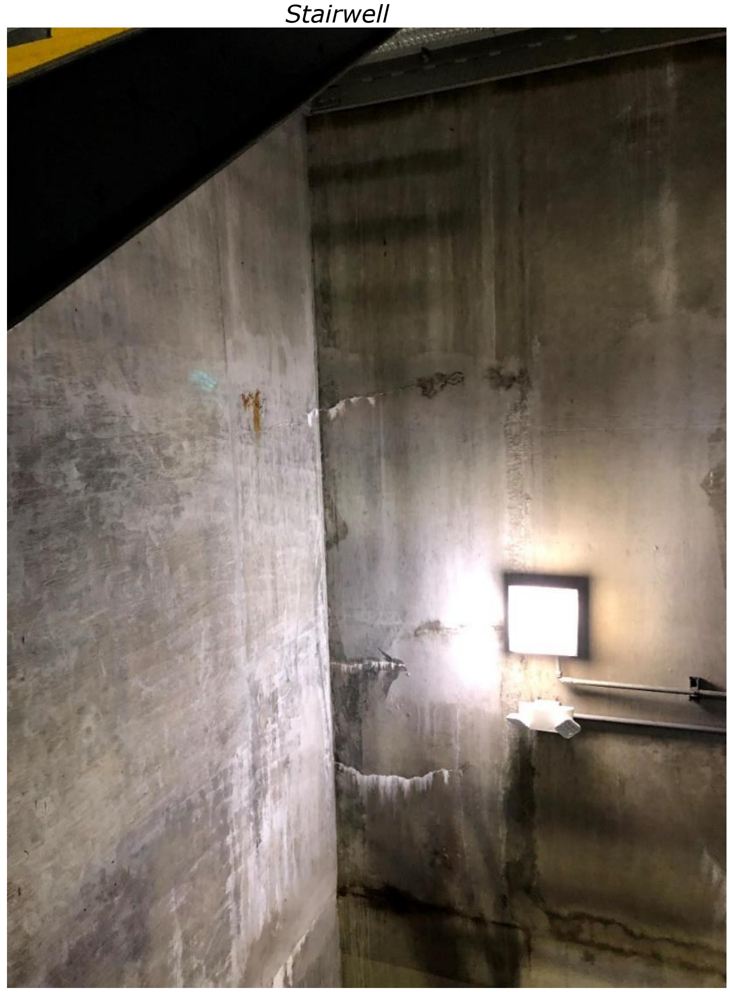


*Figure 5.30 – Dry Pit Vertical Crack in CIP Wall*

*Figure 5.31 – Large Crack at Top of Wet Well Wall*

*Figure 5.32 – Alligator Cracking Pattern at Top of Wet Well Wall*

*Figure 5.33 – Dry Pit Hairline Crack in Floor Slab*

*Figure 5.34 – Dry Pit Cracks in Elevated Floor Slab*

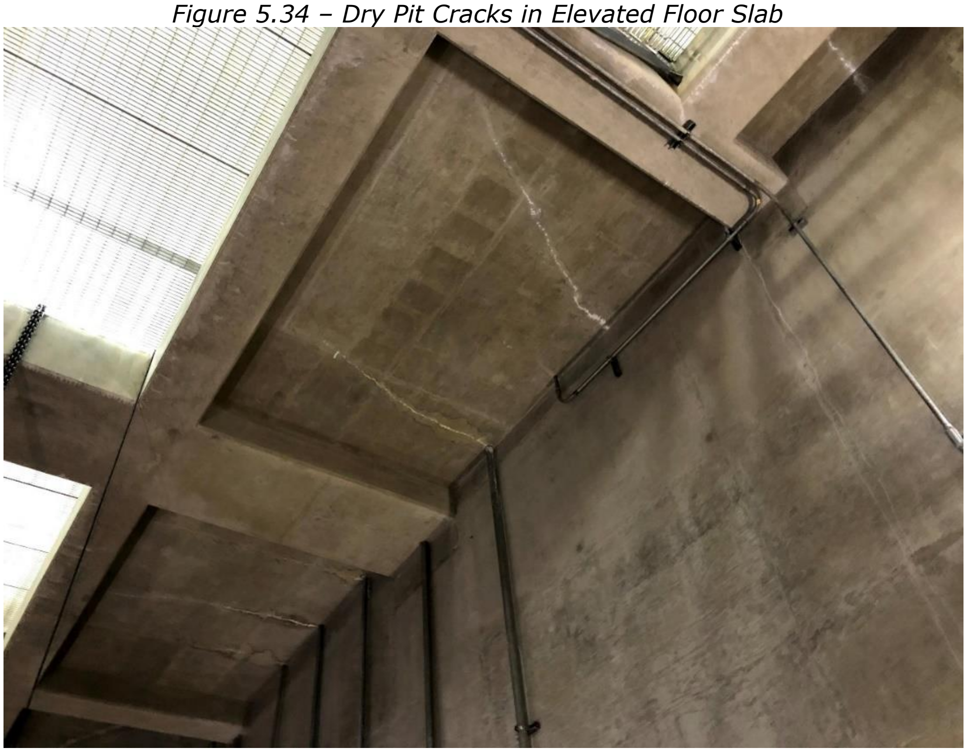


*Figure 5.35 – Dry Pit Cracks in Elevated Floor Slab*

Regarding the CMU wall construction observed the walls appeared to be in good condition. However, note the following:

- Crack in the CMU wall at the base of the far east monorail steel support beam (ref. **Figure 5.36**). A crack in the exterior veneer of the south wall was observed in the same location.

*Figure 5.36 – IPS Crack in CMU Wall at Base of Monorail Steel Support Beam*

Regarding the structural steel, it was noted the sections installed appear to vary in size from the sections indicated on the record drawings reviewed. However, the steel beam assemblies observed appeared to be in good condition.

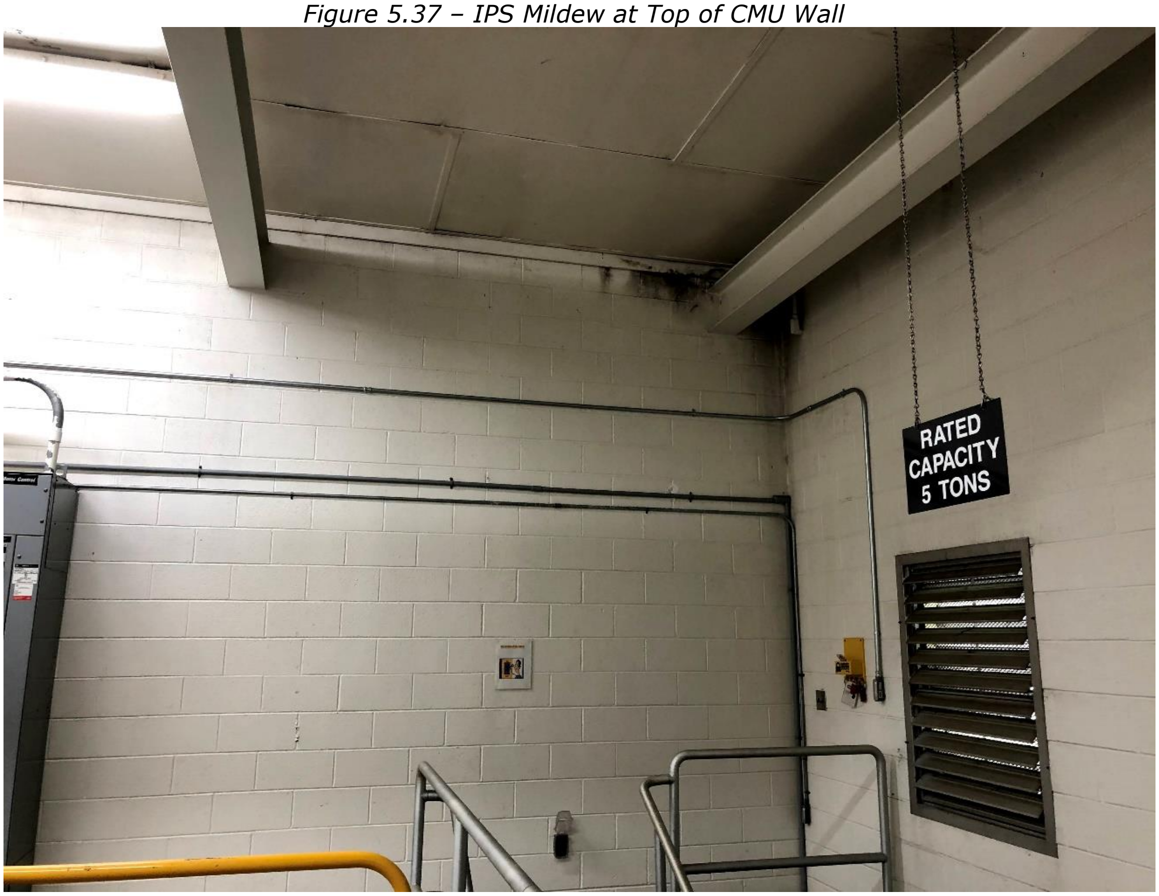
Regarding the timber truss elements, the truss assemblies could not be observed due to the roof panels and ceiling assemblies in place. No irregularities in the roof or exterior eave and soffit lines were observed, however note the following:

- There appeared to be mildew at top of the CMU wall at the interior of the southeast corner of the building (ref. **Figure 5.37**).

Recommendations for this facility include the following:

- Areas where interior cracks in walls and slabs have been observed should be monitored periodically to verify additional cracks are not developing and existing cracks are not increasing in width and / or length. Plan to repair cracks within the next 5 – 10 years to prolong the facility level of service. Repair of cracks should include a low viscosity, hydrophilic expanding polyurethane injection chemical grout adhesive system and polymer modified cementitious mortar.
- During the next maintenance shutdown allowing access to the interior wall and base slab surfaces that were unable to be observed during this review, verify exterior cracks, delaminations or spalls do not exist. Verification of whether these potential conditions exist could require repairs noted above in addition to an interior coating and / or repair mortars or concrete mixes to try to increase the level of service for the life span of the facility.
- Repair the exterior top of wall cracks with a cementitious repair mortar.
- Where mildew was observed remove ceiling to verify no existing roof leaks.

*Figure 5.37 – IPS Mildew at Top of CMU Wall*



### 5.4.4 Electrical Condition Assessment

The electrical power distribution and custom-built pump motor controllers located in the Influent Pumping Station (IPS) appear to be in good working order and do not exhibit any signs of abnormal deterioration and/or short-circuit events. The electrical equipment installed within the IPS should provide the expected service life cycles as supported by the manufacturers and as based on year the equipment was commissioned. Still, the expected service life cycles of the various custom-built pump motor controllers are based on their main component, the Variable Frequency Drive (VFD). Most VFDs are complicated electronic assemblies which house semi-conductors, integrated-circuit boards and micro-processors, which tend to fail due to several issues, including but not limited to poor input power quality, high ambient temperature, excessive humidity, dust or extreme output current. Therefore, it is recommended that a spare VFD and/or common replacement parts be kept on hand in the event of VFD failure and that staff maintains suitable environmental conditions. It was noted during the facility walk-through that the existing VFDs in the IPS were recently replaced in the last year.

## 5.5 Screening and Grit Removal

### 5.5.1 Purpose and Description

Screening and grit removal are accomplished in two parallel channels adjacent to the aeration basins at the Hendersonville WWTF, as shown in **Figure 5.38**. The two 16-inch force mains from the Influent Pumping Station direct influent flow to a common channel ahead of the screens. Each screening and grit removal channel include a US Filter Link-Belt Cog Rake mechanical bar screen with 3/8-inch bar spacing to screen the wastewater from the Influent Pumping Station before it enters the aeration basins. Trash and debris captured by the screens is raked up and over the bars by an electrically driven rake arm to a screening screw conveyor for discharge to an adjacent dumpster Water level is monitored upstream and downstream of the screens to monitor screen blinding and control the rake arm operation.

After screening, each channel includes an aerated grit chamber with US Filter chain and bucket grit collectors. The aerated grit chambers each contain an air diffuser header and fiberglass baffle wall adjacent to the common wall between channels to induce a spiral roll velocity pattern perpendicular to flow through the channel. The spiral roll velocity pattern causes heavier grit particles to hit the chamber walls and corner chamfers and settle quickly, while lighter organic particles remain suspended and pass through the chamber. Grit that settles in the chamber collects in a center trough where it is removed by the chain and bucket grit collectors. The chain and bucket grit collectors then discharge the captured grit to a screw conveyor where it is then discharged for disposal.

*Figure 5.38 – Screening and Grit Removal Structure*

### 5.5.2 Equipment and Process Condition Assessment

During the site visit, it was noted that screen #1 was experiencing significantly higher flow velocity compared to screen #2 due to the orientation of the incoming flows. Most of the flow from the Influent Pumping Station was being discharged by one of the two 16-inch force mains, resulting in higher velocity in the associated channel and forward momentum pushing flow into screen #1. This condition can result in poor screenings capture efficiency due to high approach velocities forcing screenings through the mechanical bar screen. Minimal screenings capture was noted on screen #1 as shown in **Figure 5.39**, below. It was noted that screen #2 was exhibiting a higher capture rate of trash and debris due to the lower approach velocity. In general, approach velocities ahead of mechanical screens should be limited to a maximum of 3.0 feet per second at peak flows. Discussions with the City indicated that all isolation valves between the two force mains are always kept open in order to maximize even flow distribution. Despite this, the discharge arrangement and operation of the influent pumps appears to dictate uneven flow distribution between the two force mains.

It is recommended to evaluate modifications to the existing screening channels to improve channel hydraulics and screening capture efficiency. Modifications may include installation of baffles upstream of the screens, or extension of screen influent channels to dissipate influent flow energy.

*Figure 5.39 – Screen #1 Operation*


The existing screen rake operation is activated based on differential water level across the screens. It is noted that the existing ultrasonic level transducers upstream of the screens are located directly in front of the discharge from the 16-inch force mains. The current location of the upstream level measurement is heavily influenced by turbulence resulting from the hydraulic jump/drop occurring as flow exits the force mains. It is recommended to relocate the upstream level measurement devices closer to the front of the screens if excessive rake actuation is experienced due to the highly turbulent flow conditions.

The grit removal screw conveyor was operating at the time of the site visit, allowing observation of the grit material being removed. Grit removed from the aerated grit chambers appeared to be relatively well washed and appeared to be fairly coarse in size with a uniform dark color as seen in **Figure 5.40**, consistent with influent inorganic materials.

*Figure 5.40 – Grit Captured from Aerated Grit Chambers*

Significant build-up of grit was noted in the aeration basins when the aeration basin #2 was taken off-line in March of 2020 to replace the aeration diffusers. Facility staff noted approximately 2 to 3 feet of grit had accumulated in the basin, essentially up to the bottom of the existing diffusers.

The existing screening and grit removal equipment was installed in 2001 and is approaching the anticipated service life of 20 years for most wastewater equipment. Operations staff noted that the reliability of existing equipment was not a major issue at this time. Noted screening and grit removal efficiency suggests that significant improvement could be managed through replacing the existing equipment with higher efficiency screening and grit removal systems and improving hydraulics within the screening structure. It is also recommended that future screening improvements be made to provide screening and grit removal upstream of the influent pump station to improve the reliability of the pumping equipment installed. Enhanced screening and grit removal will improve downstream treatment system reliability and performance by preventing unnecessary abrasion and wear of mechanical equipment, minimization of grit deposition in pipelines and channels, and decreasing the accumulation of grit in thickeners and aeration basins. While the existing equipment is reliably meeting treatment needs, it is recommended that future improvements be made to improve performance as part of facility upgrades to handle future hydraulic loading conditions at the time of plant expansion. It is also noted that enhanced screening and grit removal will improve biosolids quality, affecting its marketability and appeal to potential disposal outlets.

### 5.5.3 Structural Condition Assessment

The existing Screening and Grit Removal facility is a below / above grade C.I.P. reinforced concrete liquid containment structure and includes a rectangular shaped footprint. The facility construction consists of C.I.P. reinforced concrete wall and slab elements. Note the following:

- Vertical surfaces of walls and slabs appeared to be in good condition and were level, plumb, and primarily free of surface defects. Interior surfaces of the walls were not accessible due to the facility remaining in operation.
- Top surface of the base slab could not be reviewed due to the facility remaining in operation. In addition, record drawings reviewed did not indicate the facility is supported on timber piles.
- Top surface of the elevated floor slab area appeared to be in good condition, however minor areas of linear, alligator or map pattern hairline cracking was observed (ref. **Figure 5.41**).
- At the outside face of the north wall common with the adjacent aeration basin, it appears the assembly was constructed as a cold joint between the separate pours. A continuous crack was observed between these two assemblies (ref. **Figure 5.42**).

*Figure 5.41 – Screening and Grit Removal Alligator/Map Pattern Cracking*


*Figure 5.42 – Continuous Crack between Aeration Basin & Screening/Grit Removal*

Recommendations for this facility include the following:

- Areas where exterior cracks in slabs have been observed should be monitored periodically to verify additional cracks are not developing and existing cracks are not increasing in width and / or length. Plan to repair cracks within the next 5 – 10 years to prolong the facility level of service.
- During the next maintenance shutdown allowing access to the interior wall, base slab and elevated slab surfaces that were unable to be observed during this review, verify interior cracks, delaminations or spalls do not exist. Verification of whether these potential conditions exist could require repairs utilizing a low viscosity, hydrophilic expanding polyurethane injection chemical grout adhesive system and polymer modified cementitious mortar, in addition to an interior coating and / or repair mortars or concrete mixes to try to increase the level of service for the life span of the facility.
- Repair the continuous crack between aeration basin and north wall with a cementitious repair mortar and backer rod to caulk and seal the joint.

### 5.5.4 Electrical Condition Assessment

The existing Screenings and Grit Collection equipment adjoining the Aeration Basins appear to be in good working order along with satisfactory performance as reported by operations personnel. If the processing capacity of the wastewater treatment facility is increased in the future, then the existing Influent Pumping Station (IPS) will need to be modified and/or expanded to accept the additional influent flow. In doing so, it is recommended that new Screenings and Grit Collection equipment be installed, which will require an evaluation of hazardous and classified areas and the installation of additional electrical circuits would be required to supply power to this new equipment.

## 5.6 Aeration Basins

### 5.6.1 Purpose and Description

Biological treatment of the wastewater at the Hendersonville WWTF is accomplished in the two 2.4 MG aeration basins, aeration basin #1 to the south and aeration basin #2 to the north. The aeration basins were designed for the removal of BOD, TSS, and $NH_3$ using an extended aeration process to achieve complete nitrification of influent $NH_3$. The aeration basins were designed to operate in parallel. Effluent from the screen and grit removal channels combines in a common channel at the head of the two aeration basins where it flows into the aeration basins through two sluice gates per basin.

Aeration for biological growth is supplied to each basin from three 250 HP Hoffman multi-stage centrifugal blowers through dual 12-inch diameter air headers to five (5) grids of fine bubble diffusers per basin. Each aeration basin contains 450 diffusers, with 90 diffusers per grid, with each diffuser consisting of EDI's Flexair 84P fine bubble diffuser membrane panels. Diffuser grid 5 is located at the head of each basin, while diffuser grid 1 is located at the effluent ends. Air flow for each diffuser grid is supplied by a 6-inch diameter stainless steel drop leg from the 12-inch DI air header pipe. Each 6-inch SS drop leg includes a butterfly valve, orifice plate, and two 3/8-inch diameter SS calibration tubes to allow operators to adjust and balance air flow to each grid.

The aeration blowers also supply air to the two aerated grit chambers immediately upstream and adjacent to the aeration basins, as well as a 2-inch air pipe in the aeration basin common influent channel along the length of the channel to ensure its contents stay well mixed.

Return activated sludge (RAS) from the secondary clarifiers is returned to each aeration basin through dedicated 12-inch diameter ductile iron (DI) RAS pipes to maintain a mixed liquor suspended solids (MLSS) concentration within the trains to provide adequate treatment to meet effluent permit limits for $BOD_5$ and $NH_3$. RAS may be returned to the head of each basin, between the first and second diffuser grids (4 & 5), between the second and third diffuser grids (3 & 4), or any combination of these return locations. Treated effluent from each aeration basin flows over a 25-foot wide weir plate to a common effluent channel. Effluent from the aeration basins is then directed to the secondary clarifiers.

### 5.6.2 Equipment and Process Condition Assessment

It was noted during assessment of the existing facility that the existing secondary process has been historically operated at an MLSS concentration of approximately 4,300 mg/L. Based on current loading conditions, it is recommended that the facility operate closer to an MLSS concentration of 3,100 mg/L, which corresponds to an SRT of approximately 26 days. This is more than sufficient for complete nitrification, which was estimated to require a design SRT of 13 days during winter conditions. This will reduce clarifier loading, reduce air and mixing energy demands and minimize wear on RAS pumping equipment. Further, as loading increases to the plant, it will not be sustainable to operate at the current SRT, which could result in solids separation issues within the existing secondary clarifiers. In recent discussions with the City, it was noted that facility staff have reduced the MLSS concentration and now operate at a MLSS concentration target of 3,500 mg/L.

As noted above, RAS may be returned to three locations within each basin; at the head of the basin, between aeration grids 4 and 5, and between aeration grids 3 and 4. Introducing the RAS at multiple locations across the aeration basins is expected to result in reduced treatment capacity. Step-feeding RAS to multiple locations within the aeration basins will in turn reduce the MLSS concentration at the head of each basin, where influent loading is the greatest and high microbial populations are needed most. It is recommended that RAS only be returned to the head of each basin to ensure rapid reduction of influent loading to the aeration basins. Per discussions with the City, the RAS step feed points are not used, and RAS is only fed to the head of each basin.

It was noted that facility staff currently operate the first diffuser grid in each aeration basin (grid 5) at a DO concentration of approximately 0.5 mg/L to accomplish simultaneous nitrification/denitrification for denitrification of $NO_3$-N returning with the RAS flow. This is currently done to biologically replenish alkalinity that is consumed by nitrification of the influent $NH_3$-N, eliminating the need for caustic soda or lime addition. This practice will be evaluated further using the wastewater treatment process model to determine its effectiveness and applicability at future flow and loading conditions. Recommendations for modified operations may be provided based on the results of the modeling evaluation.

All DO and aeration basin performance monitoring is currently done manually. It is generally recommended to provide the capability for remote DO monitoring (at a minimum) to improve aeration control and reaction speed to changes in process operation. It is recommended to install DO and/or ORP meters, and online $NO_3$-N analyzers at locations in each aeration basin to improve process monitoring and record keeping, process reliability, aeration efficiency, and to provide the capability to implement automated aeration control. $NO_3$-N will be easier to monitor in the aeration basins compared to $NH_3$-N because it will be present at much higher concentrations. The concentration of $NO_3$-N present in the aeration basins is directly related to nitrification of the influent $NH_3$-N.

As noted previously, the City recently replaced the aeration diffuser membranes in both aeration basins. It is recommended that all diffuser membranes in the aeration basins be replaced every 5 to 7 years. This should be done to prevent excessive wear on the membranes and to ensure efficient aeration. This scheduled maintenance should also include removal of settled grit and debris from the aeration basins, and evaluation and documentation of the structural condition of the existing aeration basins.

### 5.6.3 Structural Condition Assessment

The existing Aeration Basins facility is a below / above grade C.I.P. reinforced concrete liquid containment structure and includes a rectangular shaped footprint. The facility construction consists of C.I.P. reinforced concrete wall and slab elements. Note the following:

- Vertical surfaces of walls appeared to be in poor condition. Vertical linear cracks were observed on the exterior faces of the north, south, east and west walls (ref. **Figure 5.43**, **Figure 5.44**, **Figure 5.45**, & **Figure 5.46**). In addition, horizontal cracks were observed on the exterior face of the west wall and diagonal cracks were observed on the north wall. For the exterior wall cracks efflorescence was observed. The south and west walls appeared to be plumb, however the north wall exhibited a bow deflection profile (ref. **Figure 5.47**) and the east wall exhibited multiple locations where the exterior surface tapers out and away from the top of wall line and the outside face of wall level with existing grade and slabs (ref. **Figure 5.48** & **Figure 5.49**). Interior surfaces of the walls were not accessible due to the facility remaining in operation.

- Horizontal surfaces of exposed exterior walls were level, however at areas near walkways multiple short length linear cracks were observed (ref. **Figure 5.50**).
- Top surface of the base slab could not be reviewed due to the facility remaining in operation. In addition, record drawings reviewed indicated the facility is supported on timber piles.
- Top surface of the elevated walkway slab areas appeared to be in good condition however multiple areas of linear, alligator or map pattern hairline cracking were observed (ref. **Figure 5.51**).

*Figure 5.43 – Aeration Basin #1 Vertical Cracks on South Wall*

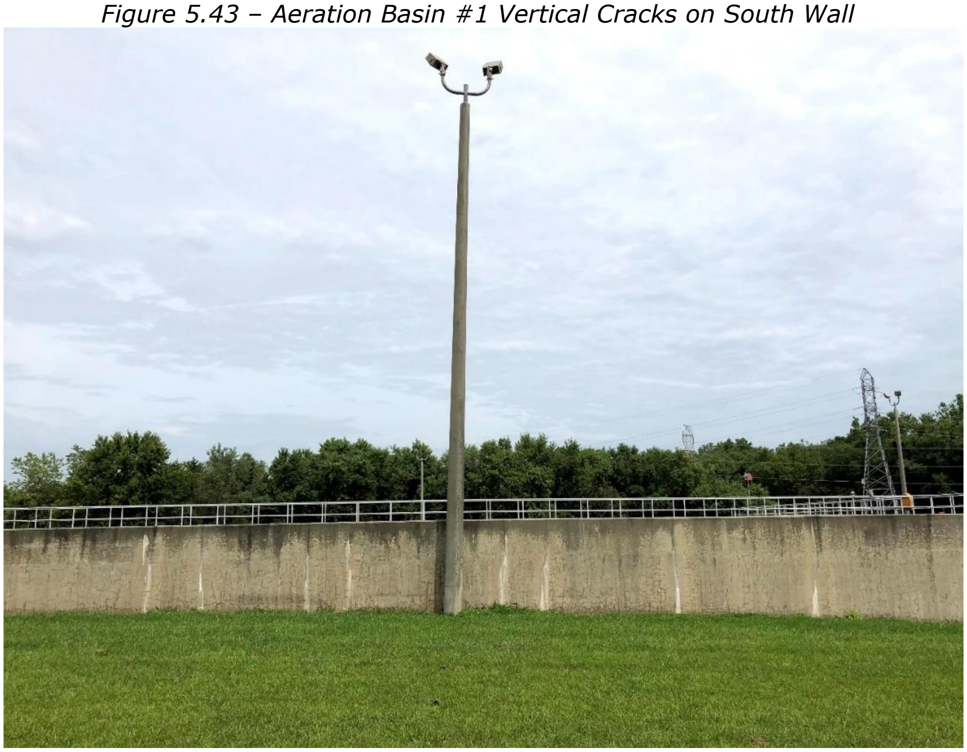


*Figure 5.44 – Aeration Basins Crack Patterns on West Wall*

*Figure 5.45 – Aeration Basins Vertical Cracks on West Wall*

Figure 5.46 – Aeration Basin #2 Crack Patterns on North Wall

Figure 5.47 – Bow Deflection in Aeration Basin #2 North Wall

Figure 5.48 – Aeration Basins Tapered Exterior Surface of East Wall

*Figure 5.49 – Aeration Basins Tapered Exterior Surface of East Wall*

*Figure 5.50 – Aeration Basin #2 Linear Cracks at Top of North Wall*

*Figure 5.51 – Aeration Basins Center Walkway Alligator/Map Crack Pattern*

Recommendations for this facility include the following:

- Areas where exterior cracks in walkway slabs and tops of walls have been observed should be monitored periodically to verify additional cracks are not developing and existing cracks are not increasing in width or length. Plan to repair cracks within the next 5 - 10 years to prolong the facility level of service.
- For all exterior walls, an engineering analysis of the basin is recommended within the next year. This analysis should include verifying the existing wall reinforcing and concrete compressive strengths are adequate. Provided analysis yields the wall assembly is sufficient, areas where exterior cracks in walls have been observed should be repaired within the next 2 – 3 years. Depending upon analysis results, repair may include excavation on the north, south, east and west side of the facility to verify cracks do not extend below grade. Repair of cracks should include a low viscosity, hydrophilic expanding polyurethane injection chemical grout adhesive system and polymer modified cementitious mortar.
- During the next maintenance shutdown allowing access to the interior wall, base slab and elevated slab surfaces that were unable to be observed during this review, verify interior cracks, delaminations or spalls do not exist. Verification of whether these potential conditions exist could require repairs noted above, in addition to an interior coating and / or repair mortars or concrete mixes to try to increase the level of service for the life span of the facility.
- For the north exterior wall exhibiting a bowed deflection, this wall is included in the recommended analysis previously noted above. It is recommended this wall be surveyed to verify that additional deflection is not occurring. Potentially this condition could be related to an unseen issue with the existing base slab and piles. Analysis work and repair may include excavation to the base slab to evaluate existing conditions further. Potential repair work may include installation of a C.I.P. reinforced concrete buttress and footing assembly.

## 5.7 Blower Building

### 5.7.1 Purpose and Description

The blower building houses the three 250 HP aeration blowers and their associated control panels under an open air shelter immediately adjacent to the effluent end of the aeration basins as shown in **Figure 5.52** below. The blower building also houses the motor control center and other associated electrical panels for the blowers and screening and grit removal equipment in an attached blower control building.

Each of the three existing 250 HP aeration blowers are driven by constant speed electrical motors, and the discharged air flow rate is controlled via manual butterfly valves on each blower's inlet. Each blower's 12-inch diameter discharge is connected to an 18-inch common discharge header, which then splits into two 12-inch aeration headers. Each 12-inch aeration header serves one of the two aeration basins. A 12-inch venturi meter is provided on each aeration header to measure the air flow rate to each aeration basin. The venturi meters are both mounted within the aeration basins below the water level, and each one is connected to a differential pressure transmitter to allow air flow monitoring via SCADA.

*Figure 5.52 – Blower Building*

### 5.7.2 Equipment and Process Condition Assessment

The existing multistage centrifugal blowers are generally oversized for the current operating conditions, resulting in excessive DO levels within Grids 1 through 4 of the aeration basins, with DO consistently in excess of 3 mg/L.  The system consists of two primary blowers with one unit acting as a redundant unit.

Multistage centrifugal blowers are designed to maintain a specified operating pressure in the air distribution system in lieu of a specified air flow rate. Air flow control is achieved by throttling the air being discharged from the blower to induce additional pressure drops within the system and force the blower back up the blower curve. This is not always an efficient way to operate. However, this does allow the blowers to deliver the design air flow rate across a wider range of operating conditions.

Turn down is more limited on multistage centrifugal blowers, with turndown typically only being 20 – 40% from the maximum output capacity of the blower. Turndown cannot be as reliably achieved with these blowers utilizing VFDs due to the potential occurrence of surge conditions which can ultimately cause damage to the equipment.

Multistage centrifugal blowers have been utilized in the wastewater treatment industry for medium and high air flow applications due to their higher efficiency and lower operating cost when compared to positive displacement blowers, which are typically utilized in smaller output applications.

The blowers were installed in 2001 as part of the initial plant installation. Typical life expectancy of blowers in this type of application are approximately 25 years, with well-maintained units capable of operating longer. The existing blowers are beginning to approach their design life. It is recommended that consideration be made to replace the existing blower equipment as part of future upgrades to increase treatment capacity at the existing WWTF. At that time, it would be recommended that alternative blower technology be considered (such as turbo blowers) to increase aeration efficiency. In addition, it would be beneficial to increase the number of blowers, which would allow more flexibility to turn down the air output to meet treatment system demands.

In general, blower discharge piping looks to be in good condition. However, the existing coating system is beginning to show significant signs of UV damage with signs of corrosion, as seen in **Figure 5.53** below. It is recommended that the existing piping be recoated with a UV resistant coating system within the next five years. Heat resistant coating systems, such as Tnemec Series 1552 Endura Heat, may be hot applied to allow coating system rehabilitation during operation of the blower equipment. It is recommended to consult with coating system manufacturers for coating system selection and detailed recommendations for application and worker safety.

Existing air isolation valves on the air header at each diffuser grid in the aeration basins appear to be in poor condition. It is recommended that flow splitting valves be replaced at the time of next diffuser replacement or when a train is taken offline for scheduled maintenance.

Figure 5.53 – Blower Discharge Piping Condition

### 5.7.3 Structural Condition Assessment

The existing Blower Building facility is an above grade structure building assembly and the facility includes a rectangular shaped footprint. The facility building construction includes C.I.P. reinforced concrete, concrete masonry units (CMU), structural steel and timber framing elements. Note the following:

- C.I.P. reinforced concrete construction observed included exterior and interior slabs on grade and equipment support pads. Record drawings indicated exterior columns are supported by shallow foundation footings. Record drawings did not indicate the facility is supported on timber piles.
- CMU construction observed included building exterior walls.
- Structural steel observed included exterior wide flange beam sections and hollow shaped steel column sections supporting the roof trusses.
- Record drawings indicated timber elements included timber trusses to support the standing seam metal roofing for the building.

Regarding the C.I.P. construction observed note the following:

- Top surface of the exposed areas of the exterior slab-on-grade appeared to be in good condition. Top surfaces appeared to be level and plumb except where sloped to drain as intended. However multiple areas of pipe support, adjacent sidewalk panels and isolated column bases appear to be continuously settling differentially from the slab-on-grade (ref. **Figure 5.54**, **Figure 5.55**, **Figure 5.56**, & **Figure 5.57**). The interior slab-on-grade area was not accessible for review.

*Figure 5.54 – Differential Settling of Slab-on-Grade at Aeration Header Wall Penetration*

*Figure 5.55 – Separation of Aeration Header Concrete Pipe Support from Slab*

*Figure 5.56 – Separation of Aeration Header Concrete Pipe Support from Pipe*

*Figure 5.57 – Separation of Stair Support at Blower Building*

Regarding the CMU wall construction observed the walls appeared to be in good condition, however, note the following:

- Assessment is based upon condition of the veneer. The building interior was not accessible for review.

Regarding the structural steel, the steel columns assemblies observed appeared to be in good condition. The beams could not be reviewed due to the ceiling finishes in place. However, note the following:

- In areas where settling has been observed, the column has settled vertically too, and bare metal was exposed above the finish exterior paint line (ref. **Figure 5.58** & **Figure 5.59**).

*Figure 5.58 – Blower Building Column Footing Settling*

*Figure 5.59 – Blower Building Column Settling*

Regarding the timber truss elements, the truss assemblies could not be observed due to the roof panels and ceiling assemblies in place. No irregularities in the roof or exterior eave and soffit lines were observed, however note the following:

- There could be issues behind the in-place finishes, based upon the settling that has been observed.

Recommendations for this facility include the following:

- Prior to future potential equipment upgrades, a subsurface soils investigation should be conducted to determine the properties of the existing foundation soils. The settling observed is impacting the adjacent sidewalk sections, pipe supports, access stairs to aeration basins and the building canopy steel columns and footings. Although beams and roof framing could not be observed, it is expected they are being impacted too.

- Results of the soils investigation should be utilized for future foundation modifications and should be installed prior to future equipment upgrades to replace / repair sidewalks, pipe supports, access stair framing, columns, footings and roof framing if required.

### 5.7.4 Electrical Condition Assessment

The existing blower assemblies and their associative Resistor Reduced Voltage Starter (RRVS) motor controllers were surveyed and appear to be in good condition. However, the RRVS units mounted with the existing Motor Control Center (MCC) are likely near the end of their useful service life. These units may have a limited source of replacement parts and manufacturer support. As noted above, the existing blower assemblies are reaching the end of their useful design life and are recommended for replacement as part of future upgrades to increase treatment capacity. Therefore, it is recommended that these RRVS units and their associated blower assemblies be replaced as part of future upgrades to increase treatment capacity. It is recommended that future equipment replacement consider variable speed control of blower assemblies with associated VFD units. Use of VFD-driven blower assemblies will reduce power consumption, based on seasonal and diurnal demand, while increasing operational flexibility.

## 5.8 Secondary Clarifiers

### 5.8.1 Purpose and Description

Mixed liquor from the aeration basins flows through one 36-inch pipe to a distribution box, where the flow is split to the two (2) 90-foot diameter secondary clarifiers. Each secondary clarifier has a side water depth (SWD) of 14-feet, and a base slab slope of 1/16-inch:12-inch (V:H) towards the center feedwell. Mixed liquor from the aeration basins flows underneath the secondary clarifiers and up into the center feedwell. Once flow enters the clarifiers, it distributes outward radially towards the effluent weir along the outer circumference of each clarifier, as the sludge settles to the bottom of the clarifier. The secondary clarifiers settle and concentrate the mixed liquor suspended solids to produce an effluent flow with low total suspended solids concentration to meet effluent permit limits. The settled sludge captured by the secondary clarifiers is withdrawn from the bottom of the clarifiers to be returned to the head of the aeration basins, and a portion of the sludge is wasted to the gravity thickeners. The return activated sludge (RAS) is sent back to the aeration basins to maintain a target MLSS concentration and ensure a healthy population of microorganisms to remove BOD and $NH_3$ from the wastewater while the waste activated sludge (WAS) is sent to the gravity thickeners to remove excess microorganisms and maintain the target MLSS concentration in the aeration basins. The WAS rate in most activated sludge systems is typically set to maintain a target solids retention time (SRT) in the biological system.

The two secondary clarifiers at the WWTF are both US Filter Envirex Tow-Bro® clarifiers that utilize a Unitube sludge withdrawal header. The sludge withdrawn by the Unitube header is directed to the recycle sludge wet well at the Recycle Pumping Station via an 18-inch DI pipe from each clarifier. Waste sludge is withdrawn from the bottom of each clarifier and piped via a 12-inch DI pipe to the waste sludge and scum wet well at the Recycle Pumping Station. Each clarifier is equipped with a scum skimmer arm that collects scum from the water's surface and discharges it to the waste sludge and scum wet well at the Recycle Pumping Station via an 8-inch DI pipe.

### 5.8.2 Equipment and Process Condition Assessment

In general, the existing clarifiers appear to be in good condition. The anticipated service life of the mechanical equipment of the secondary clarifiers is approximately 25 years or longer with good O&M practices. The maintenance and eventual replacement of clarifier mechanisms will be required, however, rebuilding of the existing mechanism may result in a significant cost savings.

It is recommended that the existing clarifier mechanical and drive mechanisms be fully inspected by the equipment manufacturer with recommendations provided to rehab the existing equipment to increase the service life of the equipment. These recommendations should be considered as part of long-term capital improvements and any necessary expansion of the existing WWTF to accommodate future loading conditions.

As part of rebuilding the existing drive systems, it is recommended that surfaces be sand blasted and coated with a high-performance coating system to protect equipment from corrosion and provide long term reliability.

It was noted that facility maintenance staff have installed a Weir-Wolf device (manufactured by Ford Hall Company, Inc.) on each secondary clarifier's scum skimmer arm to continuously clean algae growth from the effluent weir and launder trough, per **Figure 5.60** below. This system appears to adequately control algae growth on the effluent weir and within the launder trough. However, it should be noted that algae growth removed by this system is brushed into the effluent wastewater stream and discharged to the downstream tertiary filters. It is recommended that the City consider installing removable effluent launder covers in the future to prevent the growth of algae, and to further reduce odors caused by clarified effluent flowing over the weir.

*Figure 5.60 – Secondary Clarifier Effluent Weir and Launder*

During the site visit it was noted that the existing scum box is beginning to show significant wear and will require replacement at the time of clarifier rebuild. Refer to **Figure 5.61** below.

*Figure 5.61 – Secondary Clarifier Scum Box Condition*

Based on record drawing review, it appears that the existing secondary clarifiers do not include density current baffles. Both clarifiers were in operation at the time of the site visits, so this could not be confirmed. It is recommended that the City consider the installation of density current baffles in each secondary clarifier to further improve settling performance. It is widely recognized, based on extensive industry experience, that density current baffles help prevent solids from short-circuiting the designed flow pattern.

It is recommended that the existing concrete clarifier structures be coated with a high performance coating system on the inside of the structure to 1 ft below the normal water level to protect the integrity of the existing structure.

### 5.8.3 Structural Condition Assessment

The existing Secondary Clarifiers are below / above grade C.I.P. reinforced concrete liquid containment structures and include a circular shaped footprint of two adjacent tanks. The facility construction is primarily below grade and consists of C.I.P. reinforced concrete wall and slab elements. Note the following:

- Vertical surfaces of walls and slabs appeared to be in good condition and were plumb, level and primarily free of surface defects except for the exterior vertical cracks that were observed. These cracks were spaced approximately 4'-0" to 6'-0" apart. Due to the existing grade it could not be determined if the cracks continued below grade (ref. **Figure 5.62**). Interior surfaces of the walls were not accessible due to the facility remaining in operation.
- Top surface of the base slab could not be reviewed due to the facility remaining in operation. In addition, record drawings indicated the facility is supported on timber piles.

*Figure 5.62 – Secondary Clarifiers Vertical Cracks Around Perimeter*

Recommendations for this facility include the following:

- Areas where exterior cracks in walls have been observed should be monitored periodically to verify additional cracks are not developing and existing cracks are not increasing in width and / or length. Repair to include excavation around exterior walls of both tanks of the facility to verify cracks do not extend below grade. Plan to repair cracks within the next 5 – 10 years to prolong the facility level of service. Repair of cracks should include a low viscosity, hydrophilic expanding polyurethane injection chemical grout adhesive system and polymer modified cementitious mortar.
- During the next maintenance shutdown allowing access to the interior wall, base slab and launder channel slab and wall surfaces that were unable to be observed during this review, verify interior cracks, delaminations or spalls do not exist. Verification of whether these potential conditions exist could require repairs noted above, in addition to an interior coating and / or repair mortars or concrete mixes to try to increase the level of service for the life span of the facility.

### 5.8.4 Electrical Condition Assessment

No changes to the Secondary Clarifiers' electrical equipment are recommended at this time. The electrical equipment associated with the Secondary Clarifiers typically consists of packaged systems provided by the clarifier mechanism manufacturer, including a local control panel for the rake drive mechanism. Staff should continue to operate and maintain the existing equipment.

## 5.9 Recycle Pumping Station

### 5.9.1 Purpose and Description

The Recycle Pumping Station is also a dry pit style pump station with two exterior wet wells for return activated sludge and waste activated sludge/scum respectively. Return activated sludge (RAS) is pumped from the recycle sludge wet well to multiple points in the aeration basins at the head of the basins, between aeration grids 4 and 5, and between aeration grids 3 and 4. The RAS is pumped by two 50 HP, 10-inch diameter, dry pit vertical centrifugal non-clog pumps. Both recycle pump motors are controlled by variable frequency drives. Both recycle pump variable frequency drives have been recently replaced within the last year. Return activated sludge flow rate is measured by a 16-inch diameter Krohne electromagnetic flow meter on the 16-inch common discharge header. Under normal operation, the RAS flow rate is adjusted proportional to the influent wastewater flow rate measured at the Influent Pumping Station. The City recently replaced recycle pump #1 in March of 2020 with a new Grundfos 10-inch dry pit vertical centrifugal non-clog pump to match the original pump specifications.

A mixture of waste activated sludge and scum is pumped from the waste sludge wet well to the gravity thickeners by two 60 HP, 8-inch diameter, dry pit vertical centrifugal non-clog pumps. Both waste sludge pump motors are controlled by variable frequency drives. The variable frequency drives were recently replaced within the last year. Waste activated sludge flow rate is measured by an 8-inch Krohne electromagnetic flow meter on the 8-inch common discharge header. Outside of the Recycle Pumping Station, the 8-inch diameter WAS force main is split into two parallel 8-inch diameter WAS force mains under the parking area in front of the Recycle Pumping Station. Buried 8-inch isolation valves with valve boxes are noted to have been installed on each of the parallel 8-inch force mains immediately after the tee from the single 8-inch force main.

### 5.9.2 Equipment and Process Condition Assessment

The anticipated lifecycle for existing RAS/WAS dry pit pumps is approximately 20 years. As noted above, one of the RAS pumps was just recently replaced earlier this year. Therefore, it is recommended that replacement of the remaining RAS pump and the existing WAS pumps be budgeted for within the next 3 to 5 years.

The existing WAS wet well at the pump station exhibited a significant amount of floating debris within the wet well due to scum accumulation. It is recommended that improvements be considered to increase mixing within the wet well to allow removal of scum. This could be managed through providing a recycle on the WAS pumps to keep the contents of the wet well mixed or by providing a separate scum pump station at the existing clarifiers with dedicated scum chopper pumps with an internal recycle built in (similar to those manufactured by Vaughn). Discharge from the scum pump station would be tied into the existing WAS force mains going to the gravity thickeners.

Discussions with facility staff indicated that sludge recycle and wasting operations are currently manually controlled. It was noted that sludge recycling operations run continuously, while sludge wasting operations typically run 8 to 10 hours every other day. Consistent sludge recycling and wasting schedules can help improve treatment process and clarifier performance consistency. It is recommended to automate sludge recycle and wasting operations. RAS flow rates may be automated using several strategies including an operator specified percentage of influent flow to the facility, based on sludge blanket level in the clarifiers, based on mass balances, or an operator set flow rate. WAS flow rates may be automated to match an operator specified sludge age with the addition of online suspended solids sensors to monitor MLSS concentration and WAS concentration.

City operations staff noted poor ventilation and heating in the Recycle Pumping Station. At least six continuous air changes per hour should be provided within the pump station building and dry well area. Heating should be provided to provide reasonable working temperature conditions within the pump station for operators and maintenance staff. It is recommended to provide additional heating and ventilation system capacity within the existing pump station.

### 5.9.3 Structural Condition Assessment

The existing Recycle Pumping Station facility is a below grade C.I.P. reinforced concrete liquid containment structure with building assembly above grade. The facility includes a rectangular shaped footprint. The facility / building construction includes C.I.P. reinforced concrete, concrete masonry units (CMU), structural steel and timber framing elements. Note the following:

- C.I.P. reinforced concrete construction observed included exterior and interior concrete walls, a foundation base slab and beams supporting the elevated slab and grating at floor areas. Record drawings reviewed did not indicate the facility is supported on timber piles.
- CMU construction observed included building exterior walls.
- Structural steel observed included wide flange sections supporting the monorail hoist beam assembly in the building.

- Record drawings indicated timber elements included timber trusses to support the standing seam metal roofing for the building.

Regarding the C.I.P. construction observed, note the following:

- Vertical surfaces of walls and beams and horizontal bottom surfaces of beams appeared to be in good condition and were plumb, level and generally free of surface defects. Exceptions include vertical exterior cracks observed in the west wall of the return sludge wet well (ref. **Figure 5.63**) and horizontal interior cracks observed at intermittent locations of the north, south, east and west walls of the facility (ref. **Figure 5.64** & **Figure 5.65**).
- Horizontal surfaces of exposed exterior walls were level, however continuous approximate hairline to 1/8-inch wide linear cracks were observed in the exposed tops of the return sludge wet well west wall (ref. **Figure 5.66**).
- Top surfaces of the exposed areas of the base slab appeared to be in good condition. Top surfaces appeared to be level and plumb except where sloped to drain as intended. However, linear and alligator crack patterns were observed in top surfaces of the concrete floor slab between the pumps and under the stairs (ref. **Figure 5.67**). These cracks appeared to be hairline in width and varied in length. It did not appear the slab surfaces each side of these cracks had deflected or settled differentially vertically.
- Top and bottom surfaces of the elevated floor beams appeared to be in good condition. Both top and surfaces appeared to be level and plumb and primarily free of surface defects.
- At the exterior and interior wall cracks, efflorescence was observed.

*Figure 5.63 – Return Sludge Wet Well Vertical Cracks on West Wall*

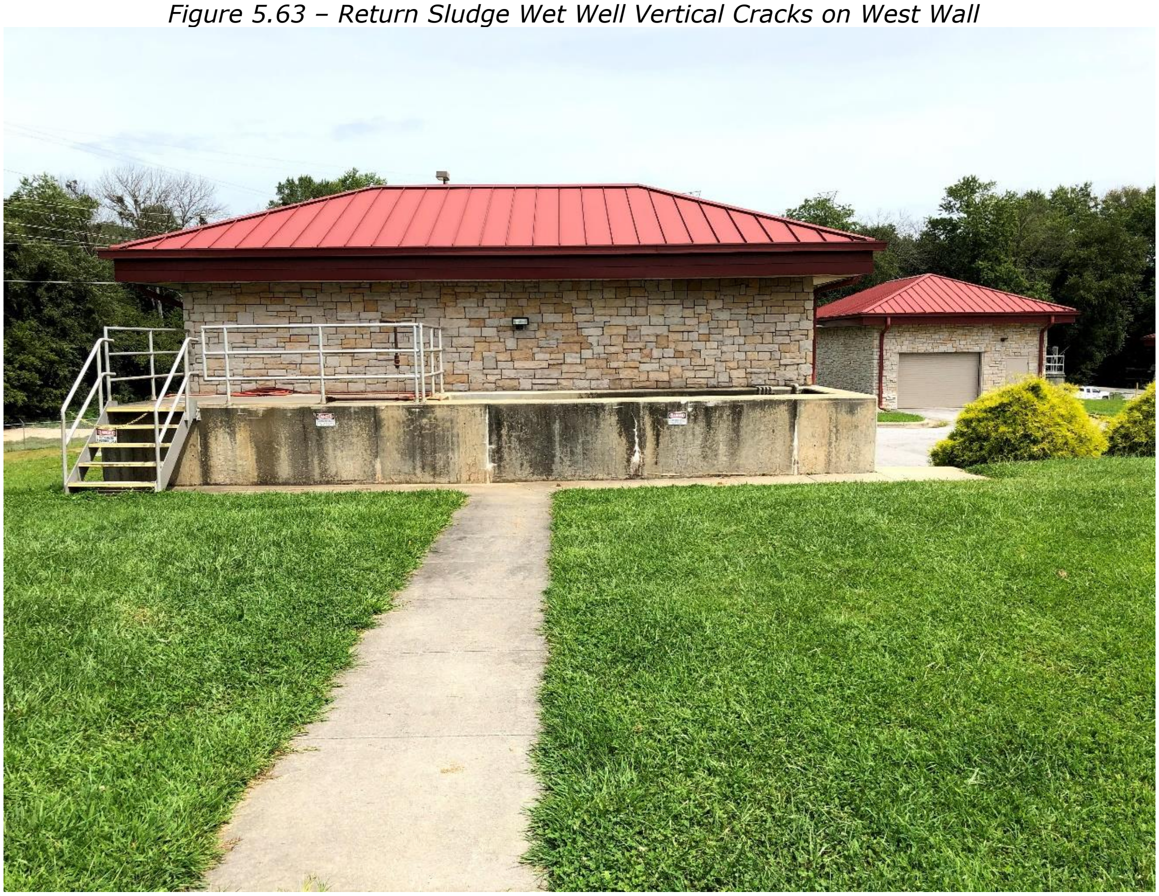


*Figure 5.64 – RPS Horizontal Crack in Dry Pit*

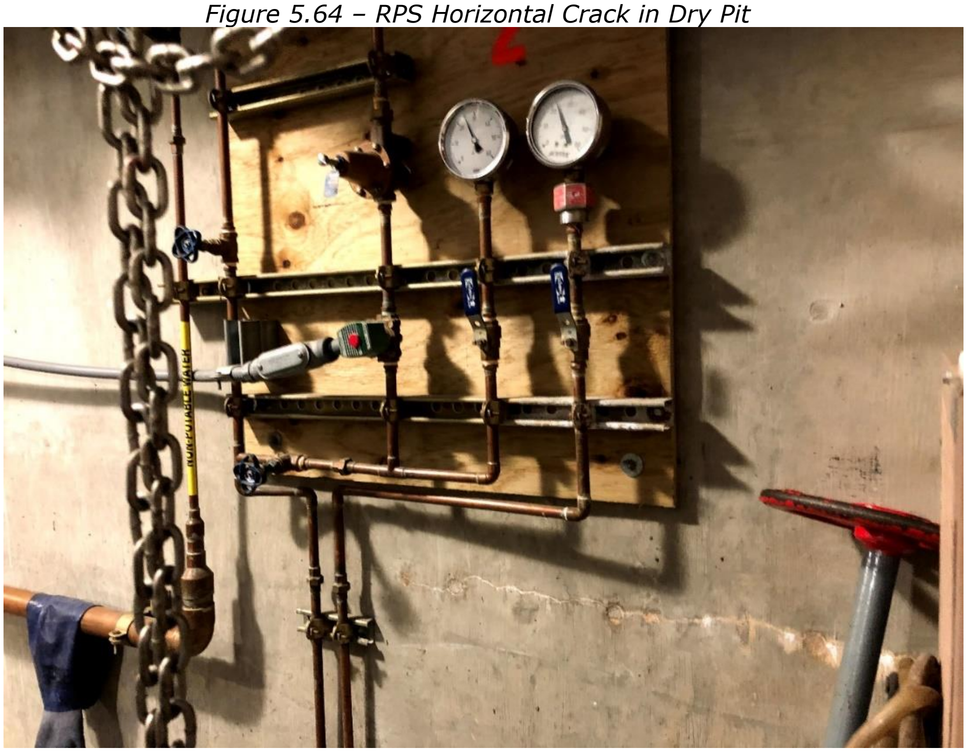


*Figure 5.65 – RPS Horizontal Crack in Dry Pit*

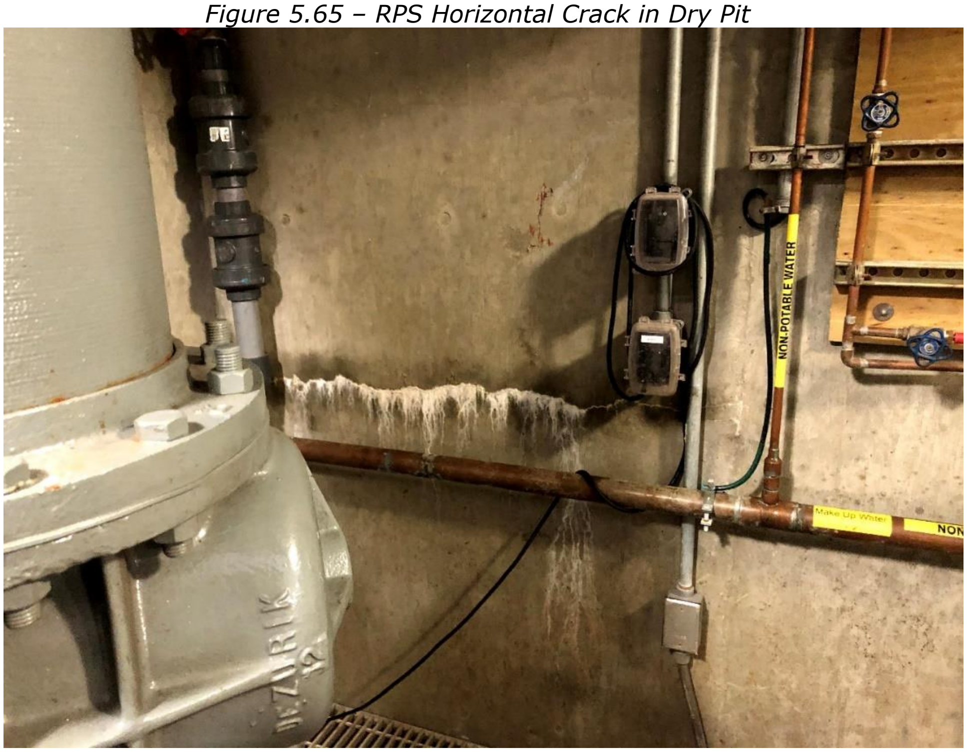
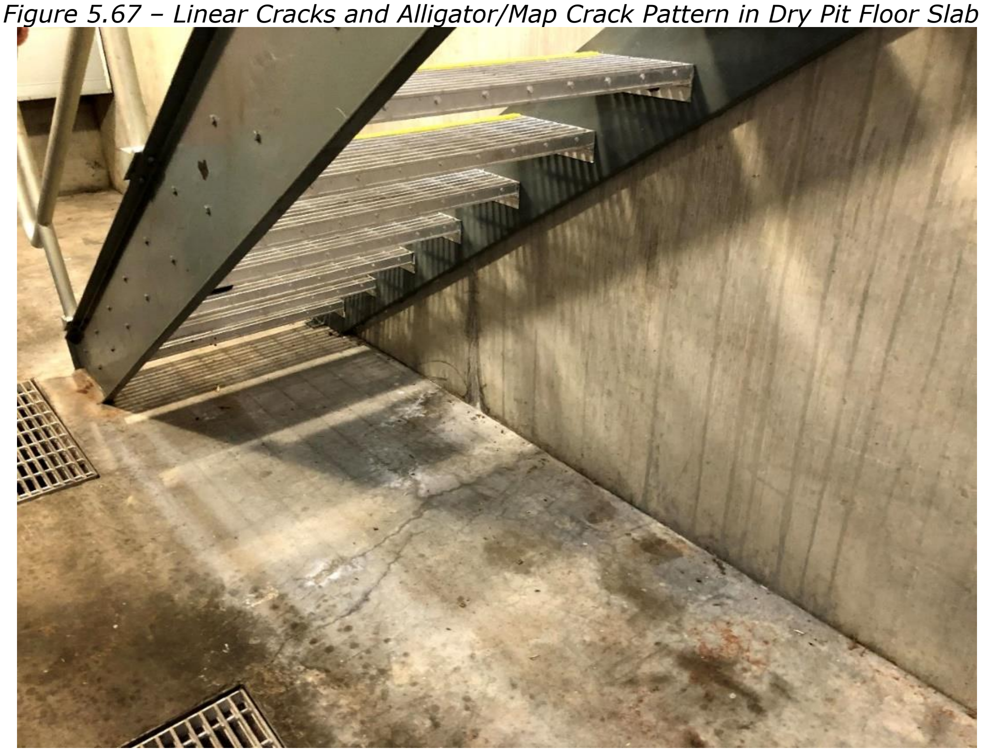


*Figure 5.66 – Return Sludge Wet Well Cracks on top of Wall*

*Figure 5.67 – Linear Cracks and Alligator/Map Crack Pattern in Dry Pit Floor Slab*



Regarding the CMU wall construction observed the walls appeared to be in good condition.

Regarding the structural steel, the steel beam assemblies observed appear to be in good condition.

Regarding the timber truss elements, the truss assemblies could not be observed due to the roof panels and ceiling assemblies in place. No irregularities in the roof or exterior eave and soffit lines were observed; however, note the following:

- There appears to be mildew at spots near the top of the CMU wall at the interior of the building.

Recommendations for this facility include the following:

- Areas where exterior and interior cracks in walls and slabs have been observed should be monitored periodically to verify additional cracks are not developing and existing cracks are not increasing in width and / or length. Repair to include excavation around exterior walls of recycle and waste sludge wet wells to verify cracks do not extend below grade. Plan to repair cracks within the next 5 - 10 years to prolong the facility level of service. Repair of cracks should include a low viscosity, hydrophilic expanding polyurethane injection chemical grout adhesive system and polymer modified cementitious mortar.
- During the next maintenance shutdown allowing access to the interior wall and base slab surfaces that were unable to be observed during this review, verify interior cracks, delaminations or spalls do not exist. Verification of whether these potential conditions exist could require repairs noted above, in addition to an interior coating and / or repair mortars or concrete mixes to try to increase the level of service for the life span of the facility.
- Repair the exterior top of wall cracks with a cementitious repair mortar.
- Where mildew was observed, remove ceiling and trim board to verify no existing roof leaks.

### 5.9.4 Electrical Condition Assessment

The electrical power distribution and custom-built pump motor controllers located in the Recycle Pumping Station (RPS) appear to be in good working order and do not exhibit any signs of abnormal deterioration and/or short-circuit events. The electrical equipment installed within the RPS should provide the expected service life cycles as supported by the manufacturers, based on the year the equipment was commissioned. Still, the expected service life cycles of the various custom-built pump motor controllers are based on their main component, the Variable Frequency Drive (VFD). Most VFDs are complicated electronic assemblies which house semi-conductors, integrated-circuit boards and micro-processors which tend to fail due to many issues including but not limited to poor input power quality, high ambient temperature, excessive humidity, dust or extreme output current. Therefore, it is recommended that a spare VFD and/or common replacement parts be kept on hand in the event of VFD failure and that suitable environmental conditions be maintained.

## 5.10 Tertiary Filters

### 5.10.1 Purpose and Description

Effluent from the secondary clarifiers combines in a 36-inch diameter pipe to the tertiary filters where it is filtered to further remove total suspended solids and turbidity prior to UV disinfection. Influent flow enters the tertiary filter common influent channel where it flows over influent weirs to two tertiary filter units. The tertiary filters originally consisted of two EIMCO traveling hood sand filters. Tertiary filter #1 was replaced in 2020 with a new Aqua-Aerobic Systems Inc. AquaDiamond cloth media traveling bridge filter. The AquaDiamond cloth media filter significantly increased the capacity of tertiary filter #1, which is now rated to treat an AADF of 6.0 MGD, and a PHF of 15.0 MGD. The original EIMCO traveling hood sand filter remains in tertiary filter #2. It is rarely used now, only when tertiary filter #1 is offline. New motor operated influent weir gates were installed on both filter basins to allow complete isolation of filter units and influent level control.

For tertiary filter #1, clarified effluent flows through two 7-foot wide motor operated influent weir gates into the filter basin and through the cloth filter media into the center of the AquaDiamond filter laterals, through the effluent wall weldment, and then over the effluent weir to the common effluent channel. The AquaDiamond traveling bridge platform includes a backwash pump, diamond lateral backwash arms and associated vacuum heads, solids collection tubes between each diamond lateral, backwash/solids collection discharge piping, and recycle piping. During backwash operations, the backwash arms and vacuum heads engage with 4 of the 8 cloth media diamond laterals. The backwash pump engages to vacuum collected solids off the cloth media as the platform travels down the length of the filter basin. The other four cloth media diamond laterals are backwashed when the traveling bridge platform reverses direction to return to its home position near the effluent wall weldment.

Backwash water is discharged to the backwash channel between filter basins where it then flows into the plant drain system and returns to the Influent Pumping Station. The original covered inlet channel for tertiary filter #1 was modified and repurposed as the backwash channel for the AquaDiamond filter system. Backwashing in tertiary filter #1 is controlled by either a timed interval setting or head differential between the filter basin and the filter #1 effluent chamber. Solids that may settle in the filter basin between the diamond laterals are periodically removed by the solids collection tubes that extend below and between the diamond laterals. The backwash pump engages for the solids collection cycle and settled solids are vacuumed from the bottom of the basin and discharged to the backwash channel. A scum collection trough on the effluent wall weldment in tertiary filter #1 collects and discharges scum to the backwash channel through a motor operated control valve at regular intervals between backwashing events.

For tertiary filter #2, clarified effluent flows through a 14-foot wide motor operated influent weir gate to a separate, parallel influent channel, and through a 30”x30” sluice gate to the covered inlet channel between filter basins, and then enters the filter basin through multiple inlet ports from the covered inlet channel along the length of the filter basin. Once in the basin, the clarified effluent flows down through the sand media, through filter underdrain system, and under the filter unit to the effluent chamber where it then flows over the effluent weir to the common effluent channel. The sand media in tertiary filter #2 is separated into uniform cells perpendicular to the length of the basin, along the entire length of the basin. Backwashing of each filter cell is accomplished by the traveling hood which includes a submersible backwash pump to fluidize the sand media and remove filtered solids. Backwash water is discharged to a fiberglass backwash trough mounted within the basin to the center wall and directed to the plant drain system. Traveling hood movement is actuated by a pneumatic lifting mechanism to lift the hood and allow it to travel to the next adjacent filter cell. Compressed air for the pneumatic controls is provided by an electric powered air compressor located within the adjacent Utility Building.

### 5.10.2 Equipment and Process Condition Assessment

As noted above, tertiary filter #1 was replaced earlier this year with a new AquaDiamond cloth media filter due to severe underdrain failure of the original traveling bridge sand filter #1. Severe underdrain failure has also been observed with existing traveling bridge sand filter #2, which is evident due to sand media build-up in the effluent chamber when the unit is operated. Tertiary filter #2 was offline during the site walkthroughs and is now solely used for redundant capacity in the event tertiary filter #1 is offline. The existing traveling hood, filter backwash pump, filter underdrain, and filter control panel for tertiary filter #2 appeared to be heavily worn, and in need of immediate replacement per **Figure 5.68** below.

The media depth in each cell of the tertiary filter #2 is well below the normal recommended level in most locations due to underdrain failure. Past observations of tertiary filter #2 operation have also indicated excessive failure rates for the existing pneumatic actuating systems that control the movement of the traveling hood. It is recommended that replacement of tertiary filter #2 be budgeted for replacement with a redundant AquaDiamond cloth media filter unit within the next 3 – 5 years. The timeframe of this recommendation is based on extremely infrequent operation of tertiary filter #2. More frequent operation of tertiary filter #2 would necessitate a more expedited replacement schedule.

*Figure 5.68 – Tertiary Filter #2 Condition*

The AquaDiamond cloth media filter in tertiary filter #1 is new and is in optimal condition per **Figure 5.69** below. The anticipated service life of the AquaDiamond filter unit is 20 years or longer with good O&M practices. It is recommended that the City budget for periodic cloth media replacement every 5 – 10 years or as recommended by the equipment manufacturer. It is also recommended that the City follow the manufacturer's recommendations for periodic chemical cleaning of the cloth media using a chloramine solution prepared from household ammonia and bleach. Periodic chemical cleaning of the cloth media will extend the useful life of the media and reduce the frequency of media replacement. The facility staff should exercise extreme caution when preparing the chloramine solution due to hazardous gases that may be produced if the solution is prepared incorrectly. After chemical cleaning, the contents of the filter basin are recommended to be returned to the Influent Pumping Station for treatment prior to discharge. Direct discharge of the chemical cleaning solution is not recommended due to the ammonia and chlorine concentrations required.

*Figure 5.69 – Tertiary Filter #1 Condition*


Algae build-up has been observed in all open areas of the tertiary filter basin, with extreme build-up in the influent channel prior to each influent weir, per **Figure 5.70** below. It is recommended to install a clear span structure over the tertiary filter basins to reduce sunlight exposure and prevent excessive algae growth. Multiple clear span structure options are available (structural fabric, metal, composite) with many options completely customizable and relocatable if desired. The clear span structure should include provisions for work lighting and clear access to the AquaDiamond filter platform home position for equipment maintenance. It is recommended that this structure be constructed at the time of tertiary filter #2 replacement, immediately after filter installation.

Figure 5.70 – Filamentous Algae growth in Tertiary Filter Influent Channel

## 5.10.3 Structural Condition Assessment

The existing Tertiary Filters facility is a below / above grade C.I.P. reinforced concrete liquid containment structure, and the facility includes a rectangular shaped footprint. The facility construction consists of C.I.P. reinforced concrete wall and slab elements. Note the following:

- The facility overall appeared to be in good condition although exterior surfaces below grade and except for Filter #2 which was offline, most of the interior surfaces of the walls were not accessible due to the facility remaining in operation.
- Record drawings did not indicate the facility is supported on timber piles.
- Concrete repairs were made during the recently completed Filter #1 replacement project. Additional concrete repairs do not appear to be needed at this time, except for the exterior face of the north wall where existing vertical cracks were observed (ref. **Figure 5.71**).

*Figure 5.71 – Tertiary Filter #2 Vertical Cracks on North Wall*


At the existing steel access stairs, multiple spots were observed where the paint coating was flaking and delaminating, and the steel was rusting (ref. **Figure 5.72**).

*Figure 5.72 – Tertiary Filters Access Stairs Coatings Failure and Rust*

Recommendations for this facility include the following:

- Areas where exterior cracks in the north wall have been observed should be monitored periodically to verify additional cracks are not developing and existing cracks are not increasing in width and / or length. Repairs include excavation at the north wall to verify cracks do not extend below grade. Plan to repair cracks within the next 5 – 10 years to prolong the facility level of service. Repair of cracks should include a low viscosity, hydrophilic expanding polyurethane injection chemical grout adhesive system and polymer modified cementitious mortar.
- During the next maintenance shutdown allowing access to the interior wall and base slab surfaces that were unable to be observed during this review, verify interior cracks, delaminations or spalls do not exist. Verification of whether these potential conditions exist could require repairs noted above, in addition to an interior coating and / or repair mortars or concrete mixes to try to increase the level of service for the life span of the facility.

### 5.10.4 Electrical Condition Assessment

The existing electrical equipment for Tertiary Filter #1 is new and in good condition. The design and construction of the replacement for Tertiary Filter #1 provided power supply and controls accommodations for the future replacement of Tertiary Filter #2. These accommodations included spare conduits (as seen in **Figure 5.72** beneath access stairs), spare breakers, and other equipment spares necessary. The existing electrical and controls equipment for Tertiary Filter #2 shall be replaced at the time of Tertiary Filter #2 replacement with a redundant AquaDiamond cloth media filter system.

## 5.11 Utility Building

### 5.11.1 Purpose and Description

The Utility Building houses power supply panels, controls, and equipment for multiple processes and utilities throughout the WWTF. The Utility Building is located on the east side of the WWTF between the Influent Pumping Station and the Tertiary Filters. The power supply panels and motor control center in the Utility Building provide power to the Tertiary Filter units, UV Disinfection system, plant water pumps, seal water pumps, and the filter air compressors. The Utility Building also houses the control panels for the UV disinfection system and plant water pumps, the plant water pump VFDs, network communication to the tertiary filter system controls, and SCADA network infrastructure that transmits process signals back to the Administration Building control room. The power supply and control panels within the Utility Building are shown in **Figure 5.73** below.

*Figure 5.73 – Utility Building Power Supply and Control Panels*


Potable water for lubrication of pump shaft seals on the influent pumps, recycle pumps, and waste pumps is provided by the seal water pumps located in the Utility Building. The seal water system in the Utility Building includes a seal water tank and two (2) 3 HP seal water pumps that pump seal water through a 2-inch water line to the Influent Pumping Station and Recycle Pumping Station. Potable water for the seal water system is supplied by a 2” potable water line with a water meter and backflow preventer located in the Utility Building before discharging into the seal water tank. Water level in the seal water tank is controlled by a float valve that fills the tank when the level drops and shuts off flow once the water level in the tank engages the float. The seal water system is shown in **Figure 5.74** below.

*Figure 5.74 – Plant Seal Water System*

The 6-inch plant water strainer is also located in the Utility Building to separate remaining solids from the plant water flow stream before it is distributed throughout the WWTF, as seen in **Figure 5.75** below. A 2-inch strainer discharge is located inside the Utility Building to rinse out the strainer periodically. The waste from the strainer discharge is directed to a floor drain.

*Figure 5.75 – Plant Water Strainer*


## 5.11.2 Equipment and Process Condition Assessment

In general, the seal water pumping equipment located in the utility building appears to be in good condition. Small systems and pumps of this nature typically will have an operating life of approximately 20 years when well maintained. Visual inspection of the existing pumps and seal water system did not show any significant signs of excessive wear. Therefore, it is anticipated that the existing system has a life expectancy of 5 – 10 years.

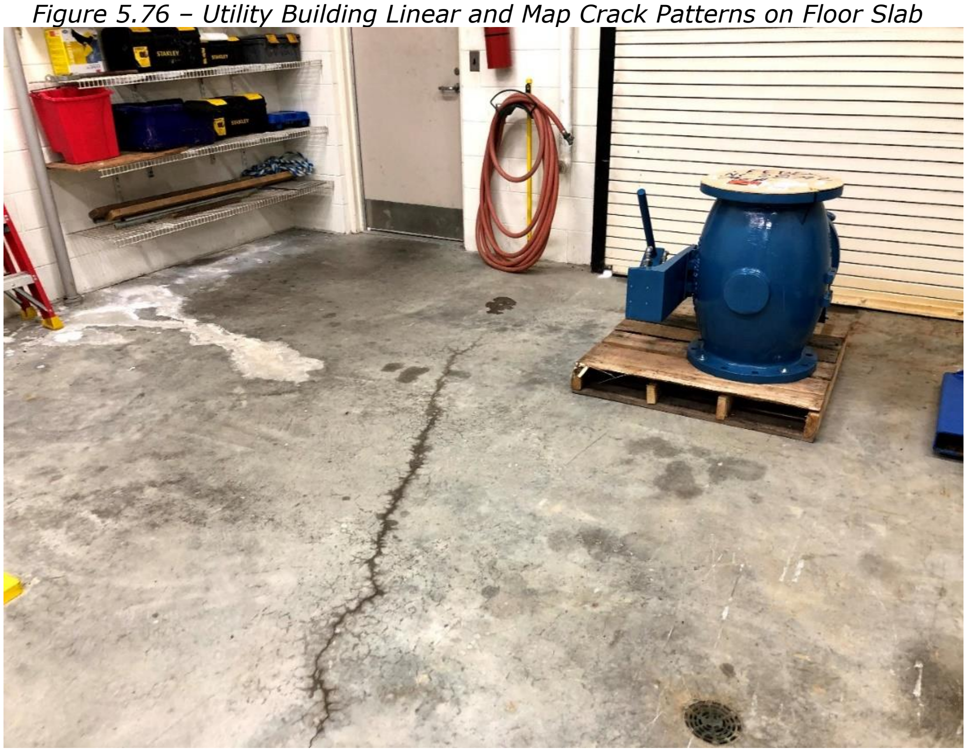
It is not anticipated that any changes will be required for the seal water system unless significant changes are planned to increase plant capacity within that timeframe. Therefore, it is recommended that the existing seal water system continue to be operated with standard operating procedures with equipment to be replaced as part of the next significant plant upgrade.

### 5.11.3 Structural Condition Assessment

The existing Utility Building facility is an above grade structure building assembly, and the facility includes a rectangular shaped footprint. The facility building construction includes C.I.P. reinforced concrete, concrete masonry units (CMU), and timber framing elements. Note the following:

- C.I.P. reinforced concrete construction observed included interior slab on grade. Record drawings indicated exterior walls are supported by continuous shallow foundation strip footings. Record drawings did not indicate the facility is supported on timber piles.
- CMU construction observed included building exterior walls.
- Record drawings indicated timber elements included timber trusses to support the standing seam metal roofing for the building.

*Figure 5.76 – Utility Building Linear and Map Crack Patterns on Floor Slab*



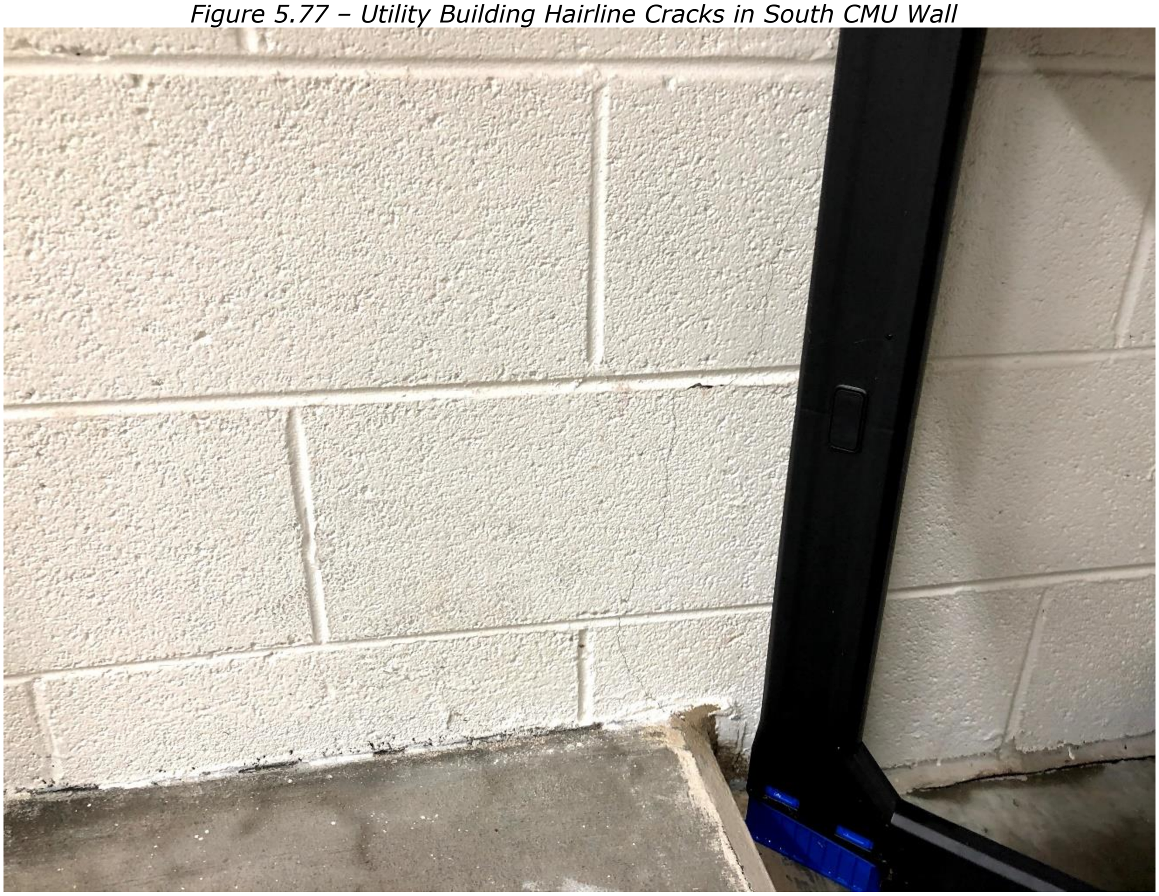
Regarding the C.I.P. construction observed note the following:

- The areas of slab observed appeared to be in good condition, although minor linear and map crack patterns were observed near the entry doors (ref. **Figure 5.76**).
- The footings are below grade and not accessible for review.

Regarding the CMU wall construction observed the walls appeared to be in good condition, however, note the following:

- Short vertical through block cracks, hairline width, were observed in the south wall in a couple locations (ref. **Figure 5.77**).

*Figure 5.77 – Utility Building Hairline Cracks in South CMU Wall*



Regarding the timber truss elements, the truss assemblies could not be observed due to the roof panels and ceiling assemblies in place. No irregularities in the roof or exterior eave and soffit lines were observed.

Recommendations for this facility include the following:

- Areas noted where CMU cracks were observed should be monitored periodically to verify new areas have not developed and existing conditions have not worsened.

### 5.11.4 Electrical Condition Assessment

As noted above, the electrical systems in the Utility Building provide power to the tertiary filters, UV disinfection system, plant water pumps, seal water pumps, and air compressors. The existing electrical equipment is in good condition and no changes to the existing electrical systems are recommended. The design and construction of the recent replacement of Tertiary Filter #1 provided modifications to the existing electrical equipment necessary to accommodate the future replacement of Tertiary Filter #2. The future replacement of existing tertiary filter and UV disinfection equipment shall include electrical equipment modifications necessary within the Utility Building to accommodate the new process equipment.

## 5.12 Disinfection Basin

### 5.12.1 Purpose and Description

Effluent disinfection at the Hendersonville WWTF is provided by a Trojan UV4000 ultraviolet light disinfection system as shown in **Figure 5.78** below. Treated effluent flow from the Tertiary Filters enters the Disinfection Basin where it is then directed into a concentrated flow stream by the UV4000's submerged UV reaction chamber. Two banks of UV lamps are located within the submerged reaction chamber, with one bank of UV lamps on either side of the reaction chamber. The UV system's submerged reaction chamber forces the flow through a narrow space so that it must pass through the two banks of UV lamps with very short distances for the light to travel.

The UV4000 system at the Hendersonville WWTF is rated to treat a peak hourly flow of 12 MGD at a UV dose of 25 mJ/cm$^2$ to meet an effluent disinfection standard of 200 MPN/100 mL with a UV transmittance of 65% and an effluent suspended solids concentration of 30 mg/L. The UV4000 systems consists of two banks of UV lamps, with the first bank located on the inlet side of the reaction chamber, and the second bank on the discharge side of the reaction chamber. Each bank consists of multiple modules mounted on a hinged arm that may be swung up by a hoist on the UV4000 unit to clean and replace UV lamps. The effluent wastewater is disinfected as it passes through the reaction chamber by the UV radiation emitted by the UV lamps. The disinfected effluent wastewater then flows over a sharp-crested weir plate to the plant water wet well. Two 20 HP Floway 8L-9, 9 stage, vertical turbine pumps pull non-potable treated effluent out of the plant effluent wet well for distribution to the in-plant non-potable water network for basin cleaning, equipment spray wash applications, and other plant maintenance needs. Effluent wastewater samples for regulatory compliance are sampled from the plant effluent wet well via a permanent ISCO flow paced sampler.

Effluent flow finally passes over a sharp-crested weir plate in the plant effluent wet well to the cascade reaeration steps. Effluent wastewater is naturally reaerated as it flows down the cascade reaeration steps, to meet the 5.0 mg/L effluent dissolved oxygen permit limit. The cascade reaeration steps are 8-feet wide, with a total of 6 steps, each 16-inches tall, and 2-feet deep. The reaerated, disinfected effluent wastewater is then discharged to the outfall in Mud Creek via a 36-inch diameter DI gravity sewer. The outfall location at Mud Creek is protected by a concrete end wall and fully rip-rap lined ditch as shown in **Figure 5.79** below.

*Figure 5.78 – UV Disinfection Equipment*


*Figure 5.79 – Effluent Outfall at Mud Creek*

### 5.12.2 Equipment and Process Condition Assessment

As noted in Section 4.3, the existing UV disinfection (UVD) system is recommended to be replaced due to equipment age, continual maintenance issues, cost, and increasing costs of replacement parts. In addition, the existing UV disinfection system does not meet the reliability criteria established in the NCDEQ Minimum Design Criteria for NPDES Wastewater Treatment Facilities. The reliability criteria require UV disinfection systems to be designed to treat the peak hourly flow rate with one bank out of service. The existing UVD system was designed to treat the existing design peak hourly flow of 12 MGD with both banks in service. It is recommended that a new UVD system be installed in a new channel adjacent to the existing disinfection basin in FY 2023 per the City's current capital improvement plan. The new UVD system shall be designed to treat the anticipated peak hourly flow rate with a minimum of one bank out of service. The existing disinfection basin is recommended to be left in place as a spare channel after installation of a new UVD system. The existing UVD system is recommended to be removed from within the channel, and an isolation sluice gate installed at the head of the existing channel. The existing fiberglass grating over the existing channel is in poor condition. The grating is recommended to be replaced with new fiberglass or aluminum grating at the time of the UVD system replacement.

The design of the new disinfection channel should include a pre-engineered clear span structure over the channel to protect equipment and personnel from sunlight exposure and adverse weather conditions. The pre-engineered clear span structure should include provisions for work lighting and clear access to equipment similar to the structure recommended for the tertiary filters. It is recommended the structure be constructed at the same time as the new disinfection channel.

### 5.12.3 Structural Condition Assessment

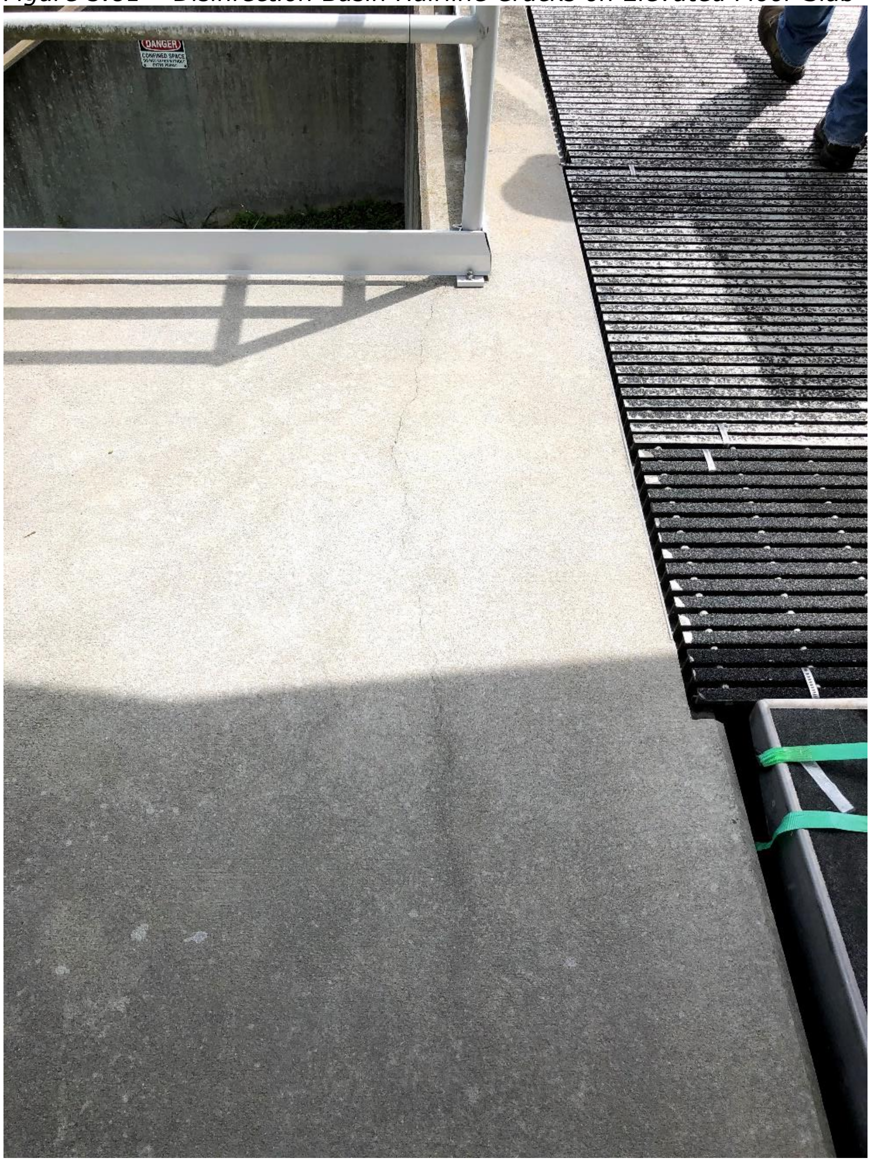
The existing Disinfection Basin facility is a below / above grade C.I.P. reinforced concrete liquid containment structure and includes a rectangular shaped footprint. The facility construction consists of C.I.P. reinforced concrete wall, column and slab elements. Note the following:

- Vertical surfaces of walls, columns and slabs appeared to be in good condition and were plumb, level and primarily free of surface defects. However, exterior horizontal and vertical cracks were observed at the east end of the north wall of the cascade stair (ref. **Figure 5.80**). Except for the above water sections of the cascade stair, interior surfaces of the walls were not accessible due to the facility remaining in operation.
- Top surface of the base slab could not be reviewed due to the facility remaining in operation. In addition, record drawings did not indicate the facility is supported on timber piles.

*Figure 5.80 – Cascade Reaeration Cracks in North Wall*

The top surface of the elevated floor slab area appeared to be in good condition however minor areas of linear pattern hairline cracking were observed (ref. **Figure 5.81**).

*Figure 5.81 – Disinfection Basin Hairline Cracks on Elevated Floor Slab*



Recommendations for this facility include the following:

- Areas where exterior cracks in slabs have been observed should be monitored periodically to verify additional cracks are not developing and existing cracks are not increasing in width and / or length. Plan to repair cracks within the next 5 – 10 years to prolong the facility level of service.
- The exterior cracks in the vertical stair wall observed should be monitored periodically to verify additional cracks are not developing and existing cracks are not increasing in width and / or length. Plan to repair cracks within the next 5 – 10 years to prolong the facility level of service. Repair of

cracks should include a low viscosity, hydrophilic expanding polyurethane injection chemical grout adhesive system and polymer modified cementitious mortar.

- During the next maintenance shutdown allowing access to the interior wall, base slab and elevated slab surfaces that were unable to be observed during this review, verify interior cracks, delaminations or spalls do not exist. Verification of whether these potential conditions exist could require repairs noted above, in addition to an interior coating and / or repair mortars or concrete mixes to try to increase the level of service for the life span of the facility.

### 5.12.4 Electrical Condition Assessment

As noted previously, it is recommended that the existing Ultra-Violet Disinfection (UVD) unit be removed from the existing effluent channel and a new effluent channel be constructed adjacent to it. A new UVD system is recommended to be installed in the new effluent channel while the existing channel will no longer be in service. Since most UVD systems contain a large number of LED electronic drivers with significant wattage demand, it is recommended that an isolation transformer be installed upstream of the incoming power circuit supplying this equipment. This will provide a means to smooth the incoming voltage, thus minimizing sags and/or surges in the electrical power delivery.

## 5.13 Sludge Thickening

### 5.13.1 Purpose and Description

Sludge thickening at the Hendersonville WWTF is accomplished by two (2) 50-foot diameter gravity thickeners. Each gravity thickener has a side water depth (SWD) of 13-feet, and a bottom slab slope of 2.75”:12” (V:H) towards a 5-foot diameter center thickened sludge sump. Existing gravity thickener #1 was originally constructed in 1965 as the original treatment facility’s anaerobic digester. The anaerobic digester was converted to gravity thickener #1 and gravity thickener #2 was constructed in 2001 with the completion of the current WWTF. Both gravity thickeners are US Filter Envirex F-Drive, full bridge gravity thickeners.

The gravity thickeners operate very similarly to the secondary clarifiers at the Hendersonville WWTF. Waste activated sludge is pumped to the gravity thickeners from the Recycle Pumping Station via two parallel 8-inch diameter DI pipes. The waste activated sludge enters each gravity thickener at approximate elevation 2,120 ft, above the sludge rake mechanism, where it flows to the center feedwell. The center feedwell in the gravity thickeners dissipates some of the concentrated flow energy from the 8-inlet pipe, and waste sludge then flows outward radially towards the supernatant effluent weir along the outer perimeter of each gravity thickener. As the flow slowly moves through the gravity thickener, the waste activated sludge settles and thickens under Type 4 settling as the sludge blanket builds up in the bottom of the thickener. Thickened sludge is raked to the center thickened sludge sump where it is then pumped out of each gravity thickener via an 8-inch thickened sludge suction pipe to positive displacement belt press feed pumps. Supernatant from the gravity thickeners is returned to the Influent Pumping Station via the in-plant sanitary sewer system.

The thickener building located between the two existing gravity thickeners contains the sludge feed and withdrawal piping, belt filter press feed pumps, pump VFDs, pump control panel, and motor control center. The two 8-inch thickened sludge suction pipes (one per thickener) join in a common suction header within the thickener building to feed three positive displacement Penn Valley Double Disc pumps. Thickened sludge from gravity thickener #1 may be pumped to the belt filter presses using pumps 2 or 3, and thickened sludge may be pumped by pumps 1 or 2.

### 5.13.2 Equipment and Process Condition Assessment

At the time of the condition assessment there was clear evidence of anaerobic conditions in both gravity thickeners which are resulting in gas production, excessive odors, and likely reduced sludge pH. Anaerobic conditions and expected reduced sludge pH have been corroborated by a sludge testing report from Huber that was performed in 2018 when evaluating thermal drying as a solids management alternative. At this time, it is recommended to increase the dewatering schedule to reduce sludge residence time in gravity thickeners to prevent it from going anaerobic.

However, this may not always be feasible to accomplish. It was indicated by operating staff that the volume of the existing thickeners is not enough to provide storage to allow for flexibility in the belt filter pressing schedule. Based on the anticipated WAS production rates at the permitted 4.8 MGD flow, the existing thickener tanks provide approximately 2.8 days of storage. However, it should be noted that gravity thickener technology is not intended to provide for sludge storage. Gravity thickeners are intended only for sludge thickening prior to feeding to a downstream digester or other stabilization process to reduce downstream equipment sizing and operating cost.

Therefore, it is recommended that additional aerated sludge storage be provided prior to the BFPs to provide increased operational flexibility in managing pressing schedules and to significantly improve odor issues at the facility. This may be accomplished in one of two ways. First, an aerated thickened sludge holding tank may be constructed downstream of the existing thickeners. This option will require maintenance of the existing gravity thickeners, modifications to thickened sludge withdrawal operations, and rehabilitation of existing gravity thickener #1 as discussed later.

Alternatively, the City may consider installation of new gravity belt thickeners or rotary drum thickeners to replace the existing sludge thickening process. This alternative would be expected to allow better thickening than is currently achievable, to approximately 5% TS prior to the BFPs. The existing gravity thickeners may be able to be converted to aerated thickened sludge holding tanks to store thickened sludge prior to the BFPs. Improved thickening prior to the BFPs may also allow better dewatering and reduced natural gas usage with the future biosolids thermal drying facility. It is recommended to evaluate the cost-benefit of these two alternatives in further detail, along with evaluation of the preferred BFP operating schedule.

Sludge was being withdrawn from the gravity thickeners for belt filter pressing at the time of the site condition assessments, allowing visual inspection of both gravity thickeners below the normal operating water level (ref. **Figure 5.82** and **Figure 5.83**). No internal coating system was noted within the gravity thickeners during the condition assessment. In general, it is recommended that all gravity thickener facilities install a high performance coating system on the interior of the concrete structure and on all interior steel materials to protect them from corrosion and ensure their structural integrity, especially if anaerobic conditions are regularly observed.

*Figure 5.82 – Gravity Thickener #1 During Thickened Sludge Withdrawal*

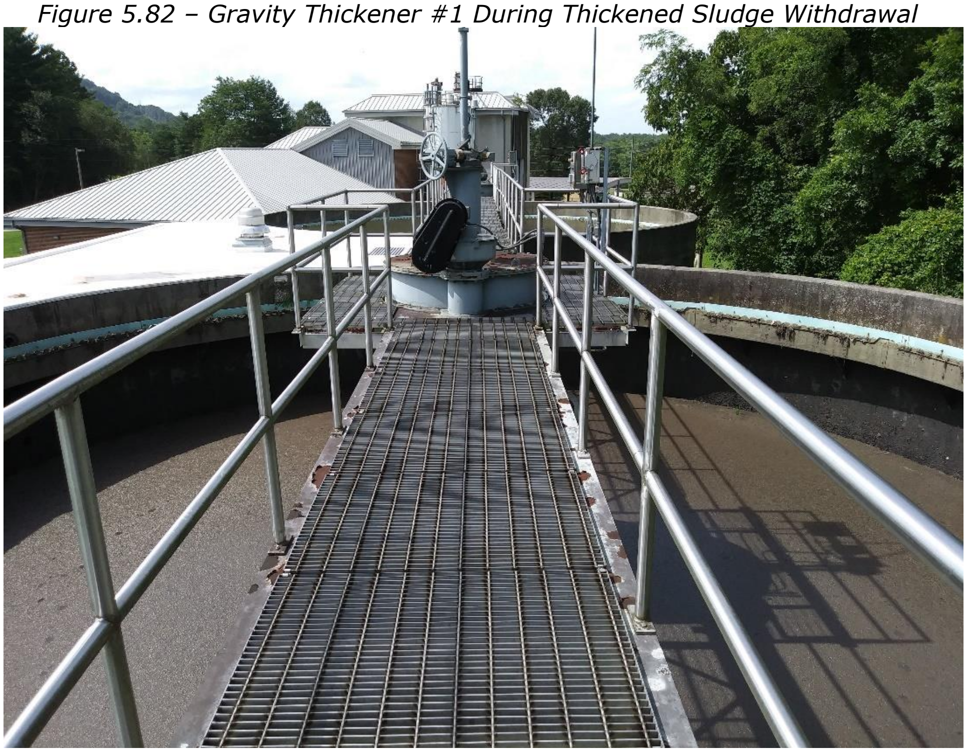


*Figure 5.83 – Gravity Thickener #2 During Thickened Sludge Withdrawal*

The maintenance and eventual replacement of the gravity thickener mechanisms will be required, however, rebuilding of the existing mechanism may result in a significant cost savings.

It is recommended that the existing gravity thickener mechanical and drive mechanisms be fully inspected by the equipment manufacturer with recommendations provided to rehab the existing equipment to increase the long-term service life of the equipment. These recommendations should be considered as part of long-term capital improvements and any necessary expansion of the existing WWTF to accommodate future loading conditions.

As part of rebuilding existing drive systems, it is recommended that their surfaces be sand blasted and coated with a high-performance coating system to protect equipment from corrosion and provide long term reliability.

Within the thickening building, the existing BFP feed pumps were installed in 2001 and appear to be in good visible condition. Pumps of this type will typically last 25 years or more if well maintained and operated. It is not anticipated that any significant immediate improvements are required. However, as part of any future upgrades of the dewatering facility, the existing pumps should be replaced. The existing isolation valves, on the BFP feed pump suction piping, are located between BFP feed pumps 1 and 2, and

2 and 3 per **Figure 5.84** below. The location of these isolation valves does not allow the use of any one of the three BFP feed pumps when withdrawing sludge from one of the gravity thickeners. It is recommended to relocate the BFP feed pump suction isolation valves between each gravity thickener and the closest respective BFP feed pump to allow any one of the three pumps to be used.

*Figure 5.84 – Existing BFP Feed Pump Suction Piping*

## 5.13.3 Structural Condition Assessment

The existing Sludge Thickening facility is a below / above grade C.I.P. reinforced concrete liquid containment structure and footprint that includes two circular shaped tanks. The facility construction is primarily above grade and consists of C.I.P. reinforced concrete wall and slab elements. Note the following:

- Vertical surfaces of walls appeared to be in an aged condition and were plumb and level, however, at multiple locations of exterior vertical and horizontal surfaces, cracks were observed around the perimeter of the tanks (ref. **Figure 5.85**, **Figure 5.86**, & **Figure 5.87**). These cracks varied in length and were intermittent in location but appeared to propagate up from the base slab wall joints that appeared to be above the grade line. Interior surfaces of the walls were not accessible due to the facility remaining in operation.
- Top surface of the tank base slab could not be reviewed due to the facility remaining in operation. In addition, record drawings did not indicate the facility is supported on timber piles.
- Top surface of the exposed areas of the Control Room base slab appeared to be in good condition. Top surfaces appeared to be level and plumb except where sloped to drain as intended.
- Bottom surfaces of the elevated roof slab area appeared to be in good condition. The bottom surfaces appeared to be level and plumb and primarily free of surface defects except for single linear cracks observed in slab panels. Cracks appeared to be larger than hairline width and length spanning from beam to wall across the width of the slab (ref. **Figure 5.88**). The City has recently replaced the roof membrane system for the thickening building, which will help prevent further progression of existing cracks in the roof slab.
- Vertical and bottom surfaces of elevated roof beams appeared to be in good condition and plumb and level. Exterior surfaces of the elevated roof beams and slabs were not observed due to the existing roofing membrane in place.
- At the exterior wall cracks in the tanks, efflorescence was observed.

*Figure 5.85 – Gravity Thickener #1 Vertical and Horizontal Cracks at Front*

*Figure 5.86 – Gravity Thickener #1 Vertical and Horizontal Cracks at Rear*

*Figure 5.87 – Gravity Thickener #2 Vertical and Horizontal Cracks*

*Figure 5.88 – Thickening Building Cracks in Elevated Roof Slab*

At the top of the tanks existing steel access walkway platform, multiple rust spots were observed on the support framing (ref. **Figure 5.89**).

*Figure 5.89 – Gravity Thickeners Rust on Top Center Walkway Framing*

Recommendations for this facility include the following:

- Areas where interior cracks in slabs were observed should be monitored periodically to verify additional cracks are not developing and existing cracks are not increasing in width and / or length. The City should plan to repair cracks within the next 5 – 10 years to prolong the facility level of service. Repair the cracks with a cementitious repair mortar. In addition, repair may include addition of new steel beams to span from existing C.I.P. reinforced concrete beams to the existing exterior wall.
- During the next maintenance shutdown allowing access to the interior wall and base slab surfaces that were unable to be observed during this review, verify interior cracks, delaminations or spalls do not exist. Verification of whether these potential conditions exist could require repairs noted above, in addition to repair mortars or concrete mixes.
- Areas where exterior cracks in walls were observed should be repaired within the next 2 – 5 years. Plan to excavate around the tank walls of the facility to verify cracks do not extend below grade. Repair of cracks should include a low viscosity, hydrophilic expanding polyurethane injection chemical grout adhesive system and polymer modified cementitious mortar.

- Due to the quantity and severity of the cracks observed consideration of an interior / exterior coating system should be evaluated.

### 5.13.4 Electrical Condition Assessment

The existing electrical equipment within the sludge thickening building appeared to be in good condition, and no changes to the existing equipment are recommended. It was noted that the existing sludge pump VFDs were recently replaced this year. The new sludge pump VFDs are in good condition and are expected to have a typical operational life of 20 years.

## 5.14 Sludge Dewatering

### 5.14.1 Purpose and Description

Thickened sludge from the gravity thickeners is dewatered by two 2-meter Sernagiotto Technologies BPF 2000 WR 15 belt filter presses. The belts on each belt filter press are 82.7-inches wide and 88′ – 3″ long, each. Each belt filter press consists of two equal-sized belts. The City currently dewaters thickened sludge periodically when one of the two existing gravity thickeners has reached the maximum sludge blanket level. During dewatering operations, thickened sludge is pumped from the thickener building to the sludge/polymer mixing tank at the head of each belt filter press.

The thickened sludge mixes with polymer pumped from the adjacent polymer room to aid in dewatering and it is then fed onto the belt filter press. Once on the belt filter press, the thickened sludge is first dewatered by gravity in the gravity section of the belt. The sludge then enters the wedge section of the belt filter press where the two belts join together to "wedge" the sludge together and further reduce the water content of the sludge. Finally, the sludge enters the pressure zone of the belt filter press where it is under high pressure between the two filter belts as it passes through series of rollers under tension to further extrude remaining water content out of the sludge. The filter belts are washed by a belt spray wash system after sludge is discharged from the belt filter press. Belt wash water is supplied by the in-plant non-potable water system, and pressure is boosted by two 20 HP end suction wash water pumps. Spent belt wash water is returned to the Influent Pumping Station via the in-plant sanitary sewer system.

The dewatered sludge exits the belt filter presses via a discharge chute onto a Serpentix Model H heavy duty dewatered sludge conveyor. The sludge conveyor has a capacity of five Tons/hour assuming a sludge density of 65 $lbs/ft^3$ per the original design. The dewatered sludge is discharged to the product storage bay under the solids processor room. The City no longer uses the lime stabilization process that was constructed in 2001 and bypasses this system by discharging the dewatered sludge directly to the product storage bay. The dewatered sludge would have originally been discharged into the solids processor by the sludge conveyor, where it would be mixed with lime, allowed to react with the sludge for a predetermined time prior to discharge to achieve a stabilized biosolids product meeting 40 CFR Part 503 regulations for

Class A land application. The City bypassed this system by constructing a track system under the solids processor to move it out of the service position under the sludge conveyor discharge. The finished biosolids product under current operations does not meet the minimum criteria under 40 CFR Part 503 to qualify for land application disposal methods since it is not stabilized to at least Class B requirements using one of the approved methods.

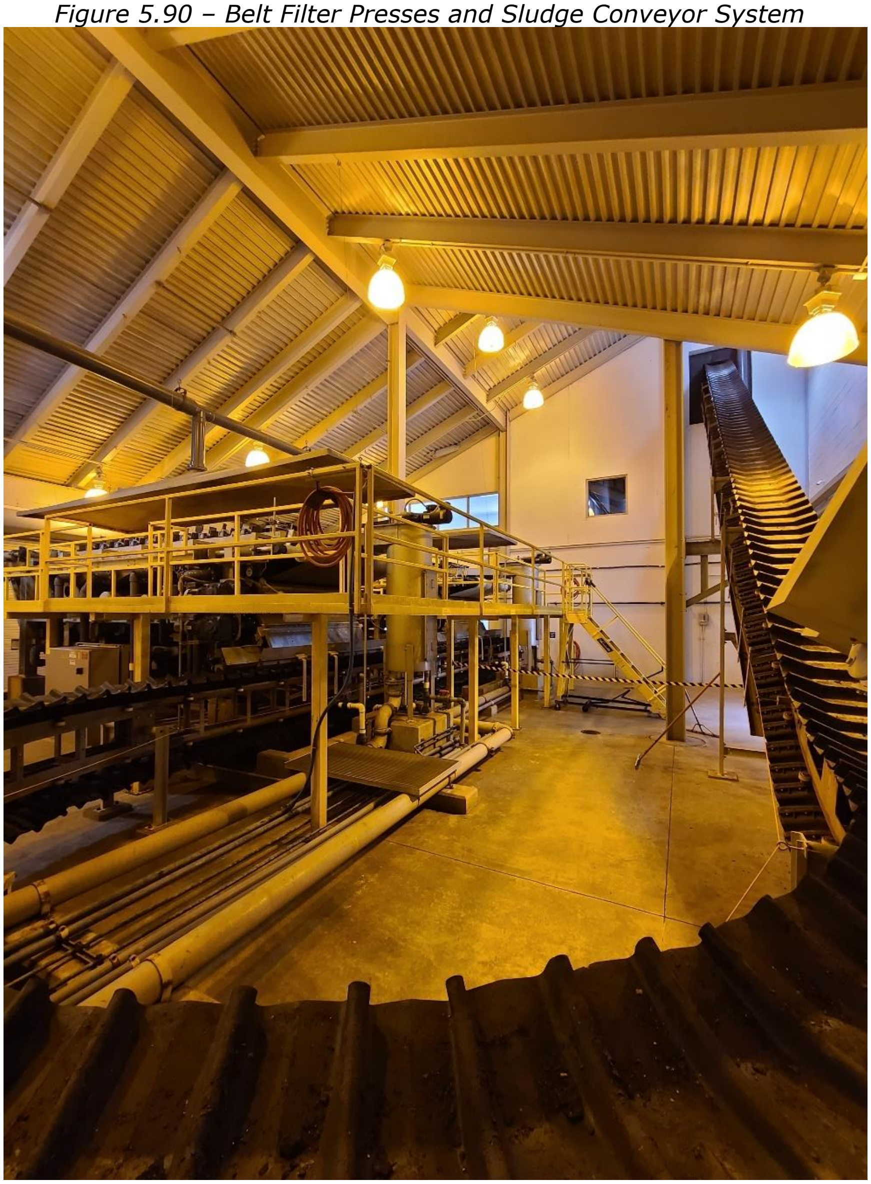
### 5.14.2 Equipment and Process Condition Assessment

The existing Belt Filter Presses were fully functional and achieving good dewatering performance. The Belt Filter Presses appear to be well maintained and generally in good condition (ref. **Figure 5.90**). It should be noted however that belt pressing operation has been a significant source of odors at the facility. Odor generation during belt pressing is suspected to be the result of the long sludge retention time in the gravity thickeners, which is believed to be causing anaerobic conditions as noted previously. In discussion with facility staff, it was noted that operation of the Belt Filter Presses is burdensome, and start-up requires extensive oversight. The Belt Filter Presses are operated manually from the dewatering building, with very little process automation. An evaluation of the desired belt filter pressing schedules and process automation is recommended to identify facility modifications to improve process operation and dewatered cake consistency.

Facility staff had just recently replaced the belts on Belt Filter Press #2 prior to the site condition assessment, and it was noted that the belts on Belt Filter Press #1 were replaced shortly after the site condition assessments. Facility staff also indicated that the existing roller bearings need replacement on each BFP. It is understood that the availability of spare parts is becoming more of an issue due to manufacturer availability and the age of the existing equipment. Therefore, it is recommended that consideration be made to replace this equipment within the next 5 – 10 years if availability of replacement parts continues to be a concern.

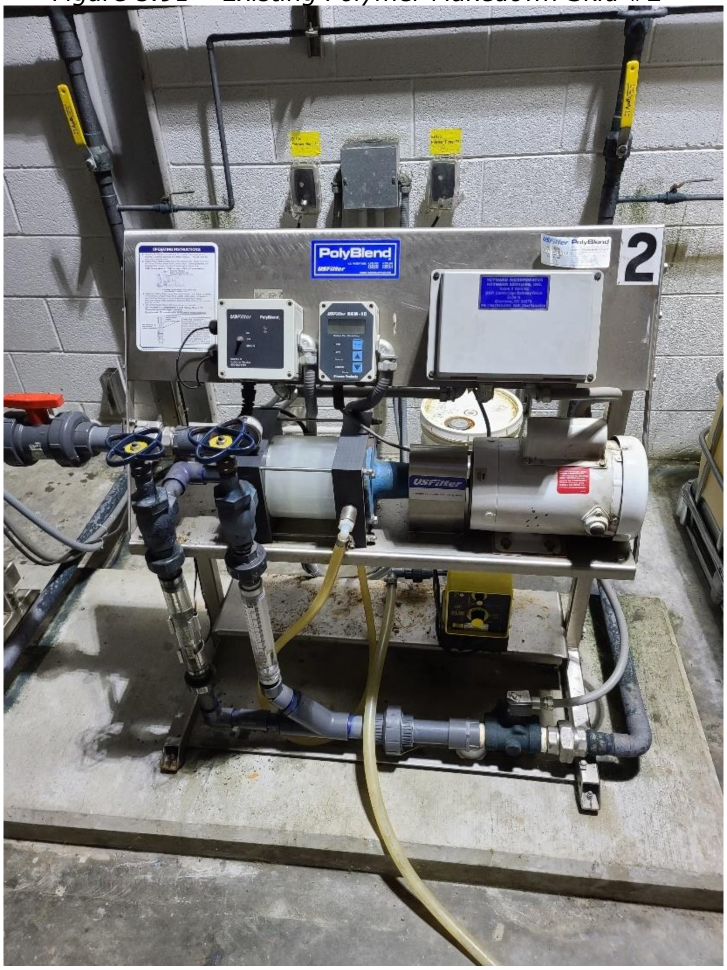
The existing Serpentix Model H dewatered sludge conveyor was reported by operations staff to need replacement. The sludge conveyor was provided as part of the 2001 expansion and therefore is reaching the end of its anticipated service life of 20 years. The sludge conveyor support structure appears to be in good condition, however the conveyor belt, main drive chain, rollers, and bearings need replacement. The equipment manufacturer, Serpentix, still produces this model of sludge conveyors. It is recommended that the City engage with the equipment manufacturer to inspect and replace the major components of the sludge conveyor system.

*Figure 5.90 – Belt Filter Presses and Sludge Conveyor System*



The two existing USFilter PolyBlend polymer makedown systems were operating in good condition at the time of the site condition assessments, however it appears the existing equipment is nearing the end of its reliable useful life (ref. **Figure 5.91**). Small support equipment skids of this type typically have an anticipated useful life of 20 years. Evidence of past repairs to the polymer makedown skids was observed, including replacement of one of the dilution water rotameters. It is recommended that the polymer makedown skids be replaced within the next five years to ensure reliable polymer feed to the belt filter presses. It is recommended that the City consider new polymer makedown skids with automated control functionality to provide precise and consistent solution strength and to simplify BFP start-up operations.

*Figure 5.91 – Existing Polymer Makedown Skid #2*



As noted previously, the existing lime stabilization equipment within and adjacent to the Dewatering Building is no longer in use. The existing equipment associated with this process may be removed or demolished to make room for future facility improvements whenever necessary.

### 5.14.3 Structural Condition Assessment

The existing Sludge Dewatering building is a partial two-story structure with a rectangular shaped footprint with the second story over north end of the building. The building construction includes C.I.P. reinforced concrete, concrete masonry units (CMU) and structural steel elements. Note the following:

- C.I.P. reinforced concrete construction observed included interior foundation slabs-on-grade, partial sections of exterior and interior walls, and beams and elevated slabs supporting the Control and Process rooms.
- Record drawings indicated the building shallow foundation includes C.I.P. reinforced concrete spread footings and continuous wall strip footings. The drawings did not indicate the building is supported by timber piles.

- Record drawings and CMU construction observed included lower level interior walls and upper level interior and exterior walls.
- Structural steel construction observed included steel columns supporting steel roof framing and the standing seam metal roofing.

Regarding the C.I.P. construction observed note the following:

- Vertical surfaces of walls and beams and horizontal bottom surfaces of beams appeared to be in good condition and were plumb, level and primarily free of surface defects.
- Top surfaces of exposed slabs-on-grade appeared to be in good condition. Top surfaces appeared to be level and plumb except where sloped to drain as intended. Minor areas of linear and map crack patterns were observed in top surfaces of the concrete floor slab-on-grade in the Belt Filter Press and Polymer Rooms. These cracks appeared to be hairline in width and varied in length. It did not appear the slab surfaces each side of these cracks had deflected or settled differentially vertically.
- Top surfaces of the Control and Processing Rooms elevated floor slab areas appeared to be in good condition. Top surfaces appeared to be level and plumb and primarily free of surface defects. The floor slab finishes consisted of exposed concrete in the Processing Room and vinyl composition tile in the Control Room. In the room with tile floor finish no defects were observed in the finishes.

Regarding the CMU wall construction observed the walls were in good condition, however, note the following:

- A couple of locations where either wall step crack patterns along mortar joint lines, vertical through block crack or separation from adjacent intersecting walls were observed at exterior and interior wall locations. It did not appear interior cracks observed were through wall cracks and regarding exterior wall cracks, existing veneer prevented verification of this condition. Note the following:
    - Control Room - vertical separation cracks were observed at the south end of the east wall (ref. **Figure 5.92**).
    - Control Room – vertical separation cracks were observed with the walls common with the access stairs.
    - Control Room – vertical through block cracks were observed in the sill below the north wall window and at the upper right corner of the window (ref. **Figure 5.93**).
    - Belt Filter Press Room – step crack was observed in the north wall (ref. **Figure 5.94**).

- At the north entry pedestrian door of the east wall of the Press Room the CMU lintel has degraded to level of lost section of bottom face shell, mortar, and grout fill (ref. **Figure 5.95**). Discussions with the City indicated this CMU lintel was modified to install a larger door.

*Figure 5.92 – Dewatering Building Control Room Vertical Separation Cracks*

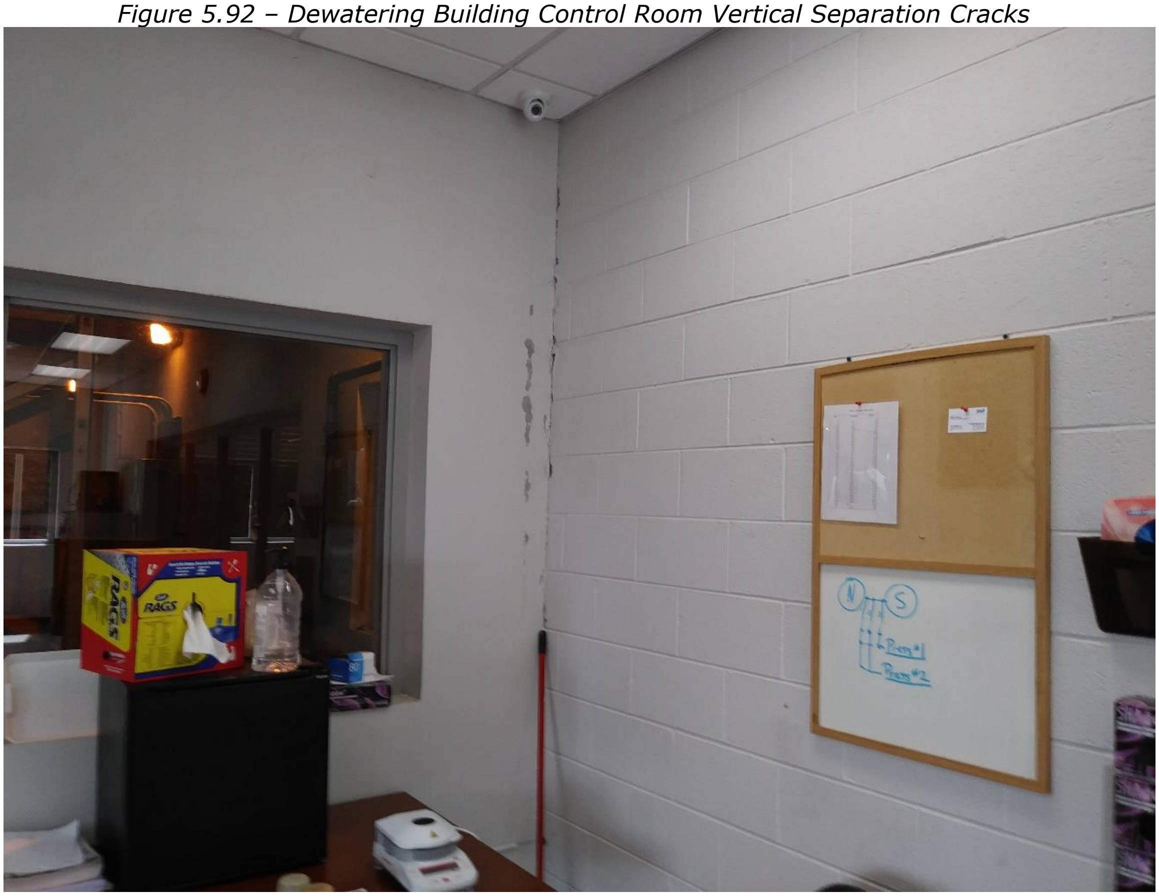


*Figure 5.93 – Dewatering Building Control Room Through Block Crack*

*Figure 5.94 – Belt Filter Press Room Step Crack in North CMU Wall*

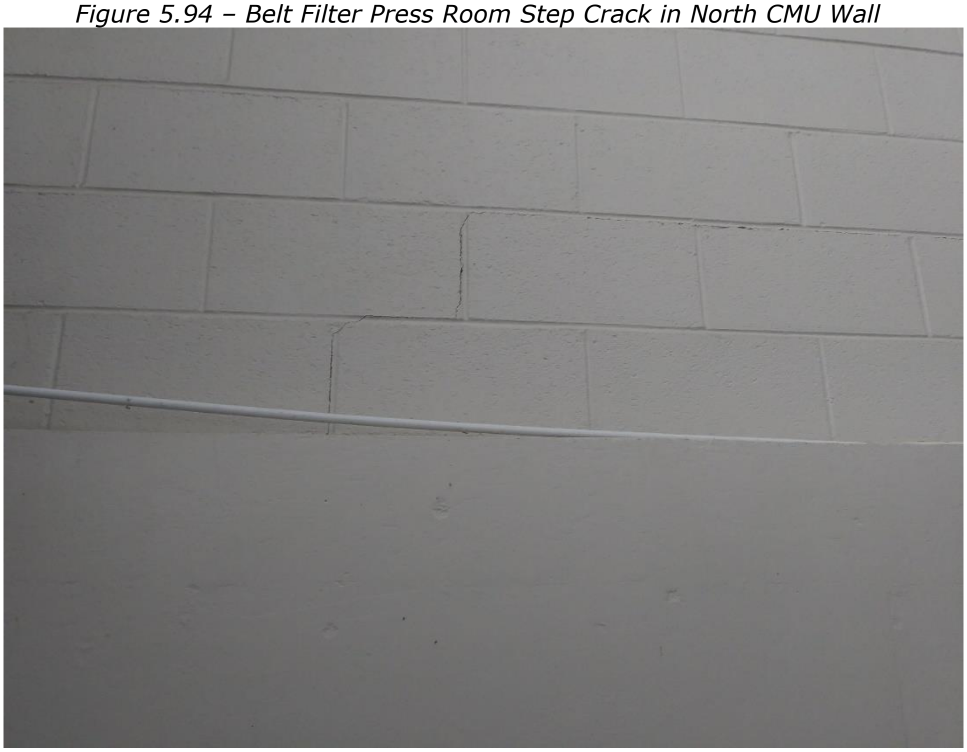


*Figure 5.95 – Belt Filter Press Room CMU Lintel Degradation*

Regarding the structural steel, the steel column, beam and bracing assemblies observed appeared to be in good condition.

Recommendations for this facility include the following:

- Areas where cracks in slabs and CMU walls were observed should be monitored periodically to verify additional cracks are not developing and existing cracks are not increasing in width and / or length.
- Remove the existing damaged CMU fragments and install a new reinforced grout filled CMU lintel beam within the next year for the Press Room east wall entry door.

### 5.14.4 Electrical Condition Assessment

The existing electrical equipment in the Dewatering Building serves the existing gravity thickening equipment, belt filter presses, BFP feed pumps, BFP wash water pumps, polymer makedown systems, and various other equipment associated with the abandoned lime stabilization process. The existing power distribution equipment located within the Dewatering Building electrical equipment room appeared to be in good condition and is expected to continue to operate satisfactorily. The various other power panels and electrical equipment associated with process equipment are typically packaged systems provided by the equipment manufacturer.

## 5.15 Biosolids Storage

### 5.15.1 Purpose and Description

Dewatered biosolids from the belt filter presses is stored under the covered biosolids storage shelter until it is removed for disposal at the Haywood County municipal solid waste landfill. The covered biosolids storage shelter is also currently used to store dewatered water treatment facility residuals prior to their disposal at the Haywood County landfill. Any drainage from the stored biosolids and water treatment facility residuals is captured by a trench drain running the length of the shelter between the shelter and the dewatering building. Drainage from the biosolids storage shelter is then directed to the plant drain system to be returned to the Influent Pumping Station.

### 5.15.2 Equipment and Process Condition Assessment

The existing facility is in good condition and provides for sufficient storage of the current biosolids production. It was noted that the existing structure would require a new metal roof to maintain weather protection of the biosolids cake with the existing structure receiving a new protective coating.

### 5.15.3 Structural Condition Assessment

The existing Biosolids Storage facility is a single story above grade open steel structure with a rectangular shaped footprint. The facility construction includes C.I.P. reinforced concrete and structural steel elements. Note the following:

- C.I.P. reinforced concrete construction observed included slab on grade. Record drawings indicated the steel exterior columns are supported by shallow foundation footings. Record drawings did not indicate the facility is supported on timber piles.
- Structural steel observed included wide flange section beams and columns supporting corrugated steel roof decking panels.

Regarding the C.I.P. construction observed note the following:

- The facility was in service and could not be thoroughly reviewed except from the perimeter of the facility. Top surfaces of the exposed areas of the exterior slab-on-grade able to be observed appeared to be in good condition, although there are areas with extensive cracking. Top surfaces appeared to be level and continuously sloped to drain as indicated on the record drawings. Below grade footings could not be observed but columns appeared to be level and plumb.

Regarding the structural steel, the steel beam and columns assemblies observed appeared to be in good condition except where it appears columns have been hit by the front-end loaders. In addition, multiple locations were observed where paint coating was delaminating, and rust was observed. Multiple rust spots were observed on the top surface of the existing roof deck (ref. **Figure 5.96**).

Recommendations for this facility include the following:

- Remove and replace the existing roof deck within the next 1 – 5 years.

*Figure 5.96 – Biosolids Storage Shelter Roof Decking Deterioration*

## 5.16 Lightning Protection

During the facility walk-throughs, it was noted that the existing facility does not have lightning protection systems installed. Since the location of the wastewater treatment facility is prone to lightning, it is suggested that a robust grounding system be installed underground, and lightning protection be extended to those structures and site light-poles throughout. Although, the best lightning protection methods and installations cannot fully protect against direct strikes, they have been proven to minimize and mitigate the effects of nearby lightning strikes. Additionally, it is advisable to install Surge Protective Devices

(SPD) on all power distribution equipment such as switchboards, panelboards, motor control centers, stand-alone motor controllers, control panels, etc.

## 5.17 Instrumentation and Control (ICS)

### 5.17.1 Purpose and Description

The Instrumentation and Control System (ICS) serves as the operators' "window into the process" and is essential to maintaining treatment process performance. The ICS can be segmented into four primary subsystems for evaluation:

1. Field Devices

   Field devices are comprised of electrical or motor controls (e.g., variable frequency drives, motor starters, motor-operated valves) and instrumentation (e.g., pressure transmitters).

2. Programmable Logic Controllers

   This subsystem is comprised of the control panels that are strategically distributed throughout the plant.

3. Communication

   This subsystem includes the communication infrastructure supporting the control networks (those involving PLCs and SCADA servers).

4. User Interface

   User interfaces include panel-mounted touchscreen panels, human-machine interface (HMI) applications, remote annunciation systems (e.g., WIN-911 software) and peripheral software (e.g., databases, key performance indicators and dashboards).

Although a comprehensive cybersecurity assessment was not performed, several cybersecurity aspects were considered during the plant walkthrough and will be noted throughout this section.

### 5.17.2 Field Devices Condition Assessment

The plant site and process areas are kept clean which will aid in extending the life of field instrumentation and electronic equipment. While a comprehensive evaluation of all installed field instrumentation was not performed, the performance of various plant processes (e.g., aeration basins) can be improved by adding instrumentation to increase the operations staff's visibility.

PLC-based control panels are strategically distributed throughout the plant to monitor and control clusters of processes and equipment. The equipment control implementations are relatively consistent throughout the plant with pilot operators (e.g., pushbuttons, selector switches, indicating lights) generally mounted on the control panels and the equipment motor controllers (MCC-resident motor starters, VFDs, equipment-specific control panels).

Access to the plant is controlled by a chain-link fence surrounding the property with an automated gate with a keypad and call box at the main plant and dewatering facility entrances. Inside the plant the process building doors and access to electrical rooms are secured via traditional keyed door locks, although there are some equipment panels and controls outside exposed to potential unauthorized access.

### 5.17.3 Programmable Logic Controller Assessment

Supervisory control of the equipment is performed at PLC-based control panels that are distributed throughout the plant. The control panels are well-built, neat and clean, with consistent components and wiring methods. The older SLC500 processors have been replaced with newer CompactLogix 1769-L30ER CPUs which communicate with the original back plane and input/output modules via the Ethernet/IP protocol using the 1747-AENTR gateway module. Smaller control panels utilize CompactLogix L23E or MicroLogix platforms.

All supervisory PLCs in the plant are manufactured by Rockwell Automation and are programmed using either Logix Designer or RSLogix 5000 software depending on their firmware version. Rockwell Automation's Logix platform is well known and has a very large install base. Thus, support and assistance are widely available for these products. Additionally, spare parts inventory can be reduced as the PLCs utilize similar hardware between installations.

All SLC500 I/O modules are in Rockwell Automation's "Active Mature" lifecycle stage which means the products remain fully supported by the vendor. However, as the SLC500 products continue to age the cost of those modules will continue to rise. These modules do not require replacement unless a catastrophic hardware failure occurs.

PLC discrete input/output (I/O) signal voltage is 120 VAC and is consistent across the PLCs installed throughout the plant. Although 24 VDC signals are safer due to the lower DC voltage, 120 VAC I/O is very common throughout the industry. Additionally, the 120 VAC wires are consistently different in color (red) so the staff can reliably identify the voltage.

PLC control panel installations are relatively consistent throughout the plant contributing to increased system reliability, better operational familiarity, simpler system management, reduced system downtime and an overall lower cost of ownership.

It is critical for the PLC programs to be backed up before and after being revised. The backups should be clearly labeled for version control and should be stored both on-site at the plant on secured portable media for fast access in an emergency and in a secure off-site location, possibly secured cloud storage, for resiliency. Care should be taken to ensure that the content of one location matches the other for version control purposes.

It is recommended that PLC spare parts be stored on-site at the plant for expedited hardware replacement when required. The inventory should be maintained over time and selected staff should be adequately trained to troubleshoot and replace PLC hardware and reload programs, so they can expeditiously remediate critical incidents involving the PLC hardware failure.

### 5.17.4 Communication Assessment

The PLCs communicate with each other and with the SCADA HMI application (running on a server in the Administration Building) using an Ethernet network using Rockwell Automation's Ethernet/IP protocol. Category 5e cable is used for Ethernet communication between PLCs, panel-mounted operator interface terminals (OITs) and variable frequency drives (VFDs) inside buildings. Fiber optic cable is used for the network links routed between process buildings, which protects the communication hardware connected at both ends of the cable from electrical transients because fiber optic cable utilizes light instead of electricity to transmit data.

Each supervisory PLC control panel uses a Hirschmann RS20 managed Ethernet switch to communicate over the Ethernet network. Managed Ethernet switches provide several benefits including increased network visibility (e.g., failure monitoring and diagnostics), port (access) control and data flow control if configured.

Reference drawings for the WWTF indicate the fiber-based ICS network nodes are physically connected in a bus topology, which functions as a single network segment. In a bus topology, communication to multiple nodes will lose communication ability if the fiber optic cable fails, depending on the location of the failure. However, per discussions with the City and Fortech, Inc. (the City's SCADA integrator), the fiber-based ICS network was converted to a ring topology after the original construction of the facility for improved resiliency and redundancy. In a ring topology, data is transmitted between all network nodes (i.e. process areas) in a circular pattern. All process areas, including the solids handling facility and the emergency generator, are connected to the ICS network ring. The Hirschmann Ethernet switches located in every PLC control panel utilize Hirschmann's HIPER Ring protocol to administer the control network such that no communication is lost if any single link between nodes fails. This provides improved network resiliency and redundancy compared to a bus topology by enabling detection of the link segment failure, while still maintaining PLC communication while the issue is remediated. Discussions with Fortech, Inc. indicated that all fiber optic network cables between process areas (nodes) are installed within embedded and encased duct banks to protect network cables from accidental damage.

The ICS network ring topology is easily maintained and is scalable for future modifications and expansions to the existing facility.

It is recommended that the City continue to maintain the ICS network using a ring topology. Future expansions to the ICS network to serve additional nodes should be integrated into the ring topology to provide redundant communication paths between nodes and increased resiliency against network failures. It is also recommended that one or two spare Ethernet switches be stocked on-site at the plant for expedited hardware replacement when required. Selected staff should be trained to configure the switches, so they can expeditiously remediate critical incidents involving an Ethernet switch hardware failure.

### 5.17.5 User Interface Assessment

Plant staff use Rockwell Automation PanelView Plus panel-mounted operator interface terminals (OIT), mounted on the supervisory PLC control panel doors, to monitor and control the processes and equipment that are connected to local PLC. Additionally, the staff uses a Dell T430 desktop server located in the Administration Building Control Room running a Rockwell Automation FactoryTalk View SE SCADA HMI application to monitor (only) the processes and equipment across the plant.

There are several important aspects to consider during a user interface evaluation, including architecture resiliency, visibility and functionality, data utilization and cybersecurity.

Architecture Resiliency

Local process and equipment monitoring and control via the panel-mounted OIT enables local operational visibility even when the control network has failed. However, there are some disadvantages with implementing only local automatic control of processes and equipment including:

1. It is more complex, generally requiring inter-PLC communication, to acquire process data from other operational systems. For example, if a locally-controlled pump station is filling up a tank in an area that is controlled by another PLC, the local PLC would need to explicitly acquire the tank level from the other PLC to display it on the OIT.
2. Panel-mounted OIT software typically offers fewer features and less functionality than a typical SCADA HMI software (e.g., FactoryTalk View SE). This inherently limits the capabilities of the OIT application relative to the HMI application, and constrains operator interaction. For example, process performance type analysis is not commonly included in an OIT application. In addition to the software limitations, application developers are constrained by the device's hardware resource (e.g., CPU, hard drive, RAM) limitations.

3. OIT applications typically offer lower access control and authentication than is possible with SCADA HMI software; OIT applications commonly implement a numeric PIN entry or no security at all. In contrast, SCADA HMI software supports user accounts with passwords and several vendors also support peripheral authentication such as biometric scanners or card/badge readers.
4. Hardware failure can cause extended downtime of automatic process/equipment control ability. Lead time for OIT panels, unless already on a vendor or local supply house shelf, is longer than that for a desktop computer. Additionally, the downtime duration will be a function of the availability of the systems integrator that supports the system unless plant staff have been trained how to replace the panel and reload the application.

Extending the existing FactoryTalk View SE SCADA HMI application to control the various plant processes and equipment, from a resiliency perspective, offers automated process redundancy: if an OIT panel fails, operators can continue to monitor and automatically control those processes while the OIT is replaced. Additional benefits of SCADA HMI applications are discussed in the subsequent sections below.

Visibility and Functionality

The OIT and SCADA HMI applications provide operators with a "window into the process" enabling them to continuously monitor and control processes and equipment. User interface configuration/implementation accuracy and consistency are essential to facilitate operator effectiveness and system reliability. Although the applications are relatively consistent, it was reported during the site walkthrough that several of the FactoryTalk View SE HMI application's equipment status and process measurements do not accurately reflect actual field conditions.

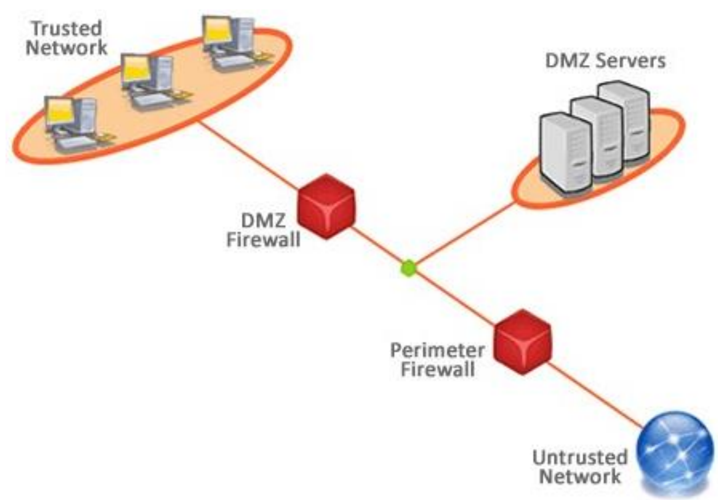
We recommend identifying and correcting all erroneous process/equipment data in the SCADA HMI application and extending the application to control the various plant processes and equipment which, from a functionality perspective, provides:

1. Better visibility and performance: additional data from related processes can be more easily included on the process displays, and key performance indicators (KPIs) and real-time analysis can be embedded in the HMI application to facilitate more effective process operation.
2. Incident response: operators can control the entire plant in a single location which enables faster alarm and incident response.
3. System extensibility: Secure mobile and remote access to the HMI application, embedded data access (e.g., O&M manuals, as-built drawings), data analysis, and peripheral system (e.g.,

Laboratory Information Management System (LIMS) or Computerized Maintenance Management System (CMMS) software) integration extend the functionality of the SCADA system.

Modern SCADA systems are no longer contained inside plant walls. They can provide secure remote access for operators to monitor and control live processes from outside plant walls using virtually any authenticated and authorized Ethernet-enabled device such as a computer, tablet or phone. Automatically generated trends and reports can also be periodically emailed to select personnel for detailed analysis. Off-site operators and supervisors can be notified of critical alarms via text or email. Informational dashboards can also be leveraged to quickly provide convenient snapshots of a treatment process or virtually any combination of measurable information.

Plant staff currently use WIN-911 software for remote alarm notification/acknowledgement, and the City's SCADA integrator (Fortech Inc.) remotely accesses the SCADA HMI server using LogMeIn software. Although LogMeIn uses an encrypted connection between client and server applications, it is not a recommended practice because a potential threat actor can gain access to the entire plant network if the LogMeIn account becomes compromised. We recommend implementing a special-purpose intermediate network called a "DMZ" between the plant control system and external networks (including but not limited to the public internet).




The general concept of this architecture is that all data exchange between the plant control network and an external network is done via the intermediate DMZ network. There is no direct interaction between the two networks. Computers and applications residing within the DMZ have limited connectivity to the two networks, with connectivity being controlled by the respective network-associated firewalls. One firewall governs communication between the plant control network and the DMZ, while the other firewall governs communication between the external network and the DMZ.

Remote monitoring and control can expedite alarm incident response and improve process awareness. However remote access must be done securely to prevent unauthorized access to the SCADA system. A DMZ network in conjunction with appropriately configured firewall appliances, data/traffic flow control and well-defined user account authentication, authorization and permission limitations in accordance with industry standards such as NIST SP800-82 can facilitate secure remote access for a more flexible SCADA system user interface.

Data Utilization

Data utilization, in this context, is an evaluation of how the data made available by the SCADA system is effectively used. According to plant staff, the FactoryTalk View SE SCADA HMI application currently collects and stores historical SCADA system data over the last (rolling) year. While this is sufficient to support trends embedded in the HMI application, it does not provide enough data to establish a reliable process performance history for modeling, engineering or analysis.

We recommend installing a database to provide robust data access and long-term storage of SCADA system historical data. This SCADA historical database should be regularly backed up on permanent offline (i.e., not connected to a network) storage. The process data should be made available to plant staff for operations improvement through graphical trends and charts, data analysis and dashboards, automated reports and other tools. Similar to any other data-driven system, the specific architecture should be carefully designed in accordance with known and (currently) unknown user needs:

1. Who - Operators, Maintenance, Managers, Engineers, Finance staff - will require access to the data?
2. What data will different users be interested in?
3. When will different users request the data? Not all use cases will be known at initial system creation.
4. Where (e.g., in the plant, at home) will users be when they request the data?
5. Why do specific users want the data?
6. How will the system deliver the requested data?

We recommend the implementation of distributed SCADA system dashboards and reports. Dashboards are concise graphic displays that present actionable information based on real-time data (Operational Dashboards) or historical data (Performance Dashboards) that help operations and management staff:

- Visualize key performance indicators (KPIs)
- Improve process performance and efficiency over time
- Make informed operational decisions
- Identify abnormal conditions before they detrimentally affect treatment processes
- Plan preventive maintenance

- Energy management

Dashboards and reports can assist staff with identifying real-time process characteristics and expediting incident response, as well as aiding in analyzing historical system performance to enable longer-term process tuning for steady operational efficiency improvement.

## 5.18 Site/Civil Assessment

It was observed during the assessment survey, that the treatment plant site and grounds were well-maintained. There was an excellent stand of grass that was mostly free of weeds and other invasive plants. Further inspection proved the grass was neatly trimmed at most structures, tankage, and curbs. The security fence around the site appeared to be in good condition and the security gates were observed to function properly.

1. Specific (minor) areas of concern included:
    - It was observed that one area, south of the Effluent Filters was in need of refreshed ground cover, though this was likely a result of recent construction efforts in the immediate vicinity.
    - Another area in need of grass trimming around the tank was adjacent to the Disinfection Facility. In this case, however, it appeared large stones in the vicinity may have prevented trimming.
    - At the Administration Building, a vertical crack was observed in the exterior retaining wall at the cold joint with the Administration Building’s west wall (ref. **Figure 5.97**).
    - At the Utility Building, a small sinkhole and cracking was noted at the pedestrian entry door (ref. **Figure 5.98**).

*Figure 5.97 – Vertical Crack in Administration Building Retaining Wall*

*Figure 5.98 – Sinkhole at Utility Building Door*

2. The observation noted as well that all grade transitions/slopes were being maintained; only a few erosion issues were observed.

    - With regards to erosion concerns, one issue was located east of the Disinfection Facility. Importantly, this erosion concern appeared to be minimally problematic and likely does not require attention at this time.
    - Also observed was the significant presence of wheel tracks north of the existing Aeration Basin #2 (ref. **Figure 5.99**), which damaged the appearance of the grounds. The observer assumes this area serves as a service road for access to the north side of the Aeration Basins. After discussing the excessive presence of "wheel ruts" with personnel on location, it appears this area stays wet and is susceptible to wheel tracks any times a vehicle traverses it.

- For that reason, it is recommended that any standing water found in these wheel tracks be tested for chemicals, sewage, and other pollutants or contaminants in order to ensure there is no leak in the adjacent structures.
- It was observed there are numerous cracks in the exterior wall of Aeration Basin #2 to include excessive "bowing" to the exterior wall. This may be a sign of a continuous leak from the structure.
- Based on the results of the suggested tests, remedial work to the integrity of the wall may be necessary. If the tests do not demonstrate a breach of contaminants, then minor re-grading may be called for to eliminate water ponding due to natural run-off.

*Figure 5.99 – Wheel Tracks North of Aeration Basin #2*

- Observed major concerns: During the assessment, the item of highest concern was the settlement between structures, piping, and tankage. Excessive settlement was found in the following locations:

1. The sidewalk west of the Recycle Pumping Station. Here, two valve operating nuts extended above the sidewalk surface producing a significant trip hazard (ref. **Figure 5.100**). It should be noted these trip hazards were painted "safety yellow", however they should be addressed properly so as to not be a continuous hazard.
2. A sewer manhole located in the site access road west of the Influent Pumping Station (ref. **Figure 5.101**) appears to have settled in the road by a significant degree. This manhole should be investigated to determine if piping into the manhole has been damaged to any extent, and corrective action should be taken to resolve any underlying issues, returning the manhole to correct elevation
3. Blower piping exiting the Blower Building shelter does not appear to rest in the supports originally cast for their support (ref. **Figure 5.102**). At the 90-degree bend, it appears the concrete support and pipe have bonded, and the support/pipe combination have lifted from the slab as the slab settled. A newer metal saddle support appears to have been added to support both the pipe and concrete support. Located between the 90-degree bend/support combination and the wall pipe entering the Aeration Basins, there is another concrete saddle support (ref. **Figure 5.103**). At this location, the pipe does not rest in the support and the existing concrete support appears to have had a strap added across the pipe to attempt to resolve/conceal this issue at some point but is currently missing. Given the unusual support combination, it is recommended that a careful analysis of the situation be conducted to confirm the pipe is supported properly as a failure of this pipe would be catastrophic to the plant process.

The information presented above regarding the site/civil assessment was discussed with the City following initial submittal of this technical memorandum. During these discussions, the City noted that they have tested the standing water in the wheel-ruts north of Aeration Basin No. 2 for wastewater indicators. These tests found no indicators of wastewater contamination, and the standing water was determined to be groundwater. Based on these findings, the source of the standing water does not appear to be related to any leaks from the adjacent aeration basins. However, the recommendations from the structural condition assessment of the existing aeration basins still apply to ensure continued successful operation. The City may perform minor regrading to eliminate water ponding due to natural run-off north of Aeration Basin No. 2.

The City also noted that CCTV investigations were performed on in-plant MH #1 after the initial submittal of this technical memorandum. The CCTV investigation identified several areas in need of repair, which the City is in the process of addressing.

*Figure 5.100 – Sidewalk Settlement at Recycle Pumping Station*

*Figure 5.101 – Potential Settlement of Sewer Manhole*

*Figure 5.102 – Slab Settlement at Blower Piping Support*

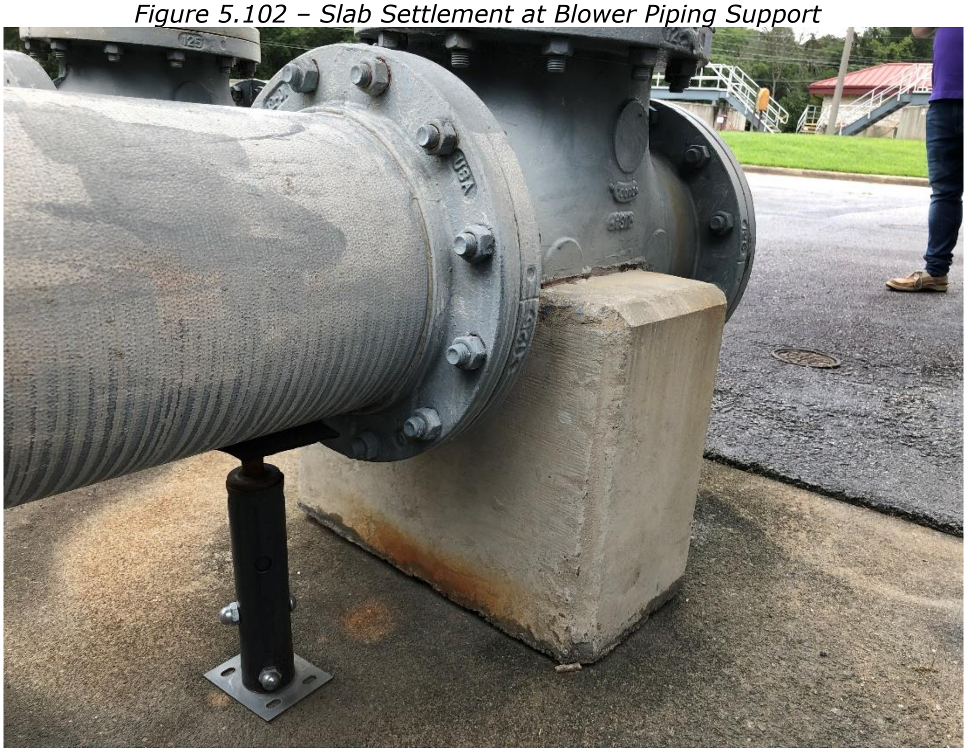


*Figure 5.103 – Slab Settlement at Blower Piping Support*

# 6. SUMMARY

The City of Hendersonville Wastewater Treatment Facility Master Plan is intended to provide a holistic review of the major systems throughout the entire facility to inform recommendations for replacement, rehabilitation, upgrades, and treatment capacity expansion. This Technical Memorandum No. 1 is the first part of this Master Plan and has summarized the following information that will be used and considered throughout the remaining phases of this Master Plan:

1. Data and recommendations from previous engineering studies.
2. Influent, effluent, and treatment process data collected from the City's WWTF to be used for capacity and alternatives analyses.
3. Currently planned capital improvement projects for the WWTF.
4. Existing condition assessments.

## 6.1 Previous Engineering Studies

The previous engineering studies, the *Sanitary Sewer Asset Inventory and Assessment Master Plan Report* and the *Process Capacity Assessment and Plant Expansion Addendum to the SSAIA Master Plan Report*, were reviewed to collect any pertinent data, information, and recommendations that would apply to this Master Plan. Our review of these documents noted four key pieces of information that will inform later phases of this Master Plan. First, the SSAIA report documented influent flow projections for the City's WWTF from 2017 through 2040, which are summarized here in **Table 6.1**. The future flow projections were developed based on historical data and other recent studies, and account for future RDI/I, future population and employee growth, elimination of private WWTP's and septic systems, and the addition of future industrial customers. Per previous discussions with the City, it has been agreed to adopt these flow projections as the basis for future design conditions for this Master Plan.

*Table 6.1 – Master Plan Flow Projections*

| COH Sewer Service Area | 2017 | 2025 | 2040 |
|---|---|---|---|
| Average Annual Flow Projections (MGD) | 3.07 | 4.23 | 5.90 |
| Maximum Month Total Flow Projections (MGD)[1] | 4.00 | 5.50 | 7.68 |

*[1]Based on 5 year average maximum month peaking factor (PF) of 1.30.*

The *Process Capacity Assessment and Plant Expansion Addendum to the SSAIA Master Plan Report* documented the results of the previous limited scope capacity assessment for the City's WWTF. This capacity assessment was primarily focused on the secondary treatment processes at the WWTF. Based on our review, the key conclusions from this report include:

- Recommended reduction of aeration basin MLSS and sludge SRT to alleviate overloading conditions in the secondary clarifiers.
- Provided alternatives for future plant expansion to the projected 2040 design capacity, including:
  - Alternative 1 – Addition of primary clarifiers
  - Alternative 2 – Addition of conventional process treatment train
  - Alternative 3 – Process intensification
  - Alternative 4 – New granular activated sludge treatment train.
- Recommended installation of an EQ basin and provided proposed sizing based on the SSAIA flow projections. The previous EQ basin sizing is summarized in **Table 6.2**, below.

*Table 6.2 – Equalization Basin Sizing Calculations from Process Capacity Assessment*

| Year | 2-Year Storm Peak Flow (MGD) | Permitted Treatment Plant Flow (MGD) | Plant Hydraulic Capacity (MGD) (PF=2.5) | Storm EQ Volume Needed (MG) |
|---|---|---|---|---|
| 2017 | 17.4 | 4.8 | 12 | 0.95 |
| 2040 | 39.4 | 9 | 22.5 | 5.74 |

## 6.2 Data Collection

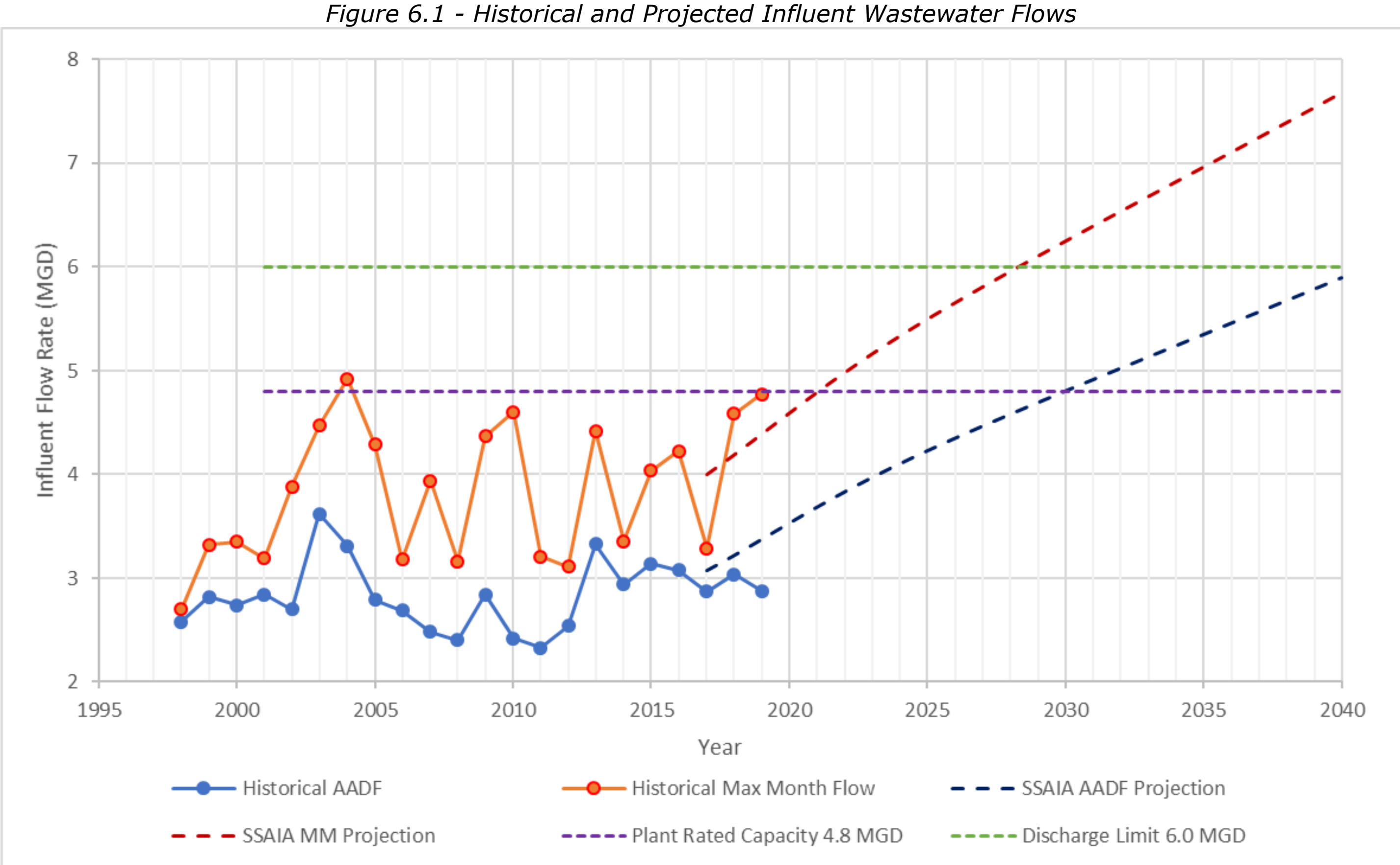
The data collected and presented in this TM will be used to establish the current and future influent loading conditions to the City's WWTF. This information will be used to evaluate current capacity limitations, capacity re-rating possibilities, and future upgrade and expansion alternatives in future phases of this Master Plan. Five years of historical facility data was reviewed for influent flow, influent concentrations, and effluent concentrations.

Historical influent flow data was reviewed to determine the current average day flow, maximum month flow, maximum day flow, and peaking factors, as shown in **Table 6.3** below. This information is important for the evaluation of the current capacity limitations. However, it also provided an informative comparison to the future flow projections presented in the SSAIA Master Plan Report. **Figure 6.1** below shows the comparison of the past 20 years of influent flow data compared to the future flow projections. The current and historical trends do not match the projected rate of increase. However, it is important to note that future projections also include assumptions for private WWTP and septic conversions, as well as potential industrial growth. The future flow projections will continue to be used for this Master Plan, however it is noted that differences between actual and projected conditions may affect the timing of future improvements and modifications.

*Table 6.3 – Historical Influent Flow from 2014 - 2019*

| Parameter | Units | Average |
|---|---|---|
| Average Influent Flow | MGD | 2.99 |
| Summer Average Influent Flow | MGD | 2.82 |
| Winter Average Influent Flow | MGD | 3.29 |
| Maximum Month Flow | MGD | 4.78 |
| Maximum Month PF | - | 1.60 |
| Maximum Day Flow | MGD | 6.28 |
| Maximum Day PF | - | 2.10 |

*Figure 6.1 - Historical and Projected Influent Wastewater Flows*




Daily information for influent $BOD_5$ and TSS concentrations was available throughout the past five years, however the facility does not monitor influent TKN, $NH_3$, nor TP on a daily basis. Influent TKN, $NH_3$, and TP are critical influent parameters for the secondary process design. To offset this lack of data, approximately one year of quarterly laboratory analysis data was collected for $NH_3$ and TP. Data for TKN was also not available from the quarterly laboratory analyses, however it can be predicted from average $NH_3$ with a reasonable degree of accuracy. Average influent concentrations for the major influent wastewater characteristics are presented in **Table 6.4**, below. Conservative adjustments were made for influent TKN and TP values to account for the small sample size of this data. The utilized values for TKN and TP are consistent with typically observed concentrations for average strength domestic wastewater. These concentrations will be used to establish current and future design loading conditions for the City's WWTF throughout this Master Plan.

*Table 6.4 – Average Influent Concentrations for Process Modeling*

| Parameter | Units | Value |
|---|---|---|
| $BOD_5$ | mg/L | 219 |
| TSS | mg/L | 223 |
| VSS | mg/L | 156 |
| TKN | mg/L | 45 |
| TP | mg/L | 7 |

## 6.3 Current Capital Improvement Plan

Prior to the initiation of this Master Plan, the City of Hendersonville has established plans for a number of improvements to the existing WWTF. The currently planned capital improvement projects for the WWTF are summarized in **Table 6.5**. These projects will be considered along with the repair, rehabilitation, replacement, upgrade, and expansion needs for the facility that will be identified and considered throughout this Master Plan. Revisions to or reassessment of these currently planned projects may be necessary as additional facility needs are identified. Revisions to or reassessment of the currently planned projects may include the need to adjust the currently planned scope, adjust the expected timing, or eliminate the project entirely.

*Table 6.5 – Currently Planned Capital Improvement Projects*

| Project | Description | Year | City Allocated Funding |
|---|---|---|---|
| WWTF Aeration Basin #1 Diffusers Replacement | Replace aeration diffuser membranes in Aeration Basin #1 | 2020 | $43,170* |
| WWTF Renovation Project | Various rehabilitation projects | 2021 | $1,370,000 |
| WWTF UV Disinfection System | Replace existing UV Disinfection system | 2023 | $1,794,000 |
| WWTF Sludge Drying System | Reduce landfill costs and increase sludge disposal options | 2024 | $4,109,000 |
| WWTF EQ Basin | 6.0 MG EQ Basin | 2024 | $6,090,000 |
| WWTF 6.0 MGD Expansion | Expand WWTF capacity to 6.0 MGD | 2025 | $5,000,000 |

Note: *Project is completed

## 6.4 Existing Condition Assessments

The existing condition assessments consisted of three one-day facility walk-throughs with lead engineers from civil, process/mechanical, structural, electrical, and instrumentation and controls disciplines to identify repair, rehabilitation, and replacement needs for the WWTF. In addition, discussions and interviews with facility staff were conducted during the facility walk-throughs to gather insight into operational issues, recent repairs made, and desired improvements. Recommendations are made below for repair, rehabilitation, and replacement needs, as well as recommendations for additional evaluations and operational improvements.

**Table 6.6** below summarizes the recommended repair, rehabilitation, replacement needs for the facility. This table also summarizes recommended evaluations and studies related to the condition of existing processes, equipment, and structures. The facility needs summarized in this table have been assigned preliminary priority rankings based on the criticality of each need with respect to continued successful facility operation. Expected timeframes for each facility need have also been established as a preliminary road map for the future develop of the Master Plan Capital Improvement Plan. In some instances, a facility need is ranked ahead of other facility needs that fall within an earlier expected timeframe. This is due to the importance of that need for continued successful operation of the WWTF. Facility needs with equal priority rankings are recommended to be addressed concurrently.

**Table 6.7** below summarizes the recommendations for operation improvements that were identified from the existing condition assessments. These recommendations are intended to improve process efficiency and consistency, address operational issues noted, improve process resiliency, and improve the ease of process operations.

*Table 6.6 – Current Facility Needs and Major Equipment Replacements*

| Process Area | Facility Need | Expected Timeframe (years) | Preliminary Priority Ranking |
|---|---|---|---|
| Administration Building | Perform engineering analysis of existing footings and pile caps to determine repair modifications to remove potential for continuing settlement. Engineering analysis to include subsurface soil investigation. Perform associated foundation and wall repairs per recommendations of the engineering analysis. | 5 | 53 |
| Power Distribution | Replace 'SB-1' and 'SB-2'. | 10 | 46 |
| Septage Receiving | Install weigh scales or flow meter to track septage receiving. | 10 | 50 |
| Influent Pumping Station | Repair cracks in exterior top of wet well wall. | 1 | 12 |
| Influent Pumping Station | Replace influent pumps. | 10 | 32 |
| Influent Pumping Station | Replace influent flow measurement. | 10 | 33 |
| Influent Pumping Station | Replace wet well level measurement equipment. | 5 | 40 |
| Influent Pumping Station | Repair cracks in walls and slabs | 10 | 41 |
| Influent Pumping Station | IPS ventilation system improvements. | 5 | 42 |
| Influent Pumping Station | Evaluate and implement modifications to alleviate FOG build-up in wet well | 10 | 47 |

| | | | |
|---|---|---|---|
| Screening and Grit Removal | Replace screening and grit removal equipment. Recommend relocation upstream of Influent Pumping Station. | 10 | 31 |
| Screening and Grit Removal | Repair cracks in slabs | 10 | 53 |
| Screening and Grit Removal | Repair continuous crack between aeration basin and north wall | 5 | 53 |
| Aeration Basins | Perform engineering analysis of bowing/deflection in aeration basin #2 north wall to develop repair recommendations. | 1 | 1 |
| Aeration Basins | Survey aeration basin #2 north wall to measure and monitor deflection. | 1 | 2 |
| Aeration Basins | Perform engineering analysis of aeration basins to verify structural integrity and develop repair plans. | 1 | 4 |
| Aeration Basins | Repair aeration basin #2 north wall bowing/deflection following recommendations of engineering analysis. | 2 | 5 |
| Aeration Basins | Repair cracks in faces of exterior walls following recommendations of engineering analysis. | 2 | 6 |
| Aeration Basins | Replace air header isolation valves in aeration basin #1 at time of diffuser replacement | 1 | 9 |
| Aeration Basins | Repair cracks in walkway slabs and top of walls. | 10 | 53 |
| Blower Building | Perform subsurface soils investigation to identify repair strategies to correct settling issues. | 1 | 11 |
| Blower Building | Recoat blower discharge piping to protect from corrosion. | 2 | 16 |
| Blower Building | Repair/replace sidewalks, pipe supports, access stair framing, columns, footings, and roof framing (if required) following recommendations of subsurface soils investigation. | 2 | 17 |
| Blower Building | Replace existing blowers and provide variable speed control. | 10 | 24 |
| Blower Building | Replace existing RRVS motor controllers at time of blower replacement. Provide variable speed control for future blowers. | 10 | 24 |
| Secondary Clarifiers | Equipment manufacturer inspect clarifier mechanical and drive mechanisms and provide rehabilitation recommendations. | 5 | 23 |
| Secondary Clarifiers | Rehabilitate/rebuild existing clarifier mechanical and drive mechanisms. | 10 | 34 |
| Secondary Clarifiers | Replace clarifier scum boxes. | 10 | 34 |
| Secondary Clarifiers | Repair cracks in exterior walls. | 10 | 53 |
| Recycle Pumping Station | Replace RAS pump #2 and WAS pumps | 5 | 27 |
| Recycle Pumping Station | RPS heating and ventilation system improvements. | 5 | 43 |
| Recycle Pumping Station | Repair cracks in walls, slabs, and exterior top of walls. | 10 | 53 |

| | | | |
|---|---|---|---|
| Tertiary Filters | Replace Tertiary Filter #2. | 5 | 25 |
| Tertiary Filters | Install clear span structure over tertiary filters. | 5 | 26 |
| Tertiary Filters | Repair cracks in north wall. | 10 | 53 |
| Utility Building | Replace seal water pumping system. | 10 | 48 |
| Disinfection Basin | Replace UV Disinfection System in new channel. | 5 | 13 |
| Disinfection Basin | Install isolation transformer with UVD system replacement. | 5 | 14 |
| Disinfection Basin | Repair cracks in walls and slabs. | 10 | 54 |
| Disinfection Basin | Replace existing fiberglass grating. | 5 | 55 |
| Sludge Thickening | Evaluate cost-benefit analysis of new aerated sludge holding tank vs. new GBT/RDT and conversion of existing thickeners to aerated sludge holding tanks. | 1 | 8 |
| Sludge Thickening | Repair cracks in gravity thickener #1 and install interior/exterior coating system to rehabilitate and protect existing concrete basin. | 2 | 15 |
| Sludge Thickening | Equipment manufacturer inspect gravity thickener mechanical and drive mechanisms and provide rehabilitation recommendations. | 5 | 22 |
| Sludge Thickening | Replace belt filter press feed pumps. | 5 | 28 |
| Sludge Thickening | Relocate isolation valves on thickened sludge suction piping. | 5 | 28 |
| Sludge Thickening | Install aerated sludge holding tank or install new GBT/RDT and convert existing thickeners to aerated thickened sludge holding. | 10 | 35 |
| Sludge Thickening | Rehabilitate/rebuild existing gravity thickener mechanical and drive mechanisms. | 10 | 36 |
| Sludge Thickening | Install interior coating systems in gravity thickener #2 | 5 | 39 |
| Sludge Thickening | Repair cracks in thickening building and install new steel beams to support roof slab (if required). | 10 | 49 |
| Sludge Dewatering | Evaluate pressing schedule and process automation to improve operation, improve dewatered cake consistency, and reduce odor issues. | 1 | 7 |
| Sludge Dewatering | Replace BFP #1 filter belts | 2 | 18 |
| Sludge Dewatering | Replace roller bearings on BFP #1 and #2 | 2 | 18 |
| Sludge Dewatering | Repair damaged CMU lintel beam on BFP room east wall entry door. | 1 | 19 |
| Sludge Dewatering | Replace dewatered cake conveyor belt, chain, rollers and bearings. | 5 | 21 |
| Sludge Dewatering | Replace polymer makedown skids | 5 | 38 |
| Sludge Dewatering | Replace existing BFPs. | 10 | 45 |
| Biosolids Storage | Replace biosolids storage shelter roof. | 5 | 20 |
| Biosolids Storage | Install new protective coatings on structural steel members. | 5 | 37 |

| | | | |
|---|---|---|---|
| Lightning Protection | Install Surge Protective Devices on all power distribution equipment. | 5 | 30 |
| Lightning Protection | Install facility wide grounding/lightning protection system. | 5 | 44 |
| Instrumentation and Control | Identify and correct all erroneous process/equipment data in SCADA HMI application. | 1 | 10 |
| Instrumentation and Control | Extend SCADA HMI application to control plant processes and equipment. | 5 | 29 |
| Instrumentation and Control | Implement intermediate DMZ network between plant control system and external networks. | 5 | 29 |
| Instrumentation and Control | Install SCADA historical database and permanent offline storage for long term data storage and use. | 5 | 29 |
| Instrumentation and Control | Implement SCADA system dashboards and reports to inform operations staff and improve facility operations. | 5 | 29 |
| Site/Civil | Test standing water on north side of aeration basins for indicators of wastewater contamination to determine presence of leaks from adjacent aeration basins. | 1 | 3 |
| Site/Civil | Investigate in-plant manhole #1 for damage to incoming piping due to potential settlement. Repair as necessary. | 1 | 51 |
| Site/Civil | Repair sidewalk settlement on west side of RPS to eliminate trip hazard from valve operating nuts. | 5 | 52 |
| Site/Civil | Regrade access road north of aeration basins to alleviate standing water issues. | 1 | 56 |

*Table 6.7 – Operational Recommendations*

| Process Area | Operational Recommendations |
|---|---|
| Screening and Grit Removal | Relocate upstream ultrasonic level transducer to reduce impacts from turbulence. |
| Aeration Basins | Reduce MLSS concentration to approx. 3,100 mg/L and corresponding SRT. |
| Aeration Basins | Install online DO and $NO_3$ analyzers in aeration basins to improve process monitoring and control. |
| Secondary Clarifiers | Install effluent launder covers to limit/eliminate algae growth. |
| Secondary Clarifiers | Install density current baffles. |
| Recycle Pumping Station | Automate sludge recycle and wasting operations to improve process control and consistency. |
| Recycle Pumping Station | Evaluate and implement improvements to provide adequate mixing or removal of scum from WAS wet well. |
| Tertiary Filters | Perform periodic chemical cleaning of cloth filter media. |

| | |
|---|---|
| Tertiary Filters | Replace cloth media every 5 to 10 years or as needed. |
| Sludge Thickening | Increase dewatering schedule to reduce sludge residence time in thickeners and prevent anaerobic conditions. |
| Sludge Dewatering | Automate sludge dewatering operations to improve operational efficiency. |
| Sludge Dewatering | Automate polymer makedown and feed systems to improve operational consistency. |
| Instrumentation and Control | Maintain stock of PLC spare parts on-site. |
| Instrumentation and Control | Maintain spare ethernet switches on site. |

# APPENDIX A – EXISTING EQUIPMENT DATASHEETS

McKIM&CREED

# Appendix A

| | |
|---|---|
| Client: | **City of Hendersonville** |
| Project: | **Wastewater Treatment Facility Master Plan** |
| Subject: | **Existing Equipment Asset Management Information** |
| Project #: | **06496-0009** |
| Date: | **February 22, 2021** |

| Process Area | Equipment Name | Equipment Type | Manufacturer | Model # | Serial # | Design Head | Design Flow | HP | Voltage | RPM | Nom. Eff. % | Drive Type | Installation/ Replacement Year |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Power Distribution | 1500 kW Diesel Emergency Generator | Diesel Emergency Standby Generator | Cummins | DQGAF | | | | | 480 | | | | 2019 |
| Power Distribution | Main Switchboard SWB-1 and NEMA 3R Enclosure | Rear-Connected/Front Accessible Electrical Switchboard | Schneider Electric | QED-6 Switchboard | | | | | 480 | | | | 2019 |
| Influent Pumping Station | 8" Influent Pump #1 | Vertical Centrifugal Dry Pit Pump | Chicago Pump | 8815-4A | 7517005372 | 60 | 2800 | 75 | 460 | 1160 | 80 | VFD | 2001 |
| Influent Pumping Station | 8" Influent Pump #1 Motor | Vertical High Thrust Motor | US Motors | 405-VP | | | | 75 | 460 | 1190 | 94.5 | VFD | 2001 |
| Influent Pumping Station | 8" Influent Pump #1 VFD | PowerFlex 750-Series AC Drive | Allen-Bradley | PowerFlex 753 | 20F11ND096AA0NNNNN | | | 75 | 480 | | | VFD | 2019 |
| Influent Pumping Station | 8" Influent Pump #2 | Vertical Centrifugal Dry Pit Pump | Chicago Pump | 8815-4A | 7517002925 | 60 | 2800 | 75 | 460 | 1160 | 80 | VFD | 2001 |
| Influent Pumping Station | 8" Influent Pump #2 Motor | Vertical High Thrust Motor | US Motors | 405-VP | | | | 75 | 460 | 1190 | 94.5 | VFD | 2001 |
| Influent Pumping Station | 8" Influent Pump #2 VFD | PowerFlex 750-Series AC Drive | Allen-Bradley | PowerFlex 753 | 20F11ND096AA0NNNNN | | | 75 | 480 | | | VFD | 2019 |
| Influent Pumping Station | 10" Influent Pump #3 | Vertical Centrifugal Dry Pit Pump | Chicago Pump | 101022-5 | 9806747 | 77 | 4500 | 125 | 460 | 880 | 77 | VFD | 2001 |
| Influent Pumping Station | 10" Influent Pump #3 Motor | Vertical High Thrust Motor | US Motors | 447-VP | | | | 125 | 460 | 885 | 94.5 | VFD | 2001 |
| Influent Pumping Station | 10" Influent Pump #3 VFD | PowerFlex 750-Series AC Drive | Allen-Bradley | PowerFlex 753 | 20F1AND156AN0NNNNN | | | 125 | 480 | | | VFD | 2019 |
| Influent Pumping Station | 10" Influent Pump #4 | Vertical Centrifugal Dry Pit Pump | Chicago Pump | 101022-5 | 9806747 | 77 | 4500 | 125 | 460 | 880 | 77 | VFD | 2001 |
| Influent Pumping Station | 10" Influent Pump #4 Motor | Vertical High Thrust Motor | US Motors | 447-VP | | | | 125 | 460 | 885 | 94.5 | VFD | 2001 |
| Influent Pumping Station | 10" Influent Pump #4 VFD | PowerFlex 750-Series AC Drive | Allen-Bradley | PowerFlex 753 | 20F1AND156AN0NNNNN | | | 125 | 480 | | | VFD | 2019 |
| Screening and Grit Collection | Mechanical bar screen #1 | Link-Belt Cog Rake Mechanical bar screen | USFilter | GA115 | | | 3333 | | | | | | 2001 |
| Screening and Grit Collection | Mechanical bar screen #1 Drive Motor | Horizontal Motor w/ Brake | US Motors | 145-TC | | | | 1.5 | 460 | 1800 | 85.5 | Constant speed | 2001 |
| Screening and Grit Collection | Mechanical bar screen #2 | Link-Belt Cog Rake Mechanical bar screen | USFilter | GA115 | | | 3333 | | | | | | 2001 |
| Screening and Grit Collection | Mechanical bar screen #2 Drive Motor | Horizontal Motor w/ Brake | US Motors | 145-TC | | | | 1.5 | 460 | 1800 | 85.5 | Constant speed | 2001 |
| Screening and Grit Collection | Shaftless screw conveyor - bar screens | Shaftless Screw Conveyor/Compactor | JDV Equipment Corp. | U260 SP210 A/SS | | | | | | 23 | | | 2001 |
| Screening and Grit Collection | Shaftless screw conveyor motor - bar screens | Shaftless Screw Conveyor/Compactor Motor | Baldor | VM7042T | SAH429799 | | | 3 | 460 | 1725 | 82.5 | Constant speed | 2001 |
| Screening and Grit Collection | Shaftless screw conveyor - grit collector | Shaftless Screw Grit Conveyor | JDV Equipment Corp. | U260 C/SS | | | | | | 4.9 | | | 2001 |
| Screening and Grit Collection | Shaftless screw conveyor motor - grit collector | Shaftless Screw Grit Conveyor Motor | Baldor | VM7034 | B-13952148 | | | 1.5 | 460 | 1725 | 78.5 | Constant speed | 2001 |
| Screening and Grit Collection | Grit collector #1 | Chain and Bucket Grit Collector | USFilter Link-Belt | | | | | | | | | | 2001 |
| Screening and Grit Collection | Grit Collector #1 Drive Motor | Chain and Bucket Grit Collector Drive Motor | US Motors | 143-TC | Y1P2CC-P | | | 1 | 460 | 1800 | 85.9 | constant speed | |
| Screening and Grit Collection | Grit collector #2 | Chain and Bucket Grit Collector | USFilter Link-Belt | | | | | | | | | | 2001 |
| Screening and Grit Collection | Grit Collector #1 Drive Motor | Chain and Bucket Grit Collector Drive Motor | US Motors | | Y1P2CC-P | | | 1 | 460 | 1800 | 85.9 | constant speed | |
| Aeration basins | Blower #1 | Multistage centrifugal blower | Hoffman | 75107A1 | M113000 | 9.0 PSIG | 4400 SCFM | 250 | 460 | 3545 | 72 | Constant speed | 2001 |
| Aeration basins | Blower #1 Motor | TEFC Horizontal Motor | Baldor | 449TS | N120/0135-01012907 | | | 250 | 460 | 3545 | 95.4 | Constant speed | 2001 |
| Aeration basins | Blower #2 | Multistage centrifugal blower | Hoffman | 75107A1 | M112990 | 9.0 PSIG | 4400 SCFM | 250 | 460 | 3545 | 72 | Constant speed | 2001 |
| Aeration basins | Blower #2 Motor | TEFC Horizontal Motor | Baldor | 449TS | PO101-00041426 | | | 250 | 460 | 3545 | 95.4 | Constant speed | 2001 |
| Aeration basins | Blower #3 | Multistage centrifugal blower | Hoffman | 75107A1 | M112980 | 9.0 PSIG | 4400 SCFM | 250 | 460 | 3545 | 72 | Constant speed | 2001 |
| Aeration basins | Blower #3 Motor | TEFC Horizontal Motor | Baldor | 449TS | PO101-00031515 | | | 250 | 460 | 3545 | 95.4 | Constant speed | 2001 |
| Secondary Clarifiers | Secondary Clarifier #1 | 90' - 0" Envirex Tow-Bro Clarifier | USFilter Envirex | | | | | | | | | | 2001 |
| Secondary Clarifiers | Secondary Clarifier #1 Drive Unit | H-Drive Circular Clarifier Drive | Evoqua Envirex | H40A-LT | | | | | | | | | 2017 |
| Secondary Clarifiers | Secondary Clarifier #1 Drive Motor | Helical Gear Motor | SEW-Eurodrive | | 87.7399734201.0001.16 | | | 0.5 | 460 | 1700 | 72 | Constant speed | 2017 |
| Secondary Clarifiers | Secondary Clarifier #2 | 90' - 0" Envirex Tow-Bro Clarifier | USFilter Envirex | | | | | | | | | | 2001 |
| Secondary Clarifiers | Secondary Clarifier #2 Drive Unit | H-Drive Circular Clarifier Drive | Evoqua Envirex | H40A-LT | | | | | | | | | 2017 |
| Secondary Clarifiers | Secondary Clarifier #2 Drive Motor | Helical Gear Motor | SEW-Eurodrive | | 87.7399734201.0001.16 | | | 0.5 | 460 | 1700 | 72 | Constant speed | 2017 |
| Recycle Pumping Station | 10" RAS Pump #1 | Vertical Centrifugal Non-Clog Dry Pit Pump | Grundfos | | | | | 50 | 460 | | | VFD | 2020 |
| Recycle Pumping Station | 10" RAS Pump #1 Motor | Vertical High Thrust Motor | US Motors | 404-VP | | | | 50 | 460 | 900 | 94.1 | VFD | 2001 |
| Recycle Pumping Station | 10" RAS Pump #1 VFD | PowerFlex 750-Series AC Drive | Allen-Bradley | PowerFlex 753 | 20F11ND065AA0NNNNN | | | 50 | 480 | | | VFD | 2019 |
| Recycle Pumping Station | 10" RAS Pump #2 | Vertical Centrifugal Non-Clog Dry Pit Pump | Chicago Pump | LS8-4A | | 18 | 2500 | 50 | 460 | 870 | 82 | VFD | 2001 |
| Recycle Pumping Station | 10" RAS Pump #2 Motor | Vertical High Thrust Motor | US Motors | 404-VP | | | | 50 | 460 | 900 | 94.1 | VFD | 2001 |
| Recycle Pumping Station | 10" RAS Pump #2 VFD | PowerFlex 750-Series AC Drive | Allen-Bradley | PowerFlex 753 | 20F11ND065AA0NNNNN | | | 50 | 480 | | | VFD | 2019 |
| Recycle Pumping Station | WAS Pump #1 | Vertical Centrifugal Non-Clog Dry Pit Pump | Goulds Pumps | HSD | 429A/B762 | 400 | 78 | 60 | 460 | | | VFD | 2001 |
| Recycle Pumping Station | WAS Pump #1 Motor | Vertical Normal Thrust Motor | US Motors | 405VPZ (Frame Size) | 429762MOTR | | | 60 | 460 | 900 | 93 | VFD | 2001 |
| Recycle Pumping Station | WAS Pump #1 VFD | PowerFlex 750-Series AC Drive | Allen-Bradley | PowerFlex 753 | 20F11ND077AA0NNNNN | | | 60 | 480 | | | VFD | 2019 |
| Recycle Pumping Station | WAS Pump #2 | Vertical Centrifugal Non-Clog Dry Pit Pump | Goulds Pumps | HSD | 429A/B762 | 400 | 78 | 60 | 460 | | | VFD | 2001 |
| Recycle Pumping Station | WAS Pump #2 Motor | Vertical Normal Thrust Motor | US Motors | 405VPZ (Frame Size) | 429762MOTR | | | 60 | 460 | 900 | 93 | VFD | 2001 |
| Recycle Pumping Station | WAS Pump #2 VFD | PowerFlex 750-Series AC Drive | Allen-Bradley | PowerFlex 753 | 20F11ND077AA0NNNNN | | | 60 | 480 | | | VFD | 2019 |
| Tertiary Filters | Filter #1 weir gate #1 | Series 40 Fabricated SS Weir Gate | Fontaine-Aquanox | 403-Y4X-84x28-B-CW-3 | | | | | 120 | | | | 2020 |
| Tertiary Filters | Filter #1 weir gate #2 | Series 40 Fabricated SS Weir Gate | Fontaine-Aquanox | 403-Y4X-84x28-B-CW-3 | | | | | 120 | | | | 2020 |
| Tertiary Filters | Filter #2 weir gate #1 | Series 40 Fabricated SS Weir Gate | Fontaine-Aquanox | 403-Y4X-168x28-B-CW-3 | | | | | 120 | | | | 2020 |
| Tertiary Filters | AquaDiamond tertiary filter | Tertiary Cloth Media Filter | Aqua-Aerobic Systems Inc. | ADIFC1650 | 911322460800-1 | | | | 120 | | | | 2020 |

| | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Tertiary Filters | Filter #1 electromagnetic flow meter | Electromagnetic flow meter | Krohne | Enviromag Series 2000F | | | | | 115 | | | | 2020 |
| Tertiary Filters | Filter #1 backwash pump | Centrifugal Backwash Pump | Gorman Rupp | T4A60S | | | | 20 | 460 | | | VFD | 2020 |
| Tertiary Filters | Traveling bridge sand filter | Traveling Hood Sand Filter | EIMCO | | | | | | | | | | 2001 |
| Disinfection Basin | UV Disinfection System | UV Disinfection System | Trojan Technologies, Inc. | UV400 | 410027 | | | | | | | | 2000 |
| Disinfection Basin | Plant Water Pump #1 | Vertical Turbine Pump | Floway | 8L-9 | | | | 20 | 460 | | | VFD | 2018 |
| Disinfection Basin | Plant Water Pump #1 Motor | Vertical High Thrust Motor | US Motors | 256TPH | | | | 20 | 460 | 1800 | 93 | VFD | 2018 |
| Disinfection Basin | Plant Water Pump #2 | Vertical Turbine Pump | Floway | 8L-9 | | | | 20 | 460 | | | VFD | 2018 |
| Disinfection Basin | Plant Water Pump #2 Motor | Vertical High Thrust Motor | US Motors | 256TPH | | | | 20 | 460 | 1800 | 93 | VFD | 2018 |
| Utility Building | Plant Water Pump #1 VFD | PowerFlex 750-Series AC Drive | Allen-Bradley | PowerFlex 752 | 20F11ND027AA0NNNNN | | | 20 | 480 | | | VFD | 2017 |
| Utility Building | Plant Water Pump #2 VFD | PowerFlex 750-Series AC Drive | Allen-Bradley | PowerFlex 753 | 20F11ND027AA0NNNNN | | | 20 | 480 | | | VFD | 2017 |
| Thickener Building | Gravity Thickener #1 | Gravity Thickener | USFilter | F-Drive | | | | | | | | | 2000 |
| Thickener Building | Gravity Thickener #1 Drive Unit | Gravity Thickener Drive | Nord Gear Corporation | SK630-90LH/4 | 200004261065 | | | | | | | | 2000 |
| Thickener Building | Gravity Thickener #1 Drive Motor | Gravity Thickener Drive Motor | Nord Gear Corporation | 90S/L | | | | 2 | 460 | 1740 | | | 2000 |
| Thickener Building | Gravity Thickener #2 | Gravity Thickener | USFilter | F-Drive | | | | | | | | | 2000 |
| Thickener Building | Gravity Thickener #2 Drive Unit | Gravity Thickener Drive | Nord Gear Corporation | SK630-90LH/4 | 200004261065 | | | | | | | | 2000 |
| Thickener Building | Gravity Thickener #2 Drive Motor | Gravity Thickener Drive Motor | Nord Gear Corporation | 90S/L | | | | 2 | 460 | 1740 | | | 2000 |
| Thickener Building | Belt Filter Press Feed Pump #1 | Double Disc Pump | Penn Valley Pump Co. | | 99K 74-75-76 | | 95 | | | 519 | | | 2001 |
| Thickener Building | Belt Filter Press Feed Pump #1 Motor | Inverter Duty Motor | Baldor | B213T | EM3770T | | | 7.5 | 460 | 1750 | 91 | constant speed | 2001 |
| Thickener Building | Belt Filter Press Feed Pump #1 VFD | PowerFlex 750-Series AC Drive | Allen-Bradley | PowerFlex 753 | 20F11ND011AA0NNNNN | | | 7.5 | 480 | | | VFD | 2017 |
| Thickener Building | Belt Filter Press Feed Pump #2 | Double Disc Pump | Penn Valley Pump Co. | | 99K 74-75-76 | | 95 | | | 519 | | | 2001 |
| Thickener Building | Belt Filter Press Feed Pump #2 Motor | Inverter Duty Motor | Baldor | B213T | EM3770T | | | 7.5 | 460 | 1750 | 91 | constant speed | 2001 |
| Thickener Building | Belt Filter Press Feed Pump #2 VFD | PowerFlex 750-Series AC Drive | Allen-Bradley | PowerFlex 753 | 20F11ND011AA0NNNNN | | | 7.5 | 480 | | | VFD | 2017 |
| Thickener Building | Belt Filter Press Feed Pump #3 | Double Disc Pump | Penn Valley Pump Co. | | 99K 74-75-76 | | 95 | | | 519 | | | 2001 |
| Thickener Building | Belt Filter Press Feed Pump #3 Motor | Inverter Duty Motor | Baldor | B213T | EM3770T | | | 7.5 | 460 | 1750 | 91 | constant speed | 2001 |
| Thickener Building | Belt Filter Press Feed Pump #3 VFD | PowerFlex 750-Series AC Drive | Allen-Bradley | PowerFlex 753 | 20F11ND011AA0NNNNN | | | 7.5 | 480 | | | VFD | 2017 |
| Dewatering Building | Wash Water Pump #1 | Belt Press Wash Pump | Goulds Pumps | 3655 | 764E945-1-2 | 330 | 90 | 20 | | 3500 | 48 | | 2000 |
| Dewatering Building | Wash Water Pump Motor #1 | Belt Press Wash Pump Motor | Goulds Pumps | 256 TCZ | | | | 20 | | 3500 | 48 | VFD | 2000 |
| Dewatering Building | Wash Water Pump #2 | Belt Press Wash Pump | Goulds Pumps | 3655 | 764E945-1-2 | 330 | 90 | 20 | | 3500 | 48 | | 2000 |
| Dewatering Building | Wash Water Pump Motor #2 | Belt Press Wash Pump Motor | Goulds Pumps | 256 TCZ | | | | 20 | | 3500 | 48 | VFD | 2000 |
| Dewatering Building | Air Compressor | Type 30 Air Compressor | Ingersoll-Rand | 2340 | | | | | | | | | 2012 |
| Dewatering Building | Belt Filter Press #1 | Belt Filter Press | SernaTech | BFP 2000 WR 15 | 60405948000001 | | | | | | | VFD | 2000 |
| Dewatering Building | Belt Filter Press #2 | Belt Filter Press | SernaTech | BFP 2000 WR 15 | 60405948000001 | | | | | | | VFD | 2000 |
| Dewatering Building | Belt Filter Press VFD (8 HP) | PowerFlex 750-Series AC Drive | Allen-Bradley | PowerFlex 753 | 20F11ND011AA0NNNNN | | | 7.5 | 480 | | | VFD | 2017 |
| Dewatering Building | Belt Filter Press VFD (2 HP) | PowerFlex 750-Series AC Drive | Allen-Bradley | PowerFlex 753 | 20F11ND3P4AA0NNNNN | | | 2 | 480 | | | VFD | 2017 |
| Dewatering Building | Sludge Conveyor | Sludge Conveyor | Serpentix | H | | | | 5 | 460 | 1750 | | Constant speed | 2001 |
| Dewatering Building | Polymer Blending System | Polymer Blending System | USFilter | M24000-D10AA | | | | | | | | | 2001 |
| Dewatering Building | Polymer Blending System Feed Pump & Motor | Polymer Blending System Feed Pump & Motor | USFilter | C771-20PBA | | 185 | 10 | 1 | 120 | | | VFD | 2001 |

# APPENDIX B:

## AS-BUILT SURVEY AND EXISTING FACILITY HYDRAULIC PROFILE


| DATE | REVISION | INITIAL |
|---|---|---|
| | | |

McKIM&CREED
8020 Tower Point Drive
Charlotte, North Carolina 28227
Phone: (704)841-2588, Fax: (704)841-2567
F-1222

Internet Site: http://www.mckimcreed.com

DATE:

AS-BUILT SURVEY
FOR
**HENDERSONVILLE WWTP**
FOR
**CITY OF HENDERSONVILLE**

DATE: MARCH 2020 SCALE: 1" = 40'
CITY OF HENDERSONVILLE HENDERSON COUNTY NORTH CAROLINA

PROJECT # : 06496-0009
PROJ. SVYR : WAE
DRAWN BY : NKD
FIELD BK. : C-1167
COMP. FILE : 06496-0009_HENDERSONVILLE
SHEET # : 1 OF 1

DWG. # : 06496-0009_HENDERSONVILLE


| | | | |
|---|---|---|---|
| A | REVISED HYDRAULIC PROFILE – FOR REFERENCE ONLY | MAY 2020 | |
| REV.NO. | DESCRIPTIONS | DATE | |

REVISIONS

SEAL

SEAL

NORTH CAROLINA PROFESSIONAL ENGINEER SEAL 046291 ZACHARY M. TRAMMEL 5/7/20

McKIM&CREED
8020 Tower Point Drive
Charlotte, North Carolina 28227
Phone: (704)841-2588, Fax: (704)841-2567
F-1222
www.mckimcreed.com

THE CITY OF HENDERSONVILLE N.C. 1847

**CITY OF HENDERSONVILLE, NC**
*"The City of Four Seasons"*
City of Hendersonville Engineering Department
305 Williams Street
Hendersonville, NC 28792
(828) 697-3000 (office)
(828) 697-3066 (fax)
www.cityofhendersonville.org

CITY OF HENDERSONVILLE
WASTEWATER TREATMENT FACILITY
MASTER PLAN

EXISTING FACILITY HYDRAULIC PROFILE

| | |
|---|---|
| DATE: | MAY 2020 |
| MCE PROJ. # | 06496-0009 |
| DRAWN | AWB |
| DESIGNED | JJM |
| CHECKED | JJM |
| PROJ. MGR. | ZMT |

SCALE
HORIZONTAL: 1"=40'
VERTICAL: NA

M&C FILE NUMBER
DRAWING NUMBER
1
1 OF 1

STATUS: REVISED HYDRAULIC PROFILE FOR REFERENCE ONLY
REVISION A

# APPENDIX C:

## TECHNICAL MEMORANDUM No. 2 – TREATMENT PROCESS EVALUATIONS

# CITY OF HENDERSONVILLE
## WASTEWATER TREATMENT FACILITY MASTER PLAN

## Technical Memorandum No.2 – Treatment Process Evaluations

Date: June 2022

**Prepared for:**


City of Hendersonville
305 Williams Street
Hendersonville, NC 28792


**Prepared by:**


McKim & Creed, Inc.
8020 Tower Point Dr.
Charlotte, NC 28227
Firm License No. F-1222


**McKim & Creed Project 06496-0009**

# CONTENTS

# TABLES

# FIGURES

# APPENDICES

# LIST OF ACRONYMS

| Abbreviation | Definition |
|---|---|
| AADF | Annual Average Daily Flow |
| ATC | Authorization to Construct |
| BEP | Best Efficiency Point |
| BFP | Belt Filter Press |
| BNR | Biological Nutrient Removal |
| $BOD_5$ | Biochemical Oxygen Demand, Five Day |
| CFM | Cubic Feet per Minute |
| CIP | Capital Improvement Plan |
| C.I.P. | Cast-In-Place |
| CLOMR | Conditional Letter of Map Revision |
| DI | Ductile Iron |
| DIP | Ductile Iron Pipe |
| DMR | Discharge Monitoring Report |
| DO | Dissolved Oxygen |
| EDI | Environmental Dynamics International |
| EQ | Equalization |
| FC | Fecal Coliform |
| FEMA | Federal Emergency Management Agency |
| FIRM | Flood Insurance Rate Map |
| FM | Force Main |
| FOG | Fats, Oils, and Grease |
| GBT | Gravity Belt Thickener |
| GPM | Gallons per Minute |
| HLR | Hydraulic Loading Rate |
| hp | Horsepower |
| HRT | Hydraulic Retention Time |
| IFAS | Integrated Fixed Film Activated Sludge |
| INF | Influent |
| IPS | Influent Pumping Station |
| IWA | International Water Association |
| L | Liters |
| LOMR | Letter of Map Revision |
| MBR | Membrane Bioreactor |
| MG | Million Gallons |
| mg | milligram |
| MGD | Million Gallons per Day |
| MLE | Modified Ludzack-Ettinger |
| MLSS | Mixed Liquor Suspended Solids |
| MLVSS | Mixed Liquor Volatile Suspended Solids |
| MMF | Maximum Month Flow |
| MOP | Manual of Practice |
| MOV | Motor Operated Valve |
| MSL | Mean Sea Level |
| NCAC | North Carolina Administrative Code |
| NCDEQ | North Carolina Department of Environmental Quality |

| | |
|---|---|
| NCDOT | North Carolina Department of Transportation |
| $NH_3$-N | Ammonia as nitrogen |
| $NO_3$-N | Nitrate as nitrogen |
| $NO_2$-N | Nitrite as nitrogen |
| NPDES | National Pollutant Discharge Elimination System |
| NPV | Net Present Value |
| NPW | Non-Potable Water |
| NRCY | Nitrified Recycle |
| OPCC | Opinion of Probable Construction Cost |
| PF | Peaking Factor |
| PHF | Peak Hourly Flow |
| PS | Pump Station |
| RAS | Return Activated Sludge |
| RH | Relative Humidity |
| RPM | Revolutions Per Minute |
| RPS | Recycle Pumping Station |
| SBR | Sequencing Batch Reactor |
| SCADA | Supervisory Control and Data Acquisition |
| SCFM | Standard Cubic Feet per Minute |
| SES | Sand Equivalent Size |
| SLR | Solids Loading Rate |
| SOR | Surface Overflow Rate |
| SPA | State Point Analysis |
| SRT | Solids Retention Time |
| SS | Stainless Steel |
| SSAIA | Sanitary Sewer Asset Inventory and Assessment |
| SSO | Sanitary Sewer Overflow |
| SWD | Side Water Depth |
| TDH | Total Dynamic Head |
| TKN | Total Kjeldahl Nitrogen |
| TN | Total Nitrogen |
| TP | Total Phosphorus |
| TS | Total Solids |
| TSS | Total Suspended Solids |
| TWAS | Thickened Waste Activated Sludge |
| US | United States |
| USA | United State of America |
| UV | Ultra-Violet |
| UVD | Ultra-Violet Disinfection |
| VFD | Variable Frequency Drive |
| VSS | Volatile Suspended Solids |
| WAS | Waste Activated Sludge |
| WEF | Water Environment Federation |
| WWTF | Wastewater Treatment Facility |

# EXECUTIVE SUMMARY

## ES.1 Purpose and Background

The City of Hendersonville's WWTF is permitted under North Carolina Department of Environmental Quality (NCDEQ) NPDES Permit No. NC0025534 to treat 4.8 million gallons per day (MGD) maximum month flow (MMF) with an extended aeration activated sludge domestic wastewater treatment plant. The current NPDES permit also includes provisions for a future permitted capacity of 6.0 MGD at MMF upon issuance of an Authorization to Construct (ATC) for expansion to the existing WWTF. Construction of the existing 4.8 MGD WWTF was completed in 2001, and since that time a comprehensive study to evaluate the ability of the WWTF to meet the City's future wastewater treatment needs has not been completed. The City of Hendersonville retained the services of McKim & Creed to evaluate, identify, and schedule recommended improvements to ensure that the WWTF continues to meet the City's wastewater treatment needs and that all current and future permit limits continue to be met.

The WWTF Master Plan consists of three separate technical memoranda that form the basis of the recommendations for the overall facility and the content of the Capital Improvement Plan (CIP) for the WWTF. The scope of this master plan includes review of previous engineering studies and influent flow projections, existing condition assessments of the WWTF, current capacity evaluations of each treatment process, recommendations for improvements and expansion to the WWTF, and a preliminary engineering evaluation to provide flow equalization facilities at the WWTF.

This technical memorandum provides improvement and expansion recommendations for each treatment process and the overall facility that will restore and maintain reliability, improve treatment and hydraulic limitations, improve efficiencies, resolve operational issues, continue to meet existing permit conditions, and meet future treatment needs and permit conditions. The following tasks were completed as part of this effort:

- Current capacity analyses for each treatment process
- A peak hour flow evaluation of the overall facility hydraulic profile beyond 12 MGD
- BioWin wastewater process modeling to evaluate current capacity and alternatives for expansion
- Evaluation of treatment process and technology alternatives for improvements and expansion
- Recommendations for improvements including conceptual cost opinions

## ES.2 Current Capacity Analyses

The overall treatment facility was reviewed by treatment process to evaluate the current capacity. The peak hour hydraulic capacity of the treatment facility was also reviewed beyond the original peak hour design flow rate of 12.0 MGD to identify hydraulic bottlenecks that may occur as peak wet weather flows are expected to increase and as the facility is expanded to meet future treatment needs. The capacity limitations of the existing treatment facility are summarized in **Table ES** below. Detailed analyses of the current capacity of each treatment process area and hydraulic limitations are described in the respective sections for each treatment process in this technical memorandum.

*Table ES.0.1 – Summary of Process Limitations*

| Process | Limiting Condition | Capacity | Notes |
|---|---|---|---|
| Influent Pumping Station | PHF | 12.4 MGD | Firm capacity of the IPS with one 125 hp pump and two 75 hp pumps operating at full speed. |
| Influent Force Mains | PHF | 15.0 MGD | Maximum capacity with both force mains in operation with a velocity of 8.3 fps in each force main. |
| Screening | PHF | 12.0 MGD | Capacity evaluated assuming one screen out of service. Velocity through the screen is 8.11 ft/sec which exceeds the maximum velocity of 4 ft/sec through the openings recommended for PHF to prevent debris pass-through. |
| Grit Removal | PHF | 17.7 MGD | Maximum peak hour flow through both aerated grit chambers to maintain a minimum hydraulic detention time of 3 minutes. |
| Activated Sludge | Maximum month winter loading to Activated Sludge | 15,300 lbs. BOD/day and 3,100 lbs. TKN/day | Limitation assumes two trains in operation. Limiting capacity exceeds 4,500 mg/L MLSS concentration limit, results in SRT below winter design SRT of 13.8 days and causes the secondary clarifiers to be critically loaded. Limiting condition corresponds to a maximum month influent flow rate of 8.39 MGD based on the assumed influent characteristics. |
| RAS Pumps | MMF | 6.2 MGD | Firm capacity with one pump operating at full speed. Typical recommended RAS pumping firm capacity is 110% of the design MMF. Existing pumps provide 103% of the expected MMF for a permitted capacity of 6.0 MGD. Existing firm capacity is sufficient for 6.0 MGD permitted capacity. Expansion will be required as MMF approaches 6.0 MGD. |

| Process | Limiting Condition | Capacity | Notes |
|---|---|---|---|
| WAS Pumps | Maximum day loading conditions | 38,000 lbs. TS/day | Firm capacity with one pump operating at design condition point 1 (400 gpm). Assumes continuous wasting operations and a WAS concentration of 0.8% TS. Maximum capacity with an intermittent wasting schedule of 8 hours per day is 12,800 lbs. TS/day. WAS pumps have sufficient capacity for 6.0 MGD permitted capacity with increased operating schedule. |
| Tertiary Filters | PHF | 2.40 MGD | Firm capacity at peak hour flow is limited by the capacity of existing tertiary filter No. 2. Replacement of tertiary filter No. 2 to match AquaDiamond tertiary filter No. 1 will increase firm capacity to 15.0 MGD. |
| UV Disinfection | PHF | 12.0 MGD | Limited by existing equipment hydraulic and treatment capacity at peak hour flow. Replace UV disinfection equipment in a new channel. |
| Cascade Reaeration | AADF | 4.23 MGD | Cascade reaeration steps limited by hydraulic loading rate per foot of step width. Expansion or replacement of cascade reaeration steps expected to be required beyond AADF of 4.23 MGD to provide maximum hydraulic loading rate of 500,000 gpd/ft of width. Monitor effluent DO to identify need for expansion/replacement. |
| Outfall | PHF | 15.0 MGD | Limiting hydraulic condition is based on the head loss that would submerge the cascade reaeration effluent weir under 100-year flood conditions. The capacity of the existing 36-inch outfall based on the normal elevation of Mud Creek exceeds the future non-equalized peak flow. |
| Gravity Thickeners | Maximum month solids loading | 15,700 lbs. TS/day | Current capacity is evaluated assuming one unit is out of service with a maximum SLR of 8 lbs/day per $ft^2$ of surface area. Both gravity thickeners may be operated to provide thickening capacity for maximum day loading conditions. |
| Belt Filter Press Feed Pumps | Maximum month solids loading | 25,100 lbs. TS/day | Firm capacity with one BFP feed pump out of service and a maximum BFP operating schedule of 40 hours per week. |
| Belt Filter Presses | Maximum month solids loading | 17,100 lbs. TS/day | Assumes both BFPs operating, based on 40 hours of operation per week, with a maximum solids loading rate of 750 lbs/hr per meter of belt width. Each BFP is two meters wide, with a maximum SLR of 1,500 lbs/hr per BFP. |
| Dewatered Sludge Conveyor | Dewatered cake production rate at maximum month solids loading | 5.0 wet tons/hr | With one BFP in operation, dewatered cake production is 4.45 wet tons/hr. With two BFPs in operation dewatered cake production is 8.9 wet tons/hr. |

## ES.3 Recommended Process Improvements

Following the current capacity analyses, improvements to each treatment process were evaluated to rehabilitate, expand, or replace the existing processes where needed to meet current and future treatment needs.

### ES.3.1 – Preliminary Treatment

Improvements to the preliminary treatment systems are recommended to:

- Expand influent pumping capacity to provide firm capacity meeting or exceeding the current and future peak influent flow rates
- Expand the capacity of influent screening to meet or exceed firm capacity requirements at current and future peak influent flow rates
- Relocate screening facilities ahead of the influent pumping station to better protect the influent pumps and downstream equipment
- Expand the capacity of grit removal to meet or exceed capacity requirements at current and future peak influent flow rates

Recommended improvements to the influent pumping station include expansion of the existing influent pumping station and upsizing of the existing influent pumps to provide firm pumping capacity to meet the current and future peak influent flow rates. Minor structural repairs and rehabilitation to the existing influent pump station structure are recommended to prolong the service life of the existing influent pumping station. Heating and ventilation improvements are also recommended within the existing influent pumping station building and dry well. Screening of influent wastewater is recommended to be relocated ahead of the expanded influent pumping station to protect the influent pumps from ragging and excessive wear. The screening facility is recommended to consist of chain-driven multi-rake bar screens with a bar spacing of ¼-inch (6 mm) per the Ten State Standards for WWTF's without primary treatment. Influent flow measurement at the influent pumping station is recommended to be replaced by new electromagnetic flow meters on the influent force mains leaving the expanded pump station, or alternatively, using multiple Parshall flumes located immediately downstream of the new screening equipment in each screening channel. A new mechanically induced vortex grit removal system is recommended to be constructed at the old plant site immediately upstream of the proposed inline flow equalization basin per the recommendations of Technical Memorandum No. 3.

The total estimated capital cost of recommended preliminary treatment improvements is $17,636,000, excluding the costs of future flow equalization facilities which are described in Technical Memorandum No.

3. The estimated capital costs of these improvements, and all other capital costs included herein, are presented in September 2021 dollars. These estimated capital costs are recommended to be revisited and updated regularly to capture changes in market conditions prior to project conception to allow for budgets to be updated appropriately.

### ES.3.2 – Secondary Treatment

BioWin wastewater process modeling was performed to evaluate the current capacity of the existing secondary treatment processes. The results of the current capacity analysis concluded that future expansion of the existing secondary treatment processes would be required to meet projected 2040 loading conditions. Based on this, preliminary alternatives screening was completed to identify feasible facility expansion alternatives to meet the 2040 loading conditions. The results of the preliminary alternatives screening recommended that the following processes be evaluated to review the feasibility of their use for future expansion to the City's WWTF:

- Modified Ludzack-Ettinger (MLE) process
- BioMag® ballasted activated sludge in an MLE configuration
- Integrated fixed-film activated sludge (IFAS)

The results of the secondary process evaluation concluded that maintenance of the existing extended aeration process and future modifications to convert it to a Modified Ludzack-Ettinger (MLE) process is preferred over other process modifications evaluated.

Recommendations for improvements to the secondary treatment processes based on the results of the process evaluation include:

- Implementation of new anoxic zone mixing consisting of a new compressed gas mixing system
- Addition of nitrified internal mixed liquor recycle pumps and piping to each aeration basin to recycle nitrified mixed liquor from the end of the aeration basin back to the head of the basin for denitrification

The total estimated capital costs for the improvements to the existing aeration basins to convert to an MLE process is $1,688,000.

Additional intermediate improvements to the existing extended aeration process prior to the conversion to an MLE process are also recommended to include:

- Replacement of the existing blowers with VFD driven turbo blowers
- Rehabilitation of the existing blower building and retrofits to the structure to provide an enclosed blower room for protection of turbo blower intakes

- Replacement of the existing RAS pump No. 2 and the existing WAS pumps in like-kind, minor structural repairs to the existing Recycle Pumping Station, and improvements to the existing Recycle Pumping Station heating and ventilation systems

The total estimated cost of intermediate improvements to the existing extended aeration process is $3,262,000. This does not include costs for repairs to the existing aeration basin No. 2 that were recommended in Technical Memorandum No. 1.

As noted above, expansion to the existing secondary treatment process is expected to be required to meet the 2040 loading conditions. The future expansion of the secondary treatment process is recommended to include the following:

- Primary effluent splitter box
- A new 2.4 MG aeration basin No. 3, to match existing aeration basins No. 1 and No. 2, including a dedicated anoxic zone with compressed gas mixing and a NRCY pump and pipeline
- A new blower building No. 2 to house new turbo blowers, a compressed gas mixing system for the anoxic zone, NRCY pump VFDs, and all associated electrical and control equipment
- A new MLSS splitter box to direct aeration basin No. 3 effluent to a new secondary clarifier No. 3, and provide long-term future expansion capability to include a fourth aeration basin and secondary clarifier
- A new 90-ft diameter secondary clarifier No. 3, to match existing secondary clarifiers No. 1 and No. 2
- A new recycle pumping station No. 2 to include RAS and WAS pumping serving aeration basin No. 3 and secondary clarifier No. 3

The total estimated capital cost of future expansion to a third secondary treatment train is $25,640,000.

### ES.3.3 – Tertiary Filtration

The existing tertiary filters consist of one AquaDiamond cloth media filter No. 1 with an average day design flow of 6.0 MGD and a peak hour hydraulic capacity of 15.0 MGD, and one traveling hood sand filter No. 2 with a peak hour hydraulic capacity of approximately 2.40 MGD. Improvements to the tertiary filters are recommended to replace traveling hood sand filter No. 2 with an AquaDiamond cloth media filter matching filter No. 1 to improve redundancy and increase the peak hour firm capacity to 15 MGD. The total estimated capital cost of replacement of filter No. 2 is $2,204,000.

Future expansion of the tertiary filters will be required to provide sufficient firm capacity at the 2040 peak hydraulic capacity of the WWTF of 19.5 MGD. Future expansion of the tertiary filters to meet 2040 hydraulic conditions is recommended to include the construction of a third tertiary filter utilizing a Hydrotech Discfilter unit, with a total estimated capital cost of $1,764,000.

### ES.3.4 – Disinfection and Post-Aeration

Existing UV disinfection at the WWTF consists of a Trojan UV4000 unit that is in immediate need of replacement due to advanced wear and inadequate hydraulic and treatment capacity. A new UV disinfection channel No. 2 is recommended to be constructed between the existing disinfection channel and the existing utility building. The existing UV disinfection equipment is recommended to be maintained to provide additional disinfection redundancy, if needed. Construction of the new UV disinfection channel is recommended to include a common influent channel to promote equal flow splitting between disinfection channels, connection to the existing NPW wet well and cascade reaeration steps, replacement of the existing fiberglass grating on the existing disinfection channel with solid covers, and construction of a new canopy structure over both disinfection channels to prevent algae growth, protect the equipment from weathering, and provide additional protection from lightning damage. The total estimated capital cost of immediate improvements to the UV disinfection process is $2,800,000.

Future improvements to UV disinfection, post-aeration, and the effluent outfall pipeline are recommended to meet 2040 design conditions. Future improvements to these processes are recommended to include:

- Retrofit the existing UV disinfection channel with new UV disinfection equipment to match UV disinfection channel No. 2
- Replace the existing cascade reaeration steps to ensure effluent DO permit limits are met
- Replace the existing 36-inch outfall pipeline to alleviate hydraulic bottlenecks at the FEMA 100-year flood conditions

The total estimated capital cost of the future improvements to UV disinfection, post-aeration, and effluent outfall pipeline is $3,127,000.

### ES.3.5 – Biosolids

Improvements to the existing biosolids processes at the WWTF are recommended to improve thickening and dewatering operational flexibility, to replace aging equipment, and to provide greater opportunities for beneficial reuse of biosolids. The following improvements to the biosolids processes are recommended:

- Rehabilitation of the existing gravity thickeners
- Construction of aerated TWAS storage tanks after gravity thickening prior to dewatering
- Replacement of the existing dewatered cake conveyor
- Replacement of the existing dewatering belt filter presses and associated polymer feed systems

- Construction of a new biosolids thermal drying facility

The total estimated capital cost for all improvements to the biosolids processes is $18,772,000. Most of the total estimated capital cost of the biosolids improvements is associated with the future construction of a biosolids thermal drying facility, which has an estimated capital cost of $11,231,000.

## ES.4 Schedule for Improvements

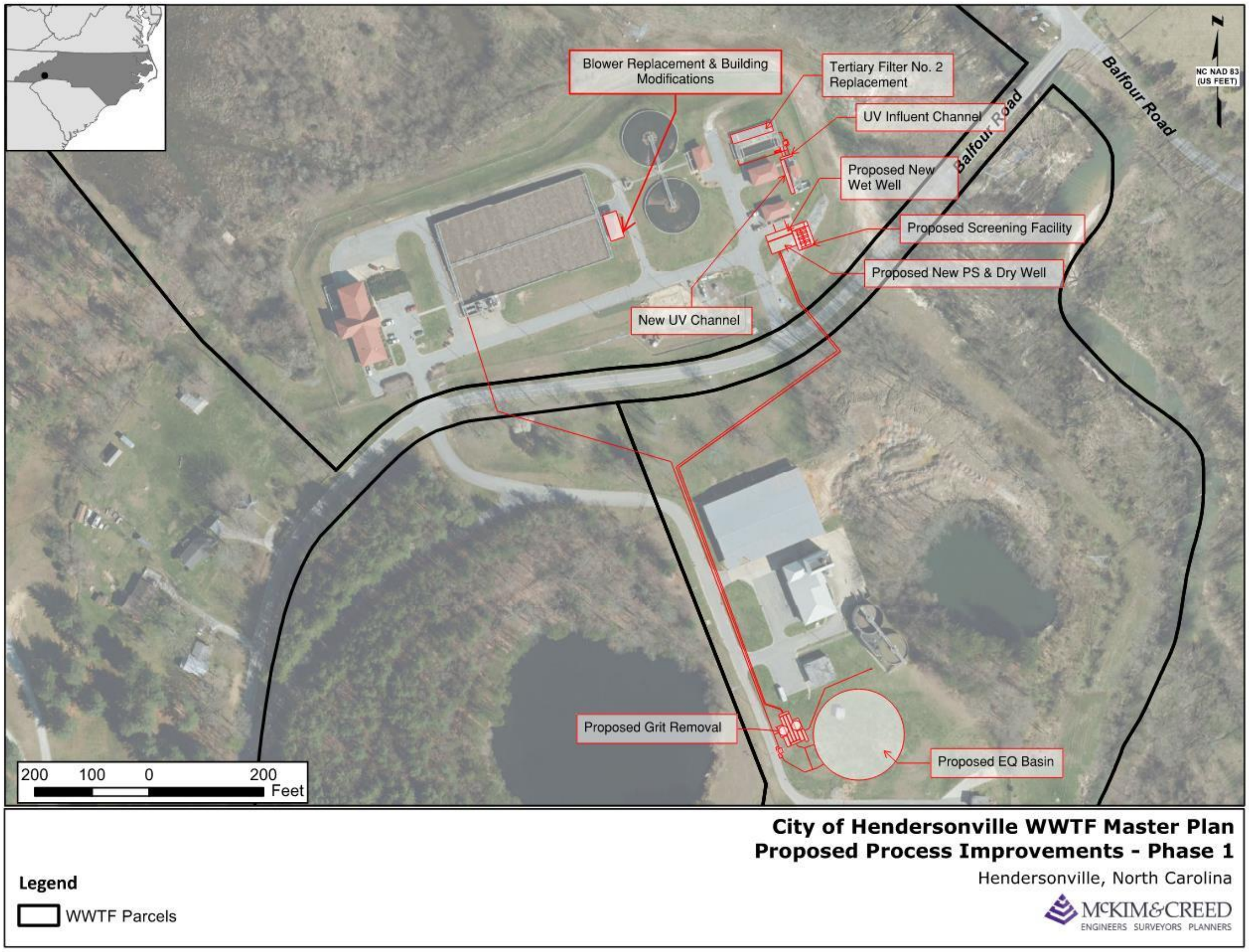
The recommended improvements to the WWTF have been grouped into three primary phases based on the immediacy of their needs. These three phases are summarized below:

### ES.4.1 Phase 1 – Immediate and Near-Term Needs

The Phase 1 WWTF Improvements are represented in **Figure ES.0.1** below, and are recommended to consist of the following:

- Construction of a new UV disinfection channel No. 2
- Replacement of tertiary filter No. 2 to match AquaDiamond filter No. 2
- Expansion of the influent pumping station
- Construction of a new screening facility upstream of the expanded headworks
- Construction of a new grit removal facility upstream of the proposed inline EQ basin
- Construction of a new inline flow EQ basin
- Blower replacement and blower building improvements
- Dewatered cake conveyor replacement
- RAS/WAS pump replacements and recycle pumping station improvements

*Figure ES.0.1 – Proposed Process Improvements Phase 1*

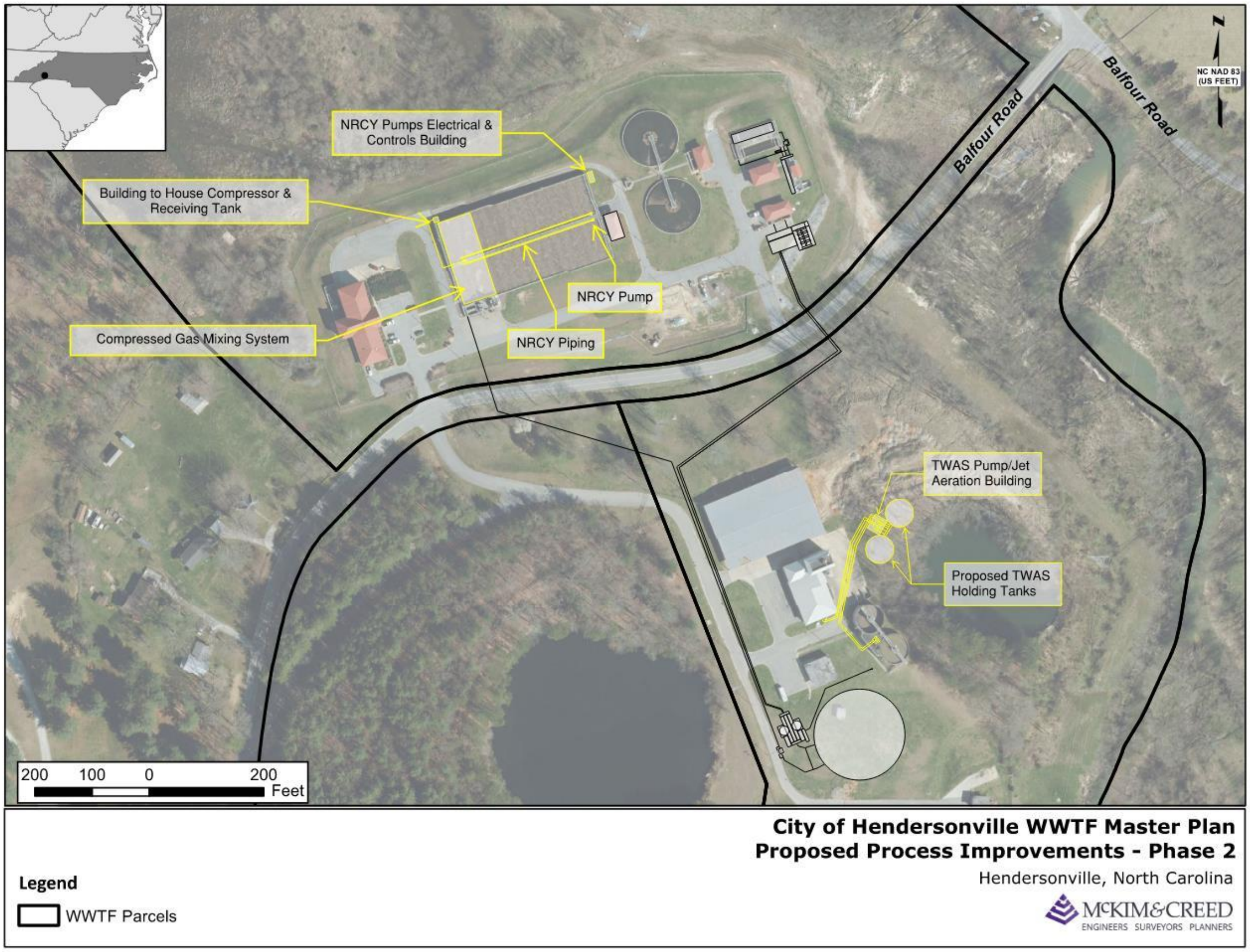



## ES.4.2 Phase 2 – Intermediate Needs

The Phase 2 WWTF Improvements are represented in **Figure ES.0.2** below, and are recommended to consist of the following:

- Rehabilitation of the existing gravity thickeners and construction of new TWAS storage
- Construction of a new biosolids thermal drying facility
- Conversion of the existing extended aeration process to a Modified Ludzack-Ettinger process including anoxic zone mixing and nitrified internal mixed liquor recycle pumps and piping
- Replacement of the existing dewatering belt filter presses and belt filter press polymer feed systems

*Figure ES.0.2 – Proposed Process Improvements Phase 2*



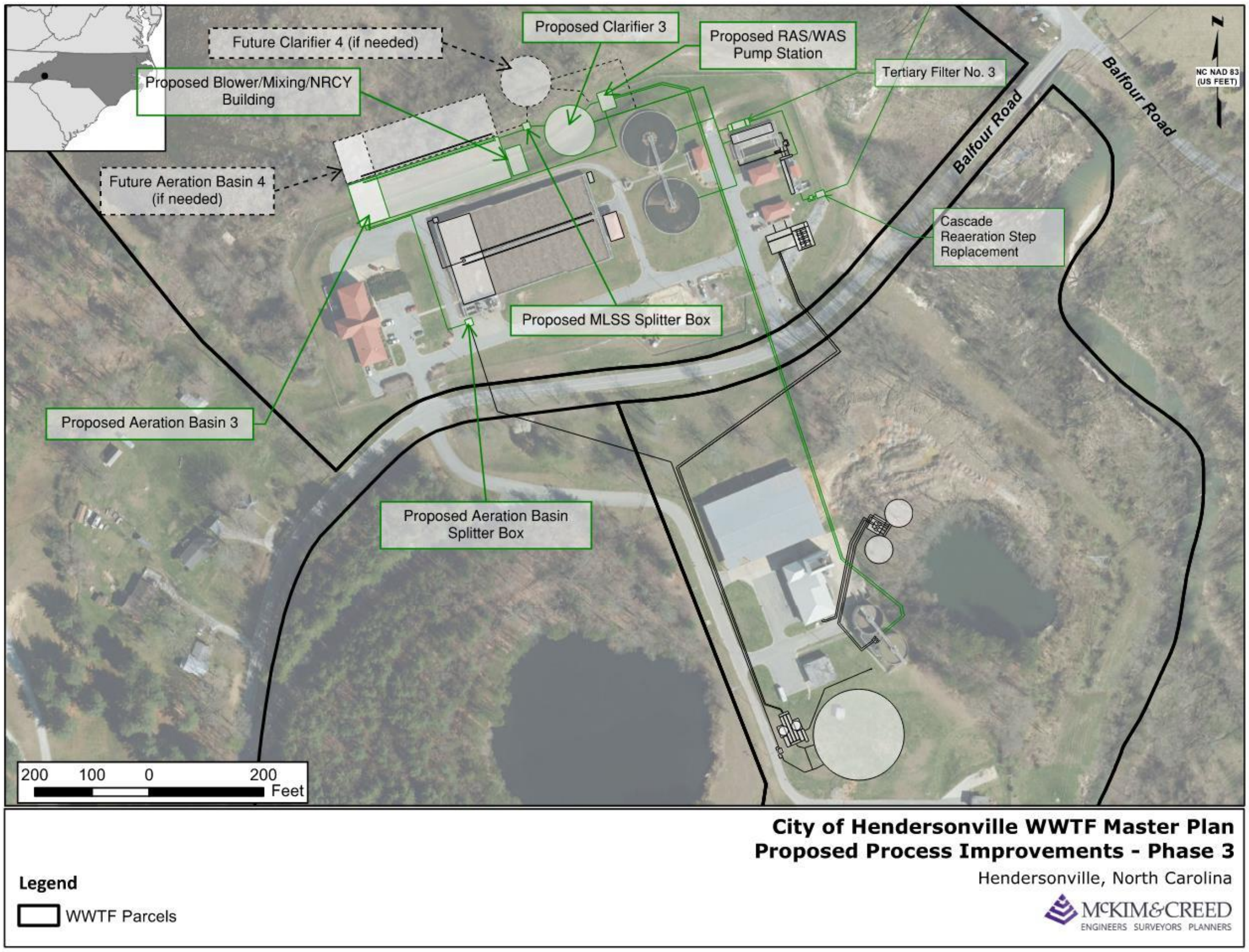

## ES.4.3 Phase 3 – Long Term Future Needs

The Phase 3 WWTF Improvements are represented in **Figure ES.0.3** below, and are recommended to consist of the following:

- Expansion to a third MLE secondary treatment train to meet 2040 loading conditions
- Construction of tertiary filter No. 3
- Expansion of UV disinfection through retrofits to the existing UV disinfection channel to match UV disinfection channel No. 2

- Replacement of cascade reaeration steps
- Replacement of the effluent outfall

*Figure ES.0.3 – Proposed Process Improvements Phase 3*



# 1. INTRODUCTION

## 1.1 Background

The City of Hendersonville retained the services of McKim & Creed to prepare a master plan for the City's existing wastewater treatment facility (WWTF). The City of Hendersonville's WWTF is permitted under North Carolina Department of Environmental Quality (NCDEQ) NPDES Permit No. NC0025534 to treat 4.8 million gallons per day (MGD) maximum month flow (MMF) with an extended aeration activated sludge domestic wastewater treatment plant. The current NPDES permit also includes provisions for a future permitted capacity of 6.0 MGD at MMF upon issuance of an Authorization to Construct (ATC) for expansion to the existing WWTF. Construction of the existing 4.8 MGD WWTF was completed in 2001, and since that time a comprehensive study to evaluate the ability of the WWTF to meet the City's future wastewater treatment needs has not been completed. The purpose of this master plan is to evaluate, identify, and schedule recommended improvements to ensure that the WWTF continues to meet the City's wastewater treatment needs. This master plan has been prepared for a planning period extending to 2040.

The WWTF Master Plan has been organized into three separate technical memoranda that will be combined to inform the formation of the comprehensive capital improvement plan (CIP) for the WWTF. Technical Memorandum No. 1 – Preliminary Evaluations and Condition Assessments provided review of previous engineering studies and flow projections, and described the findings and recommendations of condition assessments of the existing treatment processes and major equipment. The objectives of this Technical Memorandum No. 2 are described below. Technical Memorandum No. 3 has been prepared in conjunction with this technical memorandum to complete a preliminary engineering evaluation to provide flow equalization facilities at the WWTF. The recommendations of this Technical Memorandum No. 2 have been coordinated with Technical Memorandum No. 3 to ensure a cohesive and comprehensive master plan.

## 1.2 Objectives

Following completion of the existing condition assessments and Technical Memorandum No. 1, the next step of the master plan was to evaluate capacity limitations of the overall facility and each treatment process, and identify improvements necessary to overcome capacity limitations. The objective of this Technical Memorandum No. 2 is to provide improvement and expansion recommendations for each treatment process and the overall facility that will restore and maintain reliability, improve treatment and hydraulic limitations, improve efficiencies, resolve operational issues, continue to meet existing permit conditions, and meet future treatment needs and permit conditions. This is to be accomplished through current capacity analyses for each treatment process using industry developed standards and manufacturer specific information, overall facility hydraulic profile evaluations, plant process modeling using BioWin wastewater modeling software, and evaluation of treatment process and technology alternatives. These improvements, along with the previous recommendations of the existing condition assessments, are used to develop a list of recommended facility improvements, including conceptual cost opinions and a schedule for completion.

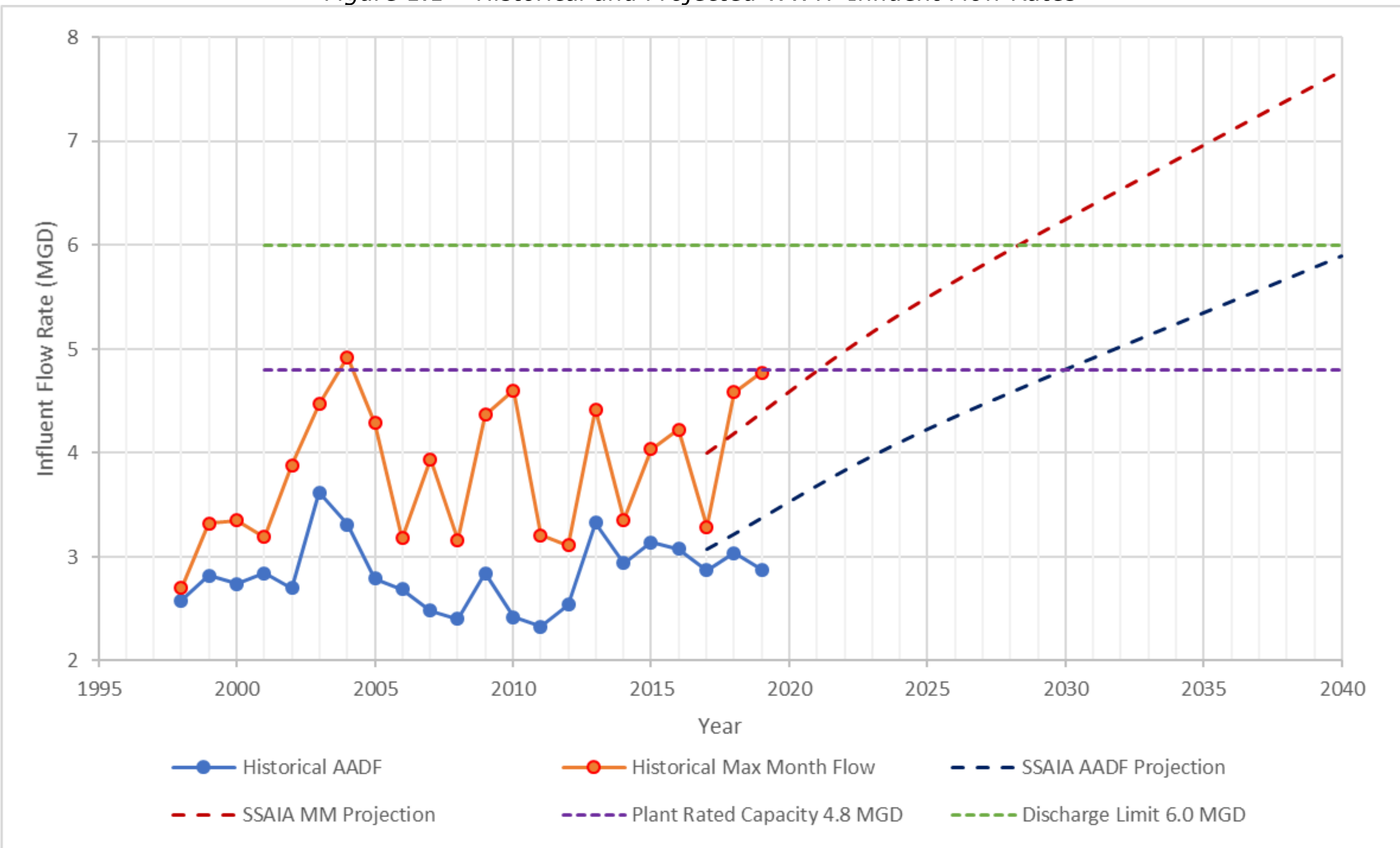
## 1.3 Basis of Evaluation

All alternatives for improvements and expansion were evaluated based on the current permitted capacity of the facility as well as the future wastewater treatment needs established by the flow projections described previously in Technical Memorandum No. 1. The future WWTF design conditions used as the basis for evaluation of process improvement and expansion alternatives were assigned based on the previous flow projections and application of the 80/90% rule per 15A NCAC 02T .0118. In general, the 80/90% rule states:

- An engineering evaluation of future expansion needs (i.e. the WWTF Master Plan) must be completed if the AADF of any calendar year exceeds 80% of the permitted capacity, and
- All permits required for expansion must be acquired, and plans and specifications for the expansion must be submitted prior to the AADF of any calendar year exceeding 90% of the permitted capacity

The future influent flow projections were previously presented and discussed in Technical Memorandum No. 1 of this master plan. The future influent flow projections are also shown in **Figure 1.1** below for reference.

*Figure 1.1 – Historical and Projected WWTF Influent Flow Rates*



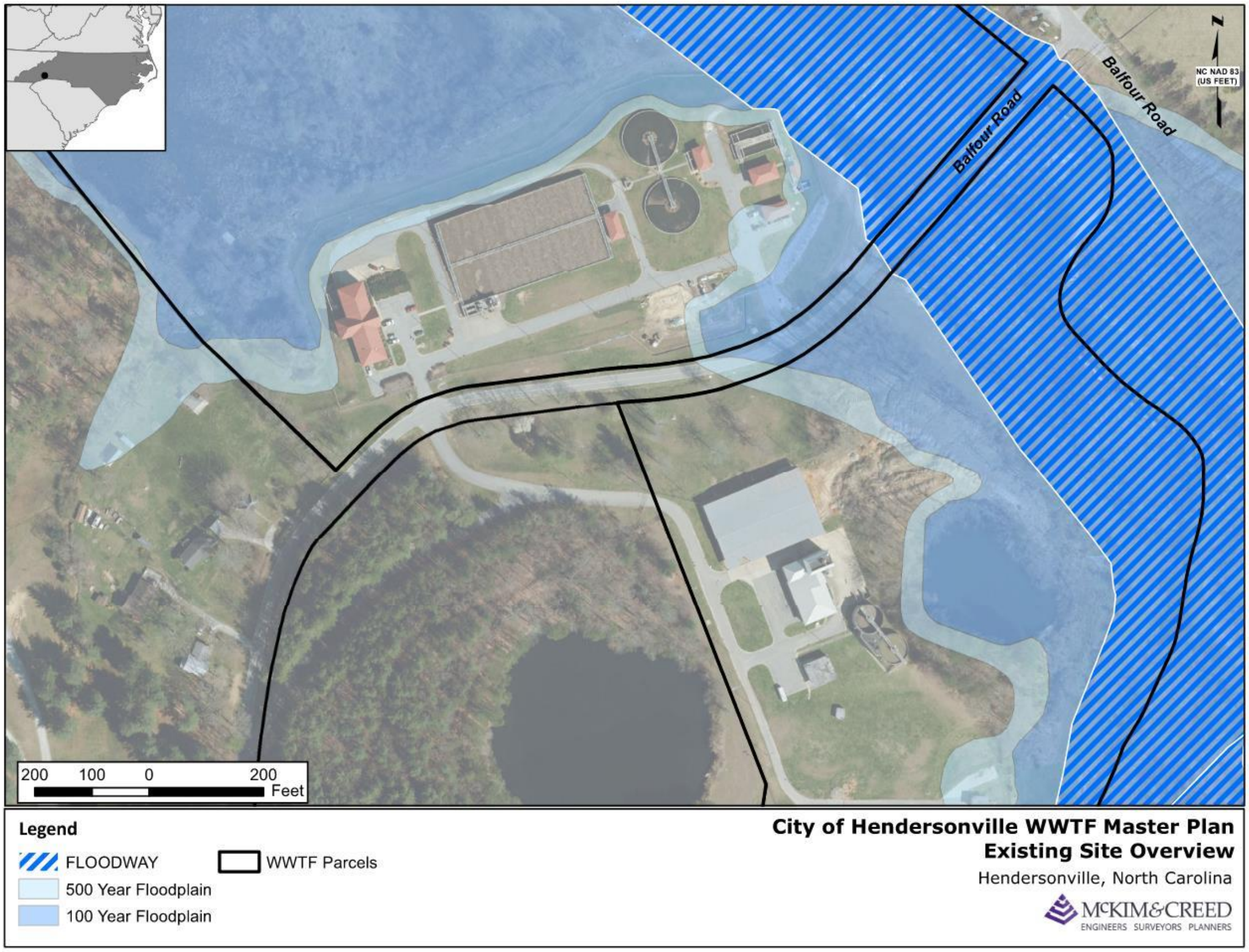

Using the 80/90% rule and the future influent flow projections, the expected timing of expansions and the associated permitted and peak hour hydraulic capacities of the WWTF are listed in **Table 1.1** below. The first expansion to a permitted capacity of 6.0 MGD is based on current permit provisions for this future discharge limit. The next expansion to a permitted capacity of 7.8 MGD is based on the 2040 maximum month flow projection of 7.68 MGD to ensure maximum month flows do not result in permit violations. The actual required timing of expansions to the existing WWTF will be based on actual flows and loading to the facility. Actual future flows and loading to the WWTF may necessitate completion of expansions sooner or later than shown below.

*Table 1.1 – Summary of Expected Timing of Future WWTF Expansions*

| Permitted WWTF Capacity (MGD) | WWTF Hydraulic Capacity (MGD) (PF = 2.5) | AADF at 90% of Permitted Capacity | Year Expansion Expected to be Completed |
|---|---|---|---|
| 4.8 (current capacity) | 12.0 | 4.32 | 2025 |
| 6.0 | 15.0 | 5.40 | 2035 |
| 7.8 | 19.5 | 7.02 | 2050* |

The location of the existing WWTF is a focal point of importance when considering alternatives for future improvements and expansions due to its location adjacent to the FEMA 100-year floodplain and floodway of Mud Creek as shown in **Figure 1.2** below. The existing WWTF was constructed in historical floodplain and floodway which required extensive site grading to raise the site elevation above the FEMA 100-year floodplain. The existing floodplain and floodway areas remaining to the north and east of the WWTF are suspected to consist of jurisdictional wetland areas and potential habitats for endangered and threatened species, however ecological investigations of this area were outside the scope of this master plan. Past geotechnical explorations at the site indicate that most of the existing WWTF is underlain by alluvial deposits from Mud Creek, with poorly consolidated fill materials placed directly over during the construction of the existing WWTF. A majority of the existing process structures were constructed on timber pile foundations, indicating that future improvements to the site would also be likely to require pile foundations. The alternatives evaluated herein gave consideration to these existing site conditions and their potential impacts to project costs and feasibility.

*Figure 1.2 - Existing Site Overview*



# 2. PRELIMINARY TREATMENT EVALUATION

## 2.1 Purpose and Background

The purpose of this section is to analyze the current capacity of the preliminary treatment process units at the WWTF both hydraulically and in terms of the relevant process unit treatment design criteria. The preliminary treatment processes include the influent pumping station (IPS), screening, and grit removal.

All wastewater treated by the WWTF flows to the IPS from the 42-inch Mud Creek Outfall. Influent wastewater flows through a manual trash rack prior to a Parshall flume before discharging into the IPS wet well below the influent channel. Influent wastewater is then pumped up to the screening and grit removal equipment on the western side of the WWTF adjacent to the aeration basins. Flow from the IPS enters a common influent channel at the screening and grit removal structure, where it is split into two channels, each with one mechanical bar screen followed by one aerated grit chamber. Reciprocating rake mechanical bar screens are utilized with a 3/8-inch bar spacing.

After preliminary treatment, wastewater flows by gravity to the aeration basins for secondary treatment. The existing WWTF does not utilize flow equalization. Future improvements to the WWTF are recommended to include flow equalization (EQ), which is to be installed downstream of the preliminary treatment process units and upstream of the remainder of the WWTF's process units. As a result, the preliminary treatment train must be adequately sized to handle the non-equalized 2-year storm peak flows listed in **Table 2.1** below, while process units downstream of equalization will be sized to handle the WWTF hydraulic capacity flows also listed in the table. Detailed sizing information, alternatives evaluation, and design criteria for future flow equalization facilities at the WWTF are described in more detail in Technical Memorandum No. 3 of this master plan.

*Table 2.1 - Summary of Current and Projected WWTF Flows*

| Year | Permitted WWTF Capacity (MGD) | WWTF Hydraulic Capacity (MGD) (PF = 2.5) | 2-Year Storm Peak Flow – Non-equalized (MGD) |
|---|---|---|---|
| Base (2017)* | 4.8 (current capacity) | 12.0 | 17.4 |
| 2025 | 6.0 | 15.0 | 22.5 |
| 2040 | 7.8 | 19.5 | 28.3 |

**Base year established by SSAIA Master Plan report*

In this section, the current capacity analysis for each process unit will describe the existing capacity limitations (hydraulically and otherwise), and the alternatives evaluation will discuss potential future improvements for each process unit that will be capable of handling future design conditions.

# 2.2 Influent Pumping Station

## 2.2.1 Current Capacity Analysis

The existing IPS at the WWTF is equipped with four centrifugal dry pit pumps and two interconnected parallel 16-inch force mains that discharge to the screening and grit collection channels. The two force mains can be isolated by closing the interconnecting 16-inch plug valve. Each force main is fed by one 75 hp pump and one 125 hp pump operating in parallel.

The current capacity of the existing IPS is based on the firm pumping capacity and velocity of flow in the force mains. Capacities for individual pumps and pumps in parallel were evaluated assuming the plug valve between the force mains located within the IPS is closed, hydraulically isolating the force main for a worst case condition. The firm capacity of the IPS is 12 MGD with the plug valve between the force mains located within the IPS open, which is how the IPS has historically been operated.

**Table 2.2**Table 2.2 - Velocity in Influent Pump Station Force Main below indicates flow velocities in the 16-inch force main when the pumps are operating at full speed under current flow conditions.

*Table 2.2 - Velocity in Influent Pump Station Force Main*

| Operating Condition | Flow (MGD) | TDH (ft) | Velocity in Force Main (ft/sec) |
|---|---|---|---|
| Pump 1 or 2 (75 hp) Operating[1] | 4.9 | 65 | 5.4 |
| Pump 3 or 4 (125 hp) Operating[1] | 6.7 | 75 | 7.5 |
| Pumps 1 & 3 or 2 & 4 (75 hp + 125 hp) Operating in Parallel[1] | 7.5 | 94 | 8.3 |

*[1]Assumes one force main is in operation and is hydraulically isolated from the other force main*

**Table 2.3** below lists the recommended force main velocities based on industry standards. The preferred and recommended maximum flow velocity in the force main is 6.0 ft/sec to minimize head loss and the effects of water hammer and to provide flexibility for future growth. Assuming an equal flow split between force mains, utilizing the existing 16-inch force mains for future PHFs would slightly exceed the maximum recommended velocity of 8 ft/sec in both force mains transporting the 2025 WWTF hydraulic capacity of 15 MGD listed above in **Table 2.1**.

*Table 2.3 - Recommended Force Main Velocities*

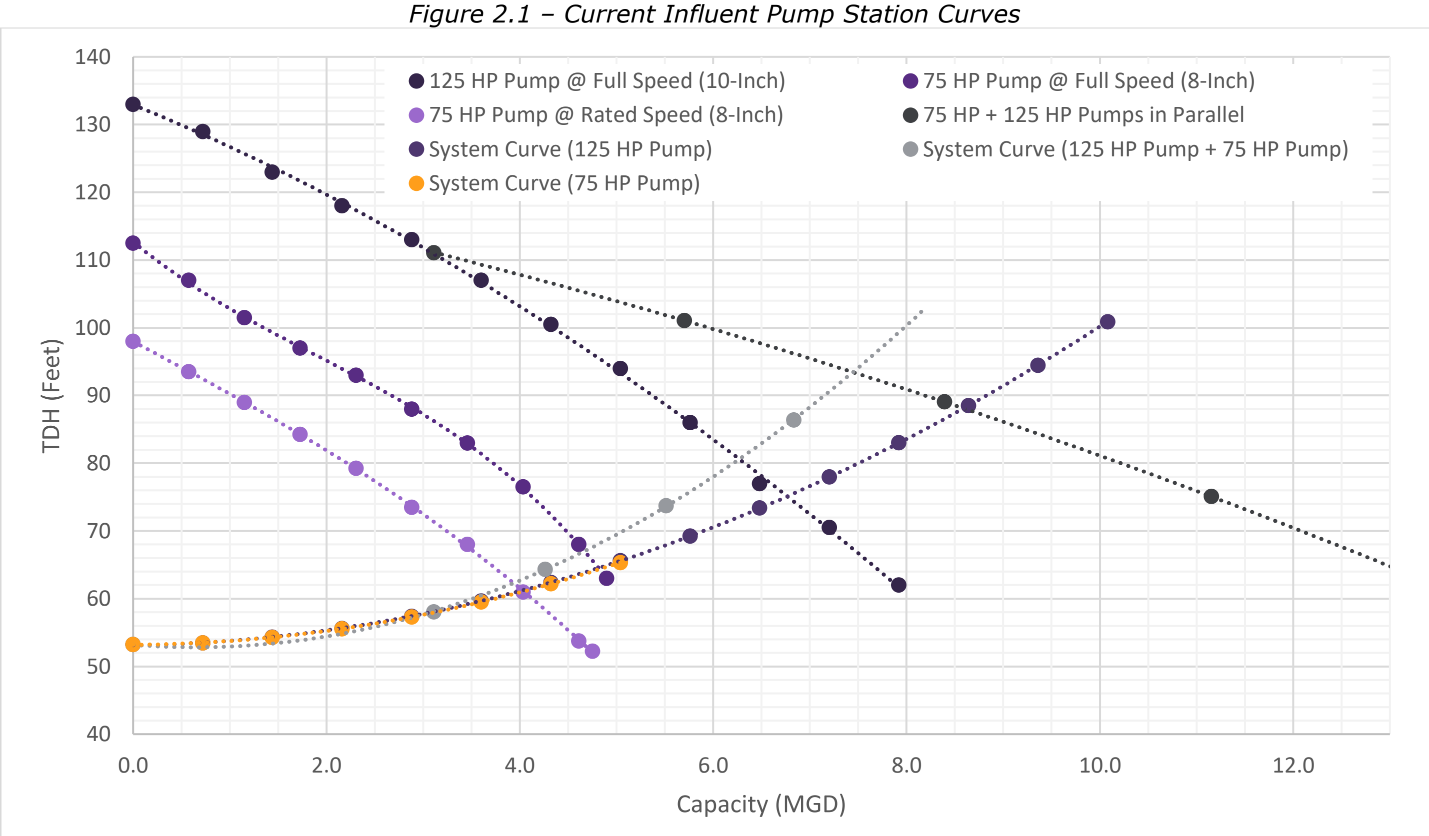
| Velocity in Force Main (ft/sec) | Flow in One 16-inch Force Main (MGD) | Flow in Two 16-inch Force Mains (MGD) | Description |
|---|---|---|---|
| 2.0 | 1.8 | 3.6 | Minimum velocity for cleansing grit[1] |
| 6.0 | 5.4 | 10.8 | Recommended maximum velocity to reduce head loss and severity of water hammer[2] |
| 8.0 | 7.2 | 14.4 | Maximum velocity to avoid high head loss and protect valves[1,2] |

*[1]10 State Standards, [2]Jones, 2008*

Although 8 ft/sec can sometimes be exceeded to accommodate intermittent high flow conditions, velocities would need to exceed 12 ft/sec in both 16-inch force mains to accommodate either the 2025 or 2040 PHF. These flows conditions, although intermittent, involve significant head loss, increased wear on the influent pumps, and increased potential for hydraulic transients. Therefore, it is recommended the existing IPS force mains be upsized to accommodate future PHFs if the existing pump station will be used to handle flows exceeding 15 MGD.

Capacity of the influent pumps is determined by conducting a Hazen Williams analysis of the pumping system from the free surface of the wet well to the free surface of the common channel where the two 16-inch force mains discharge. System heads are then calculated for incremental flows to generate system curves that are plotted against the pump curves provided from equipment data sheets. From this process, the existing firm pumping capacity of the influent pump station was determined to be 12.4 MGD (assuming the force mains are hydraulically disconnected); the sum of a 75 hp and 125 hp pump operating in parallel on one system curve (7.5 MGD) plus the 75 hp pump on a separate system curve (4.9 MGD). The influent pump station system curves are illustrated in **Figure 2.1** below.

*Figure 2.1 – Current Influent Pump Station Curves*



When operating both in parallel and on its own, the 125 hp pump operates within approximately one percentage point of its best efficiency point (BEP) of 78%. When operating on its own, the 75 hp pump's efficiency is approximately 76%, compared to its BEP of approximately 81%. This efficiency drops to approximately 67% when operating in parallel with the 125 hp pump. When operating at full speed, the 75 hp pump is running very close to the end of its window of optimal performance where pump performance begins to deteriorate and vibration increases. This operating point is also the closest to the current AADF of 4.8 MGD, indicating that the most frequent operating condition coincides with either a single 75 hp pump operating near the end of its curve, or a set of two or more pumps operating away from their BEPs.

From this capacity analysis, it is recommended that both the IPS pumps and force mains be upsized by 2025 to accommodate future peak hour flows. The possibility of expanding the IPS within its existing footprint to accommodate future PHFs is discussed under the first alternative listed in the subsequent section.

### 2.2.2 Alternatives Evaluation

As noted previously, it is recommended that future improvements be made to the influent pump station to handle future hydraulic loading conditions at the time of plant expansion or as part of construction of flow equalization facilities. The WWTF influent pump station is upstream of all equalization facility configurations considered (i.e., inline vs. offline, existing plant site vs. old plant site), and therefore must handle non-equalized future peak hours flows as listed below in **Table 2.4**, as well as future annual average daily flows.

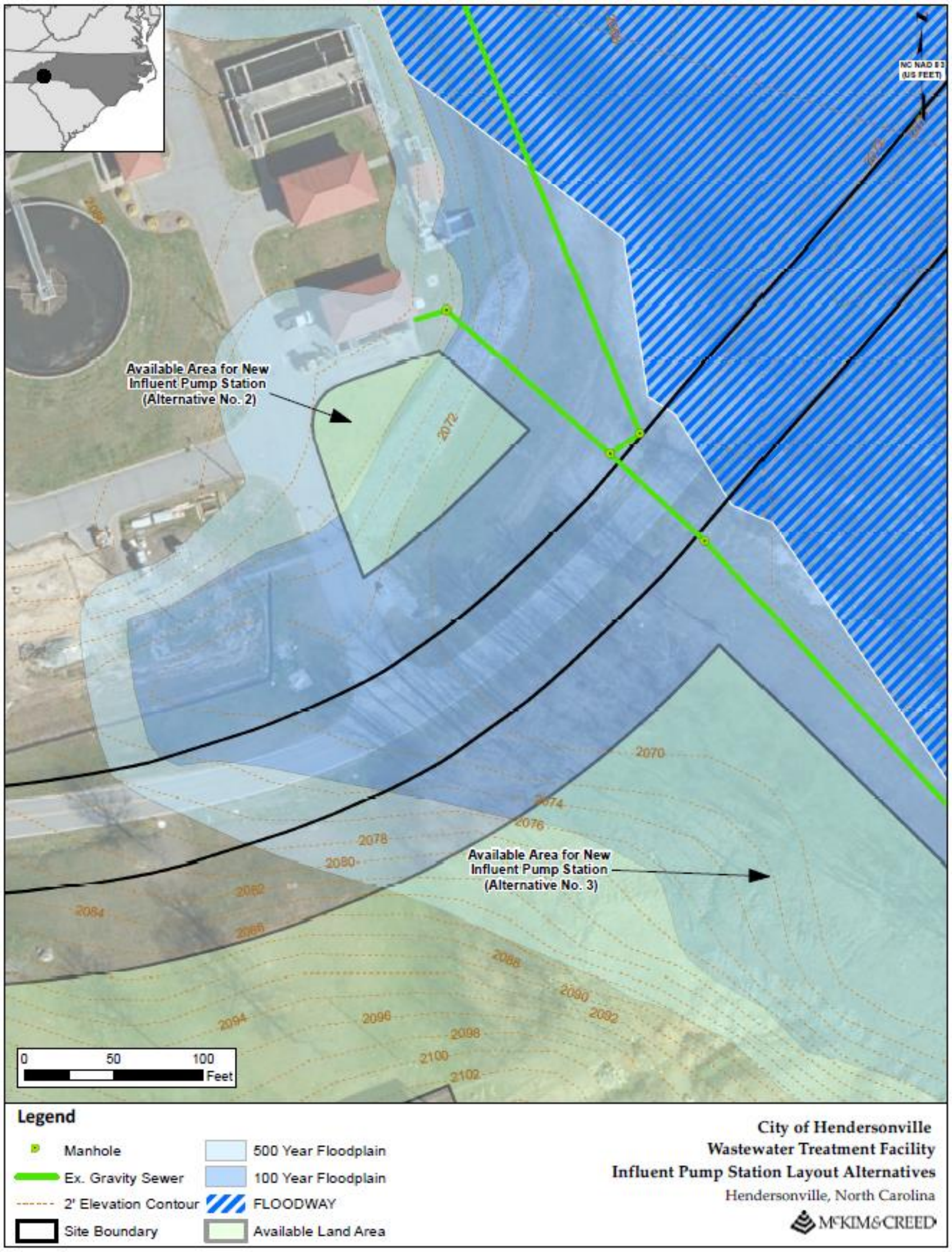
*Table 2.4 - Future Non-Equalized Flow Projections*

| Projection Year | Non-Equalized AADF (MGD) | Non-Equalized PHF (MGD) |
|---|---|---|
| 2025 | 4.23 | 22.5 |
| 2040 | 5.9 | 28.3 |

The WWTF influent pumps have historically required frequent de-ragging. The passage of rags and other debris into downstream processes is one of the most common causes of jammed pump impellers and maintenance and repair of other equipment at wastewater treatment facilities. It is therefore recommended that screening facilities be located upstream of the influent pump station for all influent pump station improvement alternatives to protect the pumps and all downstream equipment.

Three primary alternatives for influent pump station facility upgrades capable of handling future hydraulic conditions are considered subsequently. Alternative 1 (the baseline alternative) would occur utilizing the existing IPS dry well. Alternatives 2 and 3 would include new IPS facilities located in the available areas shown in **Figure 2.2**. These areas are bounded by the existing Mud Creek Outfall to the east and a 50-foot setback from the property line along Balfour Road per NCDEQ minimum setbacks for treatment facility structures. The finished grade around each facility must be elevated out of the FEMA 100-year floodplain (elevation of 2,076.4 feet), and the selected alternative must maintain firm pumping capacity to the WWTF throughout construction.

*Figure 2.2 - Available Land Area for IPS Alternatives No. 2 and No. 3*

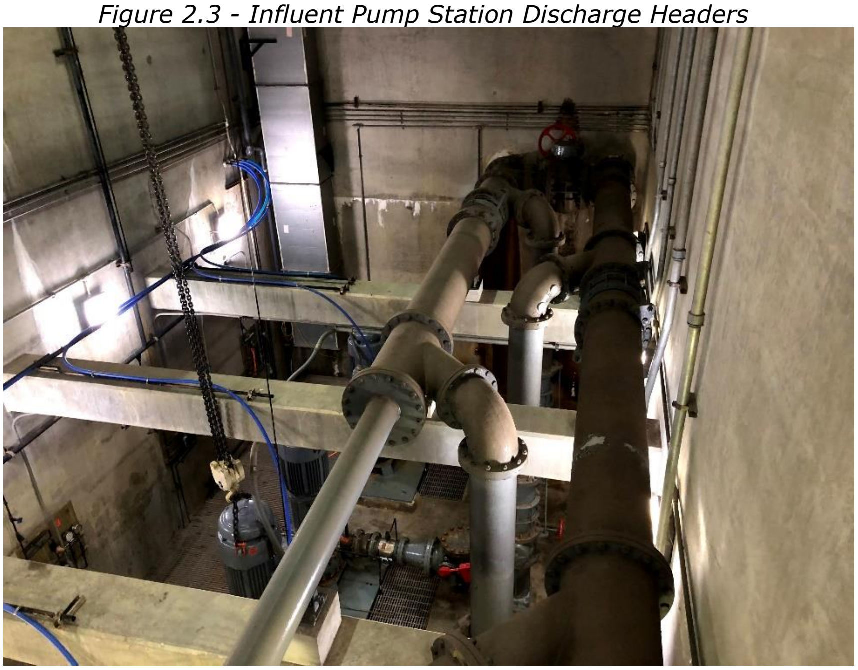


#### 2.2.2.1 *Alternative 1: Upsize Pumps and Force Mains in Existing Pump Station*

The influent pump station structural condition assessment in Technical Memorandum No. 1 concluded that the service life of the facility's structural elements can be prolonged by conducting structural repairs. Therefore, modifications to the existing influent pump station can be evaluated to determine whether it can be utilized to provide partial or total firm pumping capacity for the future hydraulic loading conditions within the existing footprint. This alternative evaluates the potential firm capacity available at the existing pump station by upsizing the existing pumps and associated electrical equipment as well as the suction / discharge piping and force mains.

The influent pump station has two 16-inch discharge headers, each connected to the discharge piping of both a 75 hp and 125 hp influent pump via a 16-inch by 12-inch wye fitting as shown in **Figure 2.3**.

*Figure 2.3 - Influent Pump Station Discharge Headers*



It is recommended that the flow velocity in the discharge piping and header not exceed 8 ft/sec at peak hour flows to avoid high head loss and to protect valves; therefore, the maximum capacity of each 16-inch discharge header is 7.2 MGD. Assuming both discharge headers are flowing at capacity, the total discharge header capacity is 14.4 MGD, which is insufficient to transport the 2025 or 2040 PHF flow. Discharge piping and header velocities would exceed 12 ft/sec to accommodate future PHFs. All force main piping inside and outside the existing IPS would need to be upsized to at least a 20-inch pipe for 2025 PHF and 24-inch pipe for 2040 PHF assuming a maximum allowable flow velocity of 8 ft/sec as illustrated in **Table 2.5** below.

*Table 2.5 - Minimum Force Main Diameters to Handle Future PHFs*

| Force Main Pipe Diameter (in) | Flow in Two Force Mains @ 8 ft/sec (MGD)[1] |
|---|---|
| 16 | 14.4 |
| 18 | 18.3 |
| 20 | 22.6 |
| 24 | 32.5 |

*[1]2025 PHF = 22.5 MGD, 2040 PHF = 28.3 MGD*

Inside the dry well, upsizing the 16-inch discharge headers would require installation of new larger diameter headers above the existing piping with new core holes through the existing dry well wall. Once the new headers are installed above the existing headers, each pump may be removed from service (individually) and connected to the new headers. Several significant constructability concerns are anticipated for this potential upsizing project, including potential structural effects of larger wall cores, establishing means of structurally supporting the new headers, means of plugging the existing discharge headers in the dry well during pump change over, protecting the operating pumps during construction, and the potential need for extended bypass pumping, and relocating the interconnecting plug valve between force mains to a buried location outside the dry well due to inadequate space between upsized discharge headers.

The hydraulic capacity of the existing IPS can be increased by upsizing the pumps, however, increases beyond a certain power requirement will require modifications to the pump station's power distribution system as illustrated by the examples listed in **Table 2.6** below. To minimize static head and therefore reduce pump horsepower requirements, this alternative assumed that the influent pump station would discharge to screening and grit removal process units at or near their existing location and elevation. This allows for the possibility of the screening and grit removal technologies to be replaced with new technologies, e.g. chain-driven multi-rake screens and vortex grit removal. However, this alternative does not accomplish the recommendations of Technical Memorandum No. 1 to relocate screening ahead of the influent pumping station. This alternative also assumes that the EQ facilities would be located downstream of the screening and grit removal facilities. This alternative will require an additional pump station to transport either EQ basin influent or effluent flows depending on EQ basin location.

*Table 2.6 - Existing IPS Power Distribution Changes from Pumps Modifications*

| Option # | Modification | Result |
|---|---|---|
| 1 | Replace (2) existing Pumps No.1 and No.2 (75 hp) with new 125 hp pumps/motors<br>Replace (2) existing Pumps No.3 and No.4 (125 hp) with new 125 hp pumps/motors | No changes to existing power distribution system |
| 2 | Replace (2) existing Pumps No.1 and No.2 (75 hp) with new 125 hp pumps/motors<br>Replace (2) existing Pumps No.3 and No.4 (125 hp) with new 150 hp pumps/motors | No changes to existing power distribution system |
| 3 | Replace (2) existing Pumps No.1 and No.2 (75 hp) with new 150hp pumps/motors<br>Replace (2) existing Pumps No.3 and No.4 (125hp) with new 150hp pumps/motors | 800A Panelboard 'HPI' requires upsizing to 1000A to include upsized feeder cabling |
| 4 | Replace (2) existing Pumps No.1 and No.2 (75 hp) with new 200 hp pumps/motors<br>Replace (2) existing Pumps No.3 and No.4 (125 hp) with new 200 hp pumps/motors | 800A Panelboard 'HPI' requires upsizing to 1200A to include upsized feeder cabling |

Assuming upsizing of the discharge piping to 16-inches and the force mains to 24-inch, along with assuming a 70% pump efficiency and an overall motor and drive efficiency of 90%, a firm capacity of 22.5 MGD (2025 PHF) could be achieved with one 12 MGD (250 hp) and two 5.25 MGD (100 hp) pumps in the existing IPS with modifications to the existing power distribution system. Two 12 MGD (250 hp) pumps would be required to be installed to meet firm capacity requirements. The 12 MGD pumps would be used to handle higher flows and the two 5.25 MGD pumps would be used most often to handle the AADF with one pump running.

Increasing the hydraulic capacity of the existing IPS to accommodate the 2040 PHF limits the ability to select pumps that efficiently meet PHF and AADF with only three pumps for firm capacity. Accommodating the AADF may require operation of one or two pumps at significantly reduced speed, resulting in inefficient operation for the most common flow conditions. In addition, hydraulic conditions at the pump intake in the existing wet well is a significant concern at the 2025 PHF, which will only be exacerbated at the 2040 PHF. Assuming no pumps are running, the existing active wet well volume (16,360 gallons) will fill in just over one minute at the 2025 PHF of 22.5 MGD, and it will fill in under one minute at the 2040 PHF of 28.3 MGD. The existing wet well is significantly undersized at these conditions if one of the large pumps must be operated as a constant speed pump due to VFD failure. With VFD operation, the wet well filling time is less of a concern. Regardless, high flow velocities, significant pre-swirl ahead of the pump intakes, and a strong potential for air entrainment and cavitation is expected at future peak hour flows in the existing wet well. If this alternative is pursued, it is strongly recommended that a physical hydraulic model study of the wet well be conducted to evaluate the hydraulic conditions of the pump intakes at future PHFs. If a physical hydraulic model study confirms the presence of significant adverse hydraulic phenomena within the existing wet well, these conditions will lead to increased rates of wear on the pumps and shorter pump life cycles.

In addition to hydraulic concerns, Technical Memorandum No. 1 also noted that the City has experienced significant FOG build-up in the existing IPS wet well. FOG build-up in the wet well has been reported to have reduced the actual active working volume available, which may further exacerbate adverse hydraulic conditions in the wet well at increased future flows. Alternatives to modify the existing wet well to alleviate FOG build-up are limited due to the existing wet well's configuration. However, modified pump operational strategies may be implemented to minimize FOG build-up as much as possible by ensuring FOG is re-entrained in the pumped flow. If the existing IPS is to remain in operation under any alternative it is recommended that the City implement scheduled wet well pump downs to lower the wet well to the minimum water level on a frequent basis. This may be scheduled to occur once a day or once every couple of days to re-entrain floating FOG and debris, and scour solids off the bottom of the wet well. Engineering practice has shown that VFD use and constant liquid level operation in wastewater pump stations has a strong tendency to allow FOG build-up, and reduce the active working volume of the wet well. Breaking the cycle of constant or near-constant liquid level in the wet well by scheduling frequent pump downs will help to minimize FOG build-up without requiring modifications to the wet well.

**Table 2.7** below describes the methodology and assumptions used to determine capital cost estimates for all cost estimates throughout this Technical Memorandum.

*Table 2.7 - Capital Cost Estimate Assumptions & Methodology*

| Item | Category | Assumption | |
|---|---|---|---|
| 1 | Equipment | - | Equipment Vendor Budget Proposal |
| | **SUBTOTAL A** | | *= Equipment Budgetary Proposal Cost* |
| 2 | Mechanical Equipment Installation | 20% | of Subtotal A |
| 3 | Electrical Installation Costs | 20% | of Subtotal A |
| 4 | Instrumentation Installation Costs | 10% | of Subtotal A |
| 5 | Structural | - | Calculated for specific alternative |
| 6 | Civil | - | Calculated for specific alternative |
| 7 | Demo | - | Calculated for specific alternative |
| 8 | Mobilization & Demobilization | 4% | of Subtotal A + sum of items 2 through 7 |
| | **SUBTOTAL B** | | *= Subtotal A + Sum of items 2 through 8* |
| 9 | Permits | 1% | of Subtotal B |
| 10 | Risk & Liability Insurance | 1.5% | of Subtotal B |
| 11 | Performance & Payment Bonds | 2% | of Subtotal B |
| | **SUBTOTAL C** | | *= Subtotal B + sum of items 8 through 10* |
| 12 | General Conditions | 6% | of Subtotal C |
| 13 | Contractor's OH & P | 15% | of Subtotal C |
| | **SUBTOTAL D** | | *= Subtotal C + sum of items 11 through 12* |

| Item | Category | Assumption | |
|---|---|---|---|
| 14 | Contingency | 30% | of Subtotal D |
| | **OPCC** | | *= Subtotal D + line 13* |
| 15 | Engineering, Legal, & Administration | 25% | of Subtotal D minus sum of items 9 -11 |
| | **TOTAL CAPITAL COST** | | *= OPCC + Item 15* |

The conceptual cost opinion for Alternative 1 is included below in **Table 2.14**. This cost opinion assumes that all pumps in the existing pump station would be replaced, and the existing force mains would be upsized to 24-inch force mains in the immediate term to handle the future 2025 PHF. This cost opinion also assumes that all pumps would be replaced again to handle the future 2040 PHF, utilizing the 24-inch force mains installed in the immediate term improvements. The cost opinion does not include potential costs related to wet well modifications to address adverse hydraulic conditions. This cost opinion also does not include the cost for an additional EQ pump station to pump either EQ influent or effluent flows depending on the location and type of EQ basin to be constructed.

*Table 2.8 - Estimated IPS Alternative 1 Cost Opinion*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $990,000 |
| 2 | Mechanical | $337,000 |
| 3 | Electrical | $400,000 |
| 4 | Instrumentation | $99,000 |
| 5 | Structural | $54,000 |
| 6 | Civil | $924,000 |
| 7 | Demo | $60,000 |
| 8 | Mobilization & Demobilization | $116,000 |
| 9 | Indirect Costs | $137,000 |
| 10 | General Conditions & Contractor Markup | $656,000 |
| 11 | 30% Contingency | $1,133,000 |
| 12 | Engineering, Legal, & Administration | $910,000 |
| | **Total Cost Opinion** | **$5,816,000** |

Annual O&M costs were estimated based on annual maintenance requirements of the equipment and the average annual energy usage. Annual equipment maintenance costs were assumed to be equal to 2% of the equipment capital cost, and the average electricity cost at the WWTF was assumed to be $0.06 per kWh. These assumptions shall remain the same for all other alternatives in this technical memorandum unless otherwise noted. **Table 2.9** and **Table 2.10** show the annual O&M costs for current and future production, respectively.

*Table 2.9 - IPS Alternative 1 - Annual O&M Costs - 2021*

| Item | Annual Cost |
|---|---|
| Maintenance | $24,000 |
| Electricity | $33,000 |
| **TOTAL** | **$57,000** |

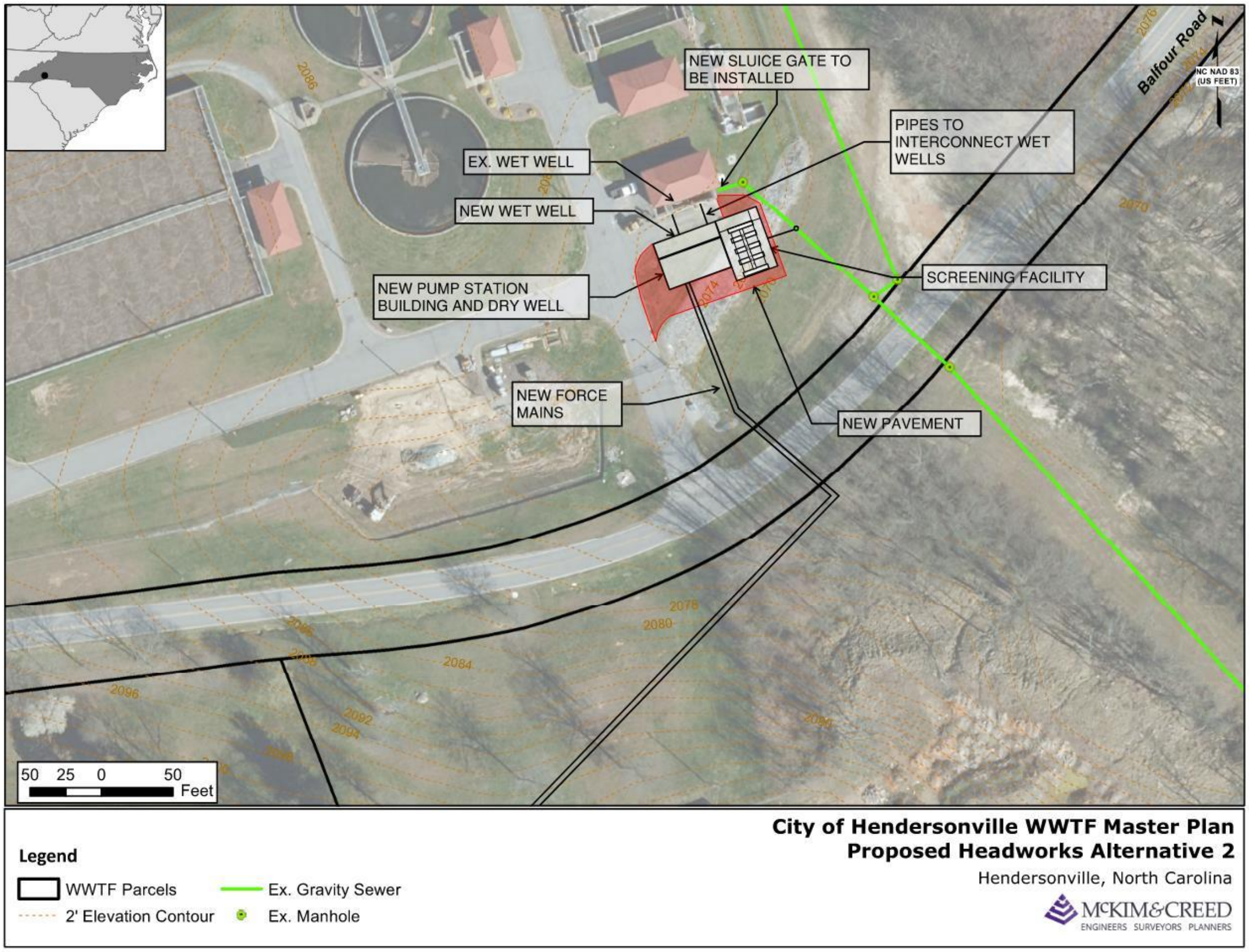
*Table 2.10 - IPS Alternative 1 - Annual O&M Costs - 2040*

| Item | Annual Cost |
|---|---|
| Maintenance | $24,000 |
| Electricity | $34,000 |
| **TOTAL** | **$58,000** |

#### 2.2.2.2 *Alternative 2: Maximize capacity of ex. IPS & build new IPS to handle flow exceeding its capacity*

This alternative consists of continuing to use the existing IPS and constructing a new IPS nearby to handle flows in excess of the existing IPS's capacity. A preliminary conceptual layout of this alternative is shown below in **Figure 2.4**, which also shows a potential screening facility location. The new IPS and wet well would be located to the south of the existing IPS and wet well. The new wet well is proposed to be connected to the existing wet well by at least two pipes to allow screened influent wastewater to be pumped from either wet well. Coring the wall and adding pipes is preferred over expanding the existing wet well to the south because it minimizes structural modification, simplifies construction, maintains current access drive location, and costs less. The discharge force mains from the existing and new IPS are proposed to be interconnected with electromagnetic flow meters installed for influent flow measurement. The layout accounts for maintenance access to both wet wells.

*Figure 2.4 - Headworks Alternative 2 Proposed Layout*




To achieve an increased firm capacity of 15.0 MGD within the existing IPS, preliminary force main sizing, pump selection, and electrical capacity analysis were conducted.

To size the pumps, first the number and size of force mains were selected to produce a system curve. Based on the recommendation of Technical Memorandum No. 3, it was assumed that the IPS force mains would be routed across Balfour Road to the proposed flow equalization (EQ) basin at the old plant site. To minimize material costs, a single force main fed by both 16-inch discharge headers was selected. As noted in **Table 2.3** above, the desired minimum and maximum velocity in force mains are 2 ft/sec and 8 ft/sec respectively, with the preferred maximum velocity at peak flows being 6 ft/sec. Flow in a single 24-inch force main exceeds the minimum velocity of 2 ft/sec at the 2025 AADF of 4.23 MGD and handles the firm capacity of 15 MGD at 7.4 ft/sec. It is therefore recommended that each existing 16-inch force main connect to a common 24-inch force main just outside of the existing IPS dry well.

For this alternative, the existing IPS would be designed to pump future AADFs and have a firm capacity of 15 MGD for the 2025 WWTF hydraulic capacity. To meet these design conditions near the pumps' BEPs and provide operational flexibility, it is recommended that two different pump sizes be installed. A typical overall pump and motor efficiency of 60% was assumed to determine the required firm capacity pump hp's. Maximum pump flows under the proposed head conditions were determined using a Hazen Williams analysis for combinations of 75, 100, 125, and 150 hp pumps as listed in **Table 2.11** below.

*Table 2.11 - Firm Capacity of Ex. IPS for Alternative 2*

| Pump Flow at Max Power (MGD)[1] | | 2 x 75 hp | 2 x 100 hp | 2 x 125 hp | 2 x 150 hp |
|---|---|---|---|---|---|
| | | 6.76 | 8.66 | 10.4 | 11.98 |
| 1 x 75 hp | *3.38* | 10.14 | 12.04 | 13.78 | 15.36 |
| 1 x 100 hp | *4.33* | 11.09 | 12.99 | 14.73 | 16.31 |
| 1 x 125 hp | *5.2* | 11.96 | 13.86 | 15.6 | 17.18 |
| 1 x150 hp | *5.99* | 12.75 | 14.65 | 16.39 | 17.97 |

*[1]Red indicates total flow<15 MGD, green indicates total flow≥15 MGD.*

Although the combination of either a single 75 hp or 100 hp pump with two 150 hp pumps would supply firm capacity, these combinations are not feasible because the limited capacity of a single 75 or 100 hp pump feeding a single discharge header would require the other header fed by two parallel pumps to have velocities near or above 12 ft/sec. Therefore, the most cost effective combination of pumps is anticipated to be two 125 hp pumps and two 150 hp pumps. This combination is preferred over four 125 hp pumps because having different sized pumps will provide multiple BEPs for conditions where one pump is running, increasing pumping efficiency over various flow conditions.

The existing power distribution system in the IPS is sized for two 75 hp and two 125 hp pumps. Assuming pumps No. 1, 2, and 3 are running simultaneously, a review of the existing electrical capacity found that replacing the existing pumps with two 125 hp pumps and two 150 hp pumps does not require changes to the existing power distribution system as described in **Table 2.6** above.

In summary, under Alternative 2, it is recommended that two 125 hp and two 150 hp pumps replace the current pumps in the existing IPS, all suction & discharge piping within the dry well will remain its current size, and each existing 16-inch force mains connect to a proposed common 24-inch force main just outside of the existing IPS dry well to increase the existing IPS firm capacity to 15 MGD.

The new IPS would handle all flow in excess of 15 MGD as shown in **Table 2.12** below. A dry well/wet well configuration was assumed for the new pump station for both IPS Alternatives 2 and 3 to match the layout of the existing IPS due to facility staff's familiarity with it. Dry well/wet well pump stations allow for easy access to pumps and equipment but have a higher initial capital cost than pump stations with only a wet well due to the added structure cost and additional associated excavation. Cost estimates for both Alternatives 2 and 3 assumed dry well/wet well configuration, however, other configurations with potential cost-savings include a wet well with submersible pumps and motors, or a wet well with vertical turbine solids handling pumps above grade. It is recommended that these options be evaluated further during the detailed design phase if this alternative is selected as the basis of design. **Table 2.13** lists the assumptions made to conduct a preliminary pump selection for the new IPS.

*Table 2.12 - IPS Alternative 2 - New IPS Firm Capacity*

| Flow Condition | PHF (MGD) | Ex. IPS Firm Capacity (MGD) | New IPS Firm Capacity (MGD)[1] |
|---|---|---|---|
| 2025 PHF | 22.5 | 15 | 7.5 |
| 2040 PHF | 28.3 | 15 | 13.3 |

*[1]The new IPS handles all flow in excess of the existing IPS's capacity under this alternative*

*Table 2.13 - Assumptions for New IPS Influent Pumps*

| # | Assumption[1] |
|---|---|
| 1 | A typical overall pump and motor efficiency of 60%. |
| 2 | A maximum of four duty pumps + one spare + space for one additional pump for future expansion. |
| 3 | A minimum of two duty pumps for the 2025 flow condition. Selecting a single duty pump to handle firm capacity would require a large pump which would be inefficient to ramp down for lower flows. |
| 4 | Two different motor sizes to provide operational flexibility. |
| 5 | Dry well/wet well configuration to match the existing IPS. |
| 6 | All pumps operating in parallel feeding a single discharge header. |

*[1]These assumptions apply to the new IPS for both Alternatives 2 and 3*

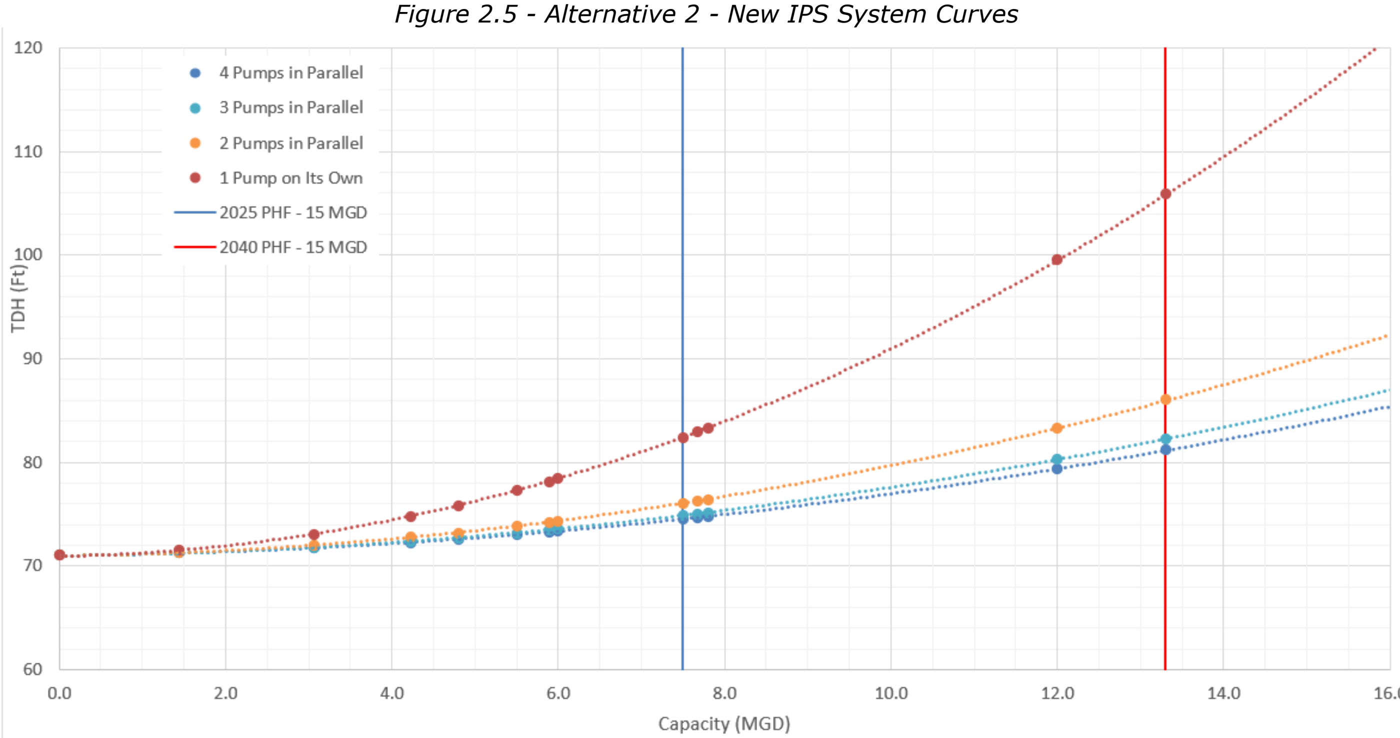
In addition to the proposed 24-inch FM from the existing IPS, it is recommended to install a second FM to provide needed capacity as well as flexibility and redundancy. Two parallel 24-inch force mains hydraulically connected by a valved interconnection would have capacity to handle the 2025 PHF of 22.5 MGD at less than 6 ft/sec and the 2040 PHF of 28.3 MGD at less than 7 ft/sec. This force main configuration was used to develop the system curves used to perform preliminary pump selection.

Using the assumptions listed in **Table 2.13** above, there are three possible pump configurations for the new IPS:

1. Two pumps in parallel for 2025 (three total), then three pumps in parallel for 2040 (four total).
2. Two pumps in parallel for 2025 (three total), then four pumps in parallel for 2040 (five total).
3. Three pumps in parallel for 2025 (four total), then four pumps in parallel for 2040 (five total).

System curves for each of these configurations were developed as shown in **Figure 2.5**. The 2025 and 2040 firm capacity flows are represented by the vertical blue and red lines, respectively. If Alternative 2 is selected, it is recommended that detailed pump evaluation and final selection be performed during the preliminary engineering design phase to optimize pumping efficiency. The conceptual cost opinion for alternative 2 was developed based on configuration number 2 listed above, utilizing two 79 hp duty pumps for the 2025 design condition, and four 79 hp duty pumps for the 2040 design condition.

*Figure 2.5 - Alternative 2 - New IPS System Curves*




Advantages of this alternative include:

- Utilizes existing influent pump station and wet well, saving cost and eliminating demolition
- Significantly lower capital cost than Alternative 3
- Does not require extension of utilities to new site
- Does not require extension of outfall sewer or in-plant sewer

Disadvantages of this alternative include:

- Configuration of hydraulically connected wet wells may not necessarily protect against FOG buildup
- Limited design flexibility due to space constraints and compatibility with existing equipment
- More challenging to spatially plan for expansion beyond 2040

The conceptual cost opinion for Alternative 2 is included below in **Table 2.14**. Note that this cost opinion does not include the cost of a new screening facility located immediately upstream of the new IPS as shown in **Figure 2.4**. Screening cost opinions are provided in the following sections of this technical memorandum.

*Table 2.14 - Estimated IPS Alternative 2 Cost Opinion*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $900,000 |
| 2 | Mechanical | $180,000 |
| 3 | Electrical | $180,000 |
| 4 | Instrumentation | $90,000 |
| 5 | Structural | $2,244,000 |
| 6 | Civil | $800,000 |
| 7 | Mobilization & Demobilization | $176,000 |
| 8 | Indirect Costs | $207,000 |
| 9 | General Conditions & Contractor Markup | $1,004,000 |
| 10 | 30% Contingency | $1,735,000 |
| 11 | Engineering, Legal, & Administration | $1,394,000 |
| | **Total Cost Opinion** | **$8,910,000** |

Annual O&M costs were estimated based on annual maintenance requirements of the equipment and the average annual energy usage. Annual equipment maintenance costs were assumed to be equal to 2% of the equipment capital cost, and the average electricity cost at the WWTF was assumed to be $0.06 per kWh. These assumptions shall remain the same for all other alternatives in this technical memorandum unless otherwise noted. **Table 2.15** and **Table 2.16** show the annual O&M costs for current and future production, respectively.

*Table 2.15 - IPS Alternative 2 - Annual O&M Costs - 2021*

| Item | Annual Cost |
|---|---|
| Maintenance | $18,000 |
| Electricity | $33,000 |
| **TOTAL** | **$51,000** |

*Table 2.16 - IPS Alternative 2 - Annual O&M Costs - 2040*

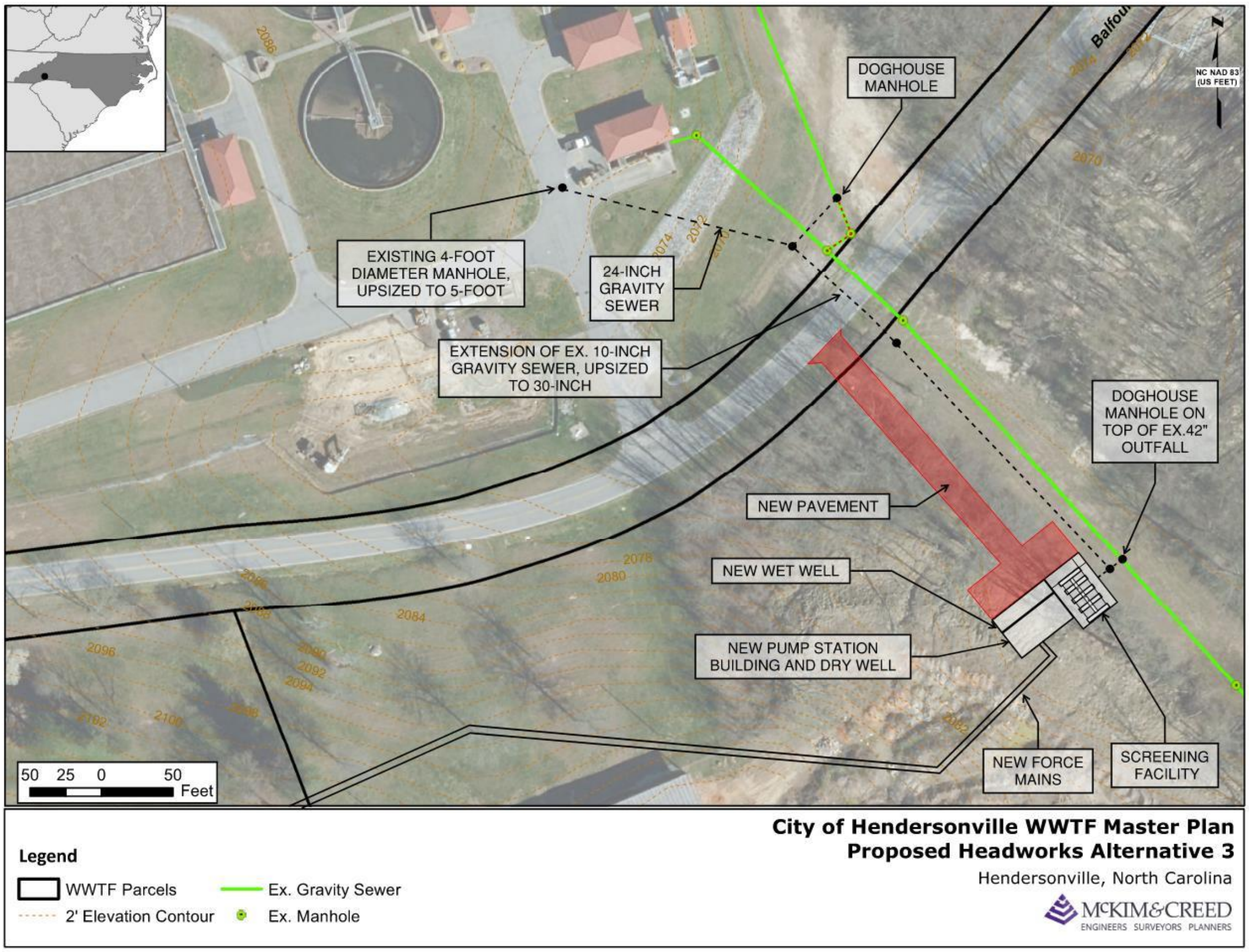
| Item | Annual Cost |
|---|---|
| Maintenance | $18,000 |
| Electricity | $34,000 |
| **TOTAL** | **$52,000** |

#### 2.2.2.3 *Alternative 3: New influent pump station to accommodate all future flows*

This alternative consists of constructing a new IPS south of Balfour Road, as shown in **Figure 2.6**, to accommodate all future AADFs and PHFs. This alternative would require clearing of a wooded area, significant earthwork, extending utilities (e.g., plant water, power, fiber optic, etc.), extending the existing

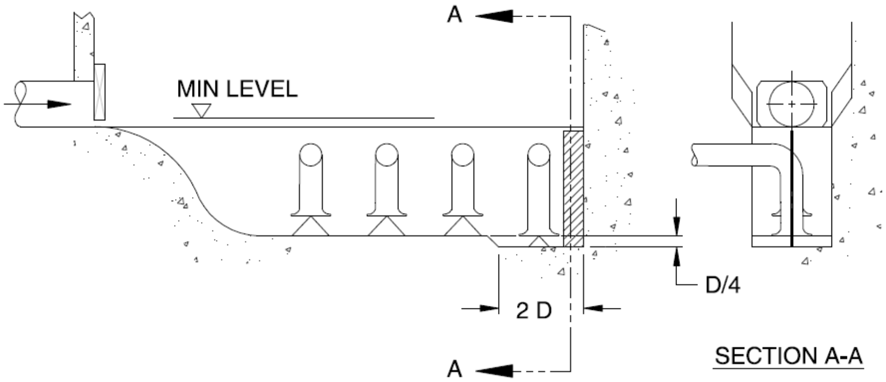
10-inch gravity sewer that flows from the north (which would include a trenchless crossing of Balfour Road), abandoning a portion the Mud Creek outfall sewer and rerouting its flow, extending the 24-inch in-plant gravity sewer as needed to flow to the new wet well, and demolishing the existing IPS. Although this alternative is the most expensive option, it also provides the most flexibility to ensure adequate hydraulic conditions, complete equipment compatibility, and space for future expansion beyond 2040.

*Figure 2.6 - Headworks Alternative 3 Proposed Layout*




As noted in Technical Memorandum No. 1, City staff have had consistent issues with FOG build-up in the existing rectangular wet well. Trench-type wet wells, as shown in **Figure 2.7**, create conducive hydraulic environments for pump intakes, minimize footprint size for wastewater wet wells, have a reduced floor area that minimizes solids accumulation, and are designed to clean the wet well by pumping it down. In trench-type wet wells, water flows down the ramp at a high velocity to the last pump which periodically pumps down the wet well water level to remove sludge and scum in a matter of minutes without manual labor. Trench-type wet wells have been empirically shown to be effective at reducing or eliminating FOG buildup in wastewater wet wells using this self-cleaning method and are suitable for design flows of 3 MGD or greater.

*Figure 2.7 - Typical trench-type wet well*




Unlike the new IPS in Alternative 2, the new IPS in Alternative 3 would handle all future flows including the 2025 and 2040 AADFs and PHFs. To achieve a desired firm capacity of 28.3 MGD (2040 PHF) with the new IPS, preliminary force main sizing and pump evaluation were conducted.

To size the pumps, first the number and size of force mains were selected to produce a system curve. As noted in **Table 2.3**, the desired minimum and maximum velocity in force mains are 2 ft/sec and 8 ft/sec, respectively, with the preferred maximum velocity at peak flows being 6 ft/sec. A single force main would not be able meet both minimum and maximum velocity requirements; therefore, it is recommended the new IPS feed two force mains, which also provides operational flexibility and redundancy. It is recommended that force mains be of equal diameter to minimize material costs and simplify hydraulic design. As noted previously, two parallel 24-inch force mains have capacity to handle the 2025 PHF of 22.5 MGD at less than 6 ft/sec and the 2040 PHF of 28.3 MGD at less than 7 ft/sec. The minimum velocity of 2 ft/sec will be achieved at the 2025 AADF of 4.23 MGD in a single 24-inch force main, which would be used to transport low flows. Two 24-inch force mains are recommended for Alternative 3, which provide capacity for future growth beyond the 2040 PHF up to 32.5 MGD combined.

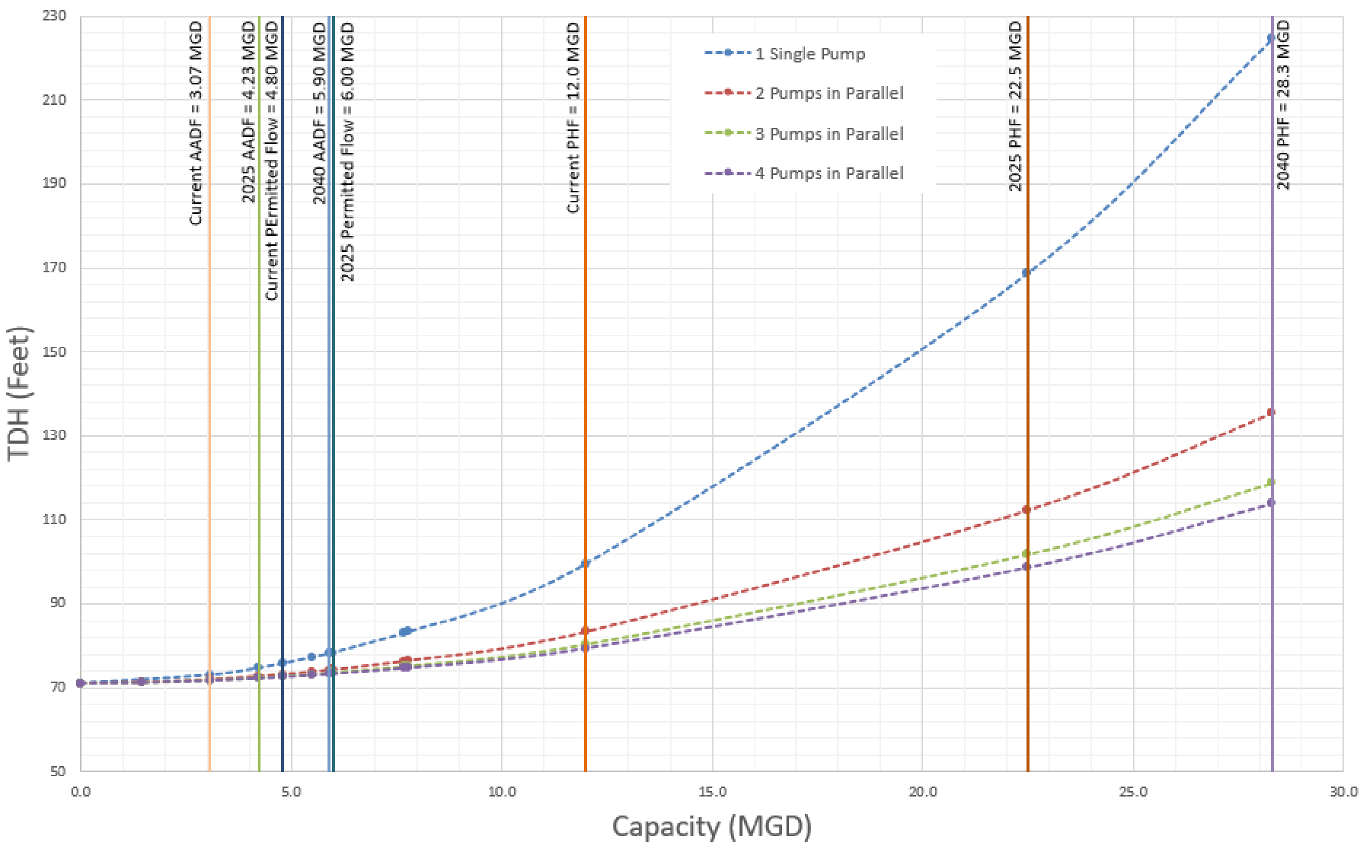
Using the assumptions listed in **Table 2.13** above, three possible pump configurations were considered for the new IPS under Alternative 3:

1. Two pumps in parallel for 2025 (three total), then three pumps in parallel for 2040 (four total).
2. Two pumps in parallel for 2025 (three total), then four pumps in parallel for 2040 (five total).

3. Three pumps in parallel for 2025 (four total), then four pumps in parallel for 2040 (five total).

The system curves for each of the pump configurations are shown in **Figure 2.8**. System curves were developed assuming that flow would be pumped to a single force main until it achieves 6 ft/sec, at which point the interconnecting valve between the two force mains would open through automated means to transport additional flow through both force mains. AADF, Permitted Flows, and PHFs for the current, 2025, and 2040 design conditions are represented by vertical lines. If Alternative 3 is selected, it is recommended that detailed pump evaluation and final selection be performed during the preliminary engineering design phase to optimize pumping efficiency. The conceptual cost opinion for alternative 3 was developed based on configuration number 3 listed above, utilizing three 201 hp pumps for the 2025 design condition, and four 201 hp pumps for the 2040 design condition. If this alternative is selected for detailed design, it is recommended that a thorough analysis of pumping conditions be completed, and the selection of multiple pump sizes be provided to ensure the pumps operate as close to their BEP as possible during the most frequent operating conditions.

*Figure 2.8 - IPS Alternative 3 System Curves*




Advantages of this alternative include:

- Highest degree of design flexibility of site layout and equipment compatibility

- Easier to plan for expansion beyond 2040
- Construction requires less coordination to maintain current influent pump station operation
- Enables construction of trench-type wet-well, which decreases maintenance and FOG accumulation issues

Disadvantages of this alternative include:

- Does not utilize existing influent pump station and wet well and involves their demolition
- Higher capital cost due to extension of existing outfall and in-plant gravity sewers, trenchless crossing of Balfour Road, extension of other utilities (natural gas, electricity, fiber optic, etc.), tree clearing, and significant site fill

The conceptual cost opinion for Alternative 3 is included below in **Table 2.17**.

*Table 2.17 - Estimated IPS Alternative 3 Cost Opinion*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $570,000 |
| 2 | Mechanical | $114,000 |
| 3 | Electrical | $114,000 |
| 4 | Instrumentation | $57,000 |
| 5 | Structural | $2,057,000 |
| 6 | Civil | $2,260,000 |
| 7 | Demo | $215,000 |
| 8 | Mobilization & Demobilization | $216,000 |
| 9 | Indirect Costs | $255,000 |
| 10 | General Conditions & Contractor Markup | $1,231,000 |
| 11 | 30% Contingency | $2,127,000 |
| 12 | Engineering, Legal, & Administration | $1,709,000 |
| | **Total Cost Opinion** | **$10,925,000** |

**Table 2.18** and **Table 2.19** show the annual O&M costs for current and future production, respectively.

*Table 2.18 -IPS Alternative 3 - Annual O&M Costs - 2021*

| Item | Annual Cost |
|---|---|
| Maintenance | $12,000 |
| Electricity | $67,000 |
| **TOTAL** | **$79,000** |

*Table 2.19 - IPS Alternative 3 - Annual O&M Costs - 2040*

| Item | Annual Cost |
|---|---|
| Maintenance | $12,000 |
| Electricity | $71,000 |
| **TOTAL** | **$83,000** |

## 2.3 Screening

### 2.3.1 Current Capacity Analysis

The existing IPS discharges influent flow via two 16-inch force mains into a common channel upstream of the two mechanical bar screens, which are situated in parallel channels upstream of the grit removal systems. The bar screens are operated by monitoring the water level upstream and downstream of the screen and engaging the rake arm operation when the differential in the water levels exceeds the set point.

The capacity of the existing bar screen systems is based on the velocity of the flow approaching and passing through the screens, head loss across the screen, and screening quantities collected. Capacity was evaluated assuming one screen out of service.

A minimum approach velocity of 1.3 ft/sec is recommended for flow entering bar screens to prevent solids depositions in the channel. However, achieving this velocity is not always feasible with diurnal fluctuations in flow. Flow velocities of at least 2.5 ft/sec during peak flows can resuspend solids, mitigating the issue of low flow velocity during AADF conditions. As shown below in **Table 2.20,** the current AADF approach velocity in the screening channel is below the recommended minimum 1.3 ft/sec when both screens are in service, but above it when only one screen is in service. At current peak flows, the approach velocity does not achieve 2.5 ft/sec with one or two screens in service, indicating a potential for solids buildup in the screening channel at current design conditions.

A maximum velocity of 4 ft/sec through the openings is recommended for peak hour flow to prevent debris pass-through. At current AADF, velocity through the screen exceeds the maximum of 4 ft/sec with one or two screens in service, as shown in **Table 2.20**. A tradeoff exists between achieving minimum approach velocity of 1.3 ft/sec at AADF and staying under a maximum pass-through velocity of 4 ft/sec at PHF. Increasing the approach velocity through channel configuration adjustments increases the likelihood of exceeding the maximum pass-through velocity. Hydraulic control structures can be installed downstream of the screens to control water level in the channel at peak flow, but additional head loss and potential for scum buildup and removal must be considered in the design of such structures.

*Table 2.20 – Ex. Screening Flow Velocities*

| Flow Description | Screens in Service | Flow (MGD) | Approach Velocity (ft/sec) | Velocity Through Screen (ft/sec) |
|---|---|---|---|---|
| Current AADF | 1 | 4.8 | 1.60 | 6.31 |
| Current AADF | 2 | 4.8 | 1.18 | 4.63 |
| Current PHF | 1 | 12.0 | 2.06 | 8.11 |
| Current PHF | 2 | 12.0 | 1.95 | 7.7 |

Approach and pass-through velocities were evaluated assuming the addition of a downstream hydraulic control structure that raised the water level downstream of the screens to 3 feet. The effect was a significant lowering of both approach and pass-through velocity as shown in **Table 2.21**, however, neither of the velocity types fall within the desired range. The inability of the existing screens to meet velocity criteria even with channel modifications indicates the necessity for expansion.

*Table 2.21 - Ex. Screening Flow Velocities Assuming Added Downstream Hydraulic Control Structure*

| Flow Description | Screens in Service | Flow (MGD) | Approach Velocity (ft/sec) | Velocity Through Screen (ft/sec) |
|---|---|---|---|---|
| Current AADF | 1 | 4.8 | 0.61 | 2.42 |
| Current AADF | 2 | 4.8 | 0.32 | 1.25 |
| Current PHF | 1 | 12.0 | 2.06 | 8.11 |
| Current PHF | 2 | 12.0 | 1.27 | 5.02 |

Head loss through the screens should be controlled to avoid the undesirable submersion of upstream elements of the WWTF. Just upstream of the screen, the existing screening channel has a 3-foot long by 8-inches tall overflow weir that spills into the aeration basin, allowing excess flow to bypass the screening and grit removal processes. Assuming one screen is out of service and the remaining screen is 50% blinded/clogged, there remains approximately 0.2 feet of freeboard below the weir at the projected 2025 PHF flow. Due to the velocity concerns noted above, it is assumed expansion of the screens will occur prior to the 2040 PHF, eliminating corresponding head loss concerns associated with the existing screens.

The key limited capacity indicator for the quantity of screenings removed is the rated capacity of the washer/compactor equipment. The existing washer/compactor is a JDV Equipment Corporation Shaftless Screw Conveyor/Compactor with a rated capacity of 70 $ft^3$/hr at a constant speed of 23 RPM. No screening quantity data for the WWTF has been collected, however, average screening removal quantities have been developed based on bar spacing size using empirical data. Average screening quantities for bar screening with 3/8” spacing (the same as the City’s WWTF) are listed below in **Table 2.22**.

*Table 2.22 - Average Wet Screenings Collection for 3/8" Bar Screen*

| Description | Average Wet Screenings Collected ($ft^3$/MG) |
|---|---|
| Lower Limit | 4.5 |
| Average | 7.0 |
| Upper Limit | 11.3 |

**Table 2.23** shows the volume of screenings captured per hour assuming the screen capture rate upper limit of 11.3 $ft^3$/MG, indicating there are no capacity concerns for the screenings screw conveyor compactor.

*Table 2.23 - Screening Collection Quantities at Peak Flows*

| Flow Description | Flow (MGD) | Screenings collected (in $ft^3/hr$) | Exceeds Capacity of Existing Screw Auger? |
|---|---|---|---|
| 2020 PHF | 17.4 | 8.2 | No |
| 2025 PHF (non-equalized) | 22.5 | 10.6 | No |
| 2040 PHF (non-equalized) | 28.3 | 15.9 | No |

In conclusion, the high flow-through velocity at current PHF cannot be completely addressed by simple hydraulic control structure modifications to the existing channels, indicating that existing debris-pass-through concerns will only increase as PHFs increase. Therefore, it is recommended that the screenings process at the WWTF be expanded to accommodate the 2025 and 2040 flows.

### 2.3.2 Alternatives Evaluation

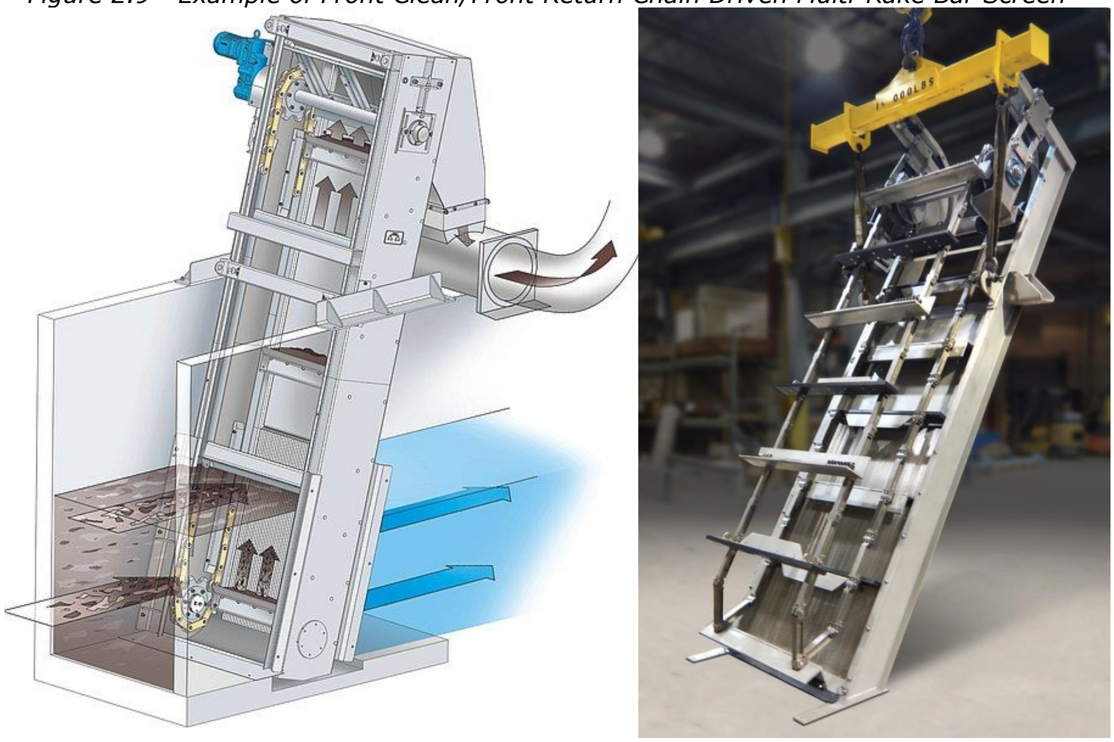
Coarse screens retain debris > 6 mm (0.25 in) such as rocks, branches, and rags, and are sufficient to protect pumps, valves, pipelines, and other equipment from clogging or damage. Fine screens retain debris from 0.5 mm to 6 mm, capturing the same debris as coarse screens and much more, including organic material that must be returned to the flow stream. The process units downstream of the screens at the WWTF evaluated in this master plan do not require fine screening for protection, so only course screens were considered. It is recommended that bar screen spacing is 6 mm (0.25 in) to maximize protection of downstream equipment. A smaller spacing between bars increases the importance of screenings washer/compactor equipment to return organics to the flow stream.

There are four (4) main types of mechanically-cleaned coarse screening systems; catenary, continuous belt, reciprocating rake, and chain-driven (multi-rake). The catenary screen - a specialized version of chain-driven screens - has a comparably large footprint, making it a complicated and uneconomic retrofit of the existing screening process. Although a continuous belt system would allow most maintenance to be performed above-deck (similar to the current screening units), excessive cost would be sunk into the extra screen material needed between the operating deck and the channel. Reciprocating rake screens, which imitate the process of a person raking a screen, have only a single rake, lengthening the cycle time and resulting in insufficient capacity for the anticipated peak debris loads. For the reasons listed above, these three types of screening systems were not considered for this master plan.

The fourth type of mechanically-cleaned bar screens is the chain-driven or multi-rake system, which is further subcategorized by whether the bar is raked on the upstream or downstream side of the bar, and whether the rakes return to the bottom of the bar screen from the front or back. These screen systems are compared in **Table 2.24**, with an example of a front clean/front return screen illustrated in **Figure** .

For the purposes of this master plan, front clean/front return screens were evaluated for all alternatives, however, it is recommended that final equipment selection should be made in a more detailed preliminary engineering analysis.

*Figure 2.9 - Example of Front Clean/Front Return Chain Driven Multi-Rake Bar Screen[1]*



*[1]Figure on the left shows a screen with a bottom sprocket permanently submerged in the channel (image from Huber). Figure on the right shows a screen with no submerged sprocket that instead has a specially designed chain that acts as the bottom sprocket (image from Parkson).*

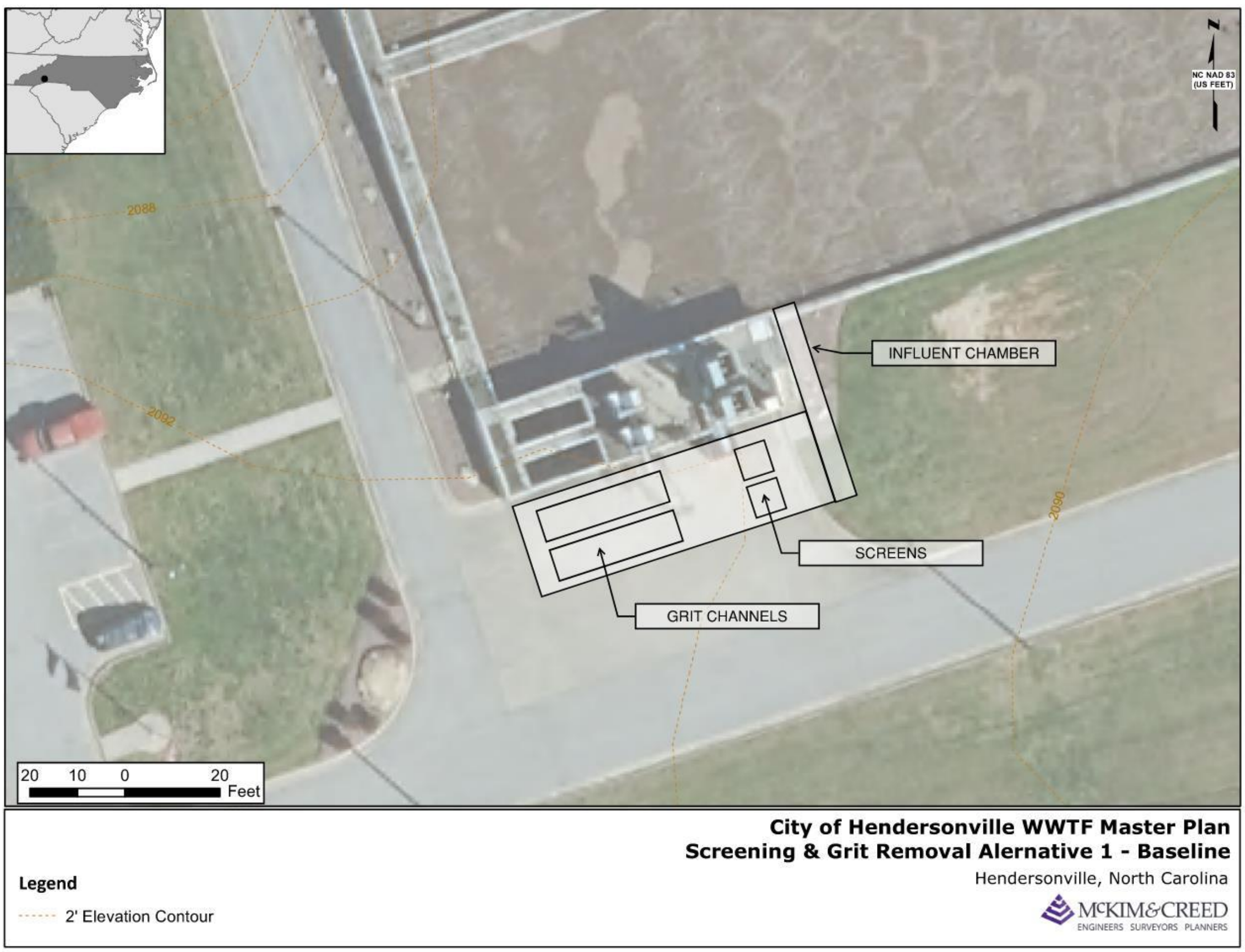
*Table 2.24 - Advantages & Disadvantages of Chain-Driven Multi-Rake Screens*

| Chain-Driven (Multi-Rake) Screens | Advantages | Disadvantages |
|---|---|---|
| Front clean/ back return | • Used for heavy duty applications | • Less efficient screenings removal<br>• Hinged plate required to seal the pocket under the screen is subject to jamming |
| Front clean/ front return | • Very little screenings carryover<br>• Systems available that do not have continuously submerged moving parts | • Submerged moving parts (chains, sprockets, and shafts) are subject to fouling<br>• Heavy objects, or solids building at the base of the bar screen may cause rake to jam |
| Back clean/ back return | • Submerged moving parts (chains, sprockets, and shafts) are protected by the bar rack | • Long rake teeth are susceptible to breakage<br>• Some susceptibility to screenings carryover as rake tines wear out<br>• Bar rack is less rugged since top of rack is unsupported to allow passage of rake tines |

#### 2.3.2.1 *Alternative 1 – Expand Existing Screening Channels*

This alternative maintains the use of the existing screening channels and requires the construction of new channels for additional screens and aerated grit chambers parallel to the existing channels as shown in **Figure 2.10**. The existing reciprocating rake screens would be replaced with two chain-driven multi-rake screens to shorten the cycle time and ensure sufficient capacity for the anticipated peak debris loads. Additional chain-driven multi-rake screens would be added in the adjacent channels as needed to meet capacity requirements. All new screens would have ¼-inch (6 mm) bar spacing as opposed to the existing 3/8-inch (9.5 mm) spacing to better protect downstream equipment. The screening inlet channel is recommended to be expanded to provide upward flow from the influent force mains which would improve hydraulic distribution to each screening channel and reduce screenings pass-through caused by the momentum of flow exiting the existing influent force mains horizontally.

*Figure 2.10 - Screening & Grit Removal Alternative 1 - Baseline Proposed Layout*



Advantages of this alternative include:

- Maximizes use of the existing screening infrastructure
- Improves screenings capture efficiency due to smaller bar spacing and increased rake capacity
- Low head loss
- Simple construction with minimal earthwork
- Lowest capital cost

Disadvantages of this alternative include:

- Does not provide screening ahead of the influent pump station as recommended in Technical Memorandum No. 1
- Existing location within the facility is not ideal for implementation of future flow equalization facilities to provide screening prior to EQ storage
- Geotechnical conditions at the existing site may require the use of timber piles for the expanded structure's foundation

The conceptual cost opinion for Alternative 1 is included below in **Table 2.25.** The cost opinion for screening Alternative 1 includes the conceptual cost opinion for grit removal Alternative 1 which consists of rehabilitating the existing aerated grit chambers and expanding grit removal with two additional aerated grit chambers constructed adjacent to the existing. These cost opinions are presented together since the screening and grit removal structures are connected and would be constructed at the same time if implemented. Additional information regarding grit removal Alternative 1 is presented in **Section 2.4.2.1**.

*Table 2.25 – Estimated Screening Alternative 1 Cost Opinion*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $1,508,000 |
| 2 | Mechanical | $302,000 |
| 3 | Electrical | $302,000 |
| 4 | Instrumentation | $151,000 |
| 5 | Structural | $283,000 |
| 6 | Civil | $247,000 |
| 7 | Demo | $30,000 |
| 8 | Mobilization & Demobilization | $113,000 |
| 9 | Indirect Costs | $134,000 |
| 10 | General Conditions & Contractor Markup | $646,000 |
| 11 | 30% Contingency | $1,115,000 |
| 12 | Engineering, Legal, & Administration | $896,000 |
| | **Total Cost Opinion** | **$5,727,000** |

Estimated annual O&M costs for Alternative 1 are summarized in **Table 2.26** and **Table 2.27** below for current and future design conditions. The O&M cost estimates presented below assume that two duty screens would be required for the current condition, and three duty screens would be required for the 2040 design conditions. O&M costs for current and future conditions will be nearly equivalent due to the low energy requirements of the proposed screening equipment. Energy usage at future conditions will be higher than current conditions since three operating screens will be required rather than two. However, energy usage will still be minimal, and rounding assumptions used for conceptual cost estimates results in current and future electricity costs being nearly equivalent.

*Table 2.26 Screening Alternative 1 - Annual O&M Costs - 2021*

| Item | Annual Cost |
|---|---|
| Maintenance | $18,000 |
| Electricity | $1,000 |
| **TOTAL** | **$19,000** |

*Table 2.27 Screening Alternative 1 - Annual O&M Costs - 2040*

| Item | Annual Cost |
|---|---|
| Maintenance | $18,000 |
| Electricity | $1,000 |
| **TOTAL** | **$19,000** |

#### 2.3.2.2 *Alternative 2 – Construct New Screening Facility Upstream of Expanded IPS*

This alternative proposes expanding the IPS as described in IPS Alternative 2 above and installing a new screening facility upstream of the expanded pump station. The new screening facility would require fill to elevate the finished grade above the FEMA 100-year floodplain elevation of 2,076.4 feet and would need to be located outside of the 50 foot setback from the property line adjoining Balfour Road. The proposed screening facility and new IPS are shown in **Figure 2.4.** Placing the screens upstream of the IPS and other WWTF process units is highly recommended and standard practice in design of most municipal WWTFs. Screens upstream of the IPS reduce damage, wear, ragging, maintenance, and repair of influent pumps and other downstream equipment. The proposed screens placement also eliminates the need for the existing trash rack and the associated manual labor and risk required to clean it.

This configuration would require rerouting all influent flow from the outfall through the screening facility before entering the IPS wet well. Construction would involve maintaining continuous flow to the IPS and the demolition of the existing screening and grit removal channels.

Preliminary screen sizing was conducted for the 2025 and 2040 AADF and PHF conditions to determine the number of screens required based on screen channel approach velocity, velocity passing through the bars, and head loss across the screen.

Design assumptions include a channel width of 4 feet, bar spacing of 6 mm (0.25 in), bar width of 0.25 inches, minimum head loss of 6 inches at AADF, and a maximum head loss of 24 inches at PHF. As previously discussed, a minimum approach velocity of 1.3 ft/sec is recommended at AADF, and a maximum velocity of 4 ft/sec through the bar screen openings is recommended at PHF. A new screening facility upstream of the IPS would have a screening channel depth of approximately 26.5 ft. This depth offers a large degree of design flexibility as flow depths can be varied significantly to meet design criteria. **Table 2.28** lists the velocities, head losses, and channel water levels for multiple screen configurations under these assumptions for the 2025 and 2040 design flows.

*Table 2.28 - Screening Alternative 2: Design Criteria*

| Design Year | Flow Type | Screens in Service | Flow (MGD) | Approach Velocity (ft/sec) | Velocity Through Screen (ft/sec) | Downstream Water Level (ft) | Head Loss Across Screen (ft) | Upstream Water Level (ft) |
|---|---|---|---|---|---|---|---|---|
| 2025 | AADF | 1 | 4.23 | 0.8 | 3.0 | 2.0 | 0.2 | 2.2 |
| 2025 | AADF | 2 | 4.23 | 0.7 | 2.8 | 1.0 | 0.2 | 1.2 |
| 2025 | AADF | 3 | 4.23 | 0.5 | 2.0 | 1.0 | 0.1 | 1.1 |
| 2040 | AADF | 1 | 5.90 | 1.0 | 3.9 | 2.0 | 0.3 | 2.3 |
| 2040 | AADF | 2 | 5.90 | 0.9 | 3.6 | 1.0 | 0.3 | 1.3 |
| 2040 | AADF | 3 | 5.90 | 0.7 | 2.6 | 1.0 | 0.1 | 1.1 |
| 2025 | PHF | 1 | 22.5 | 0.9 | 3.7 | 9.0 | 0.3 | 9.3 |
| 2025 | PHF | 2 | 22.5 | 1.0 | 4.0 | 4.0 | 0.3 | 4.3 |
| 2025 | PHF | 3 | 22.5 | 0.9 | 3.5 | 3.0 | 0.3 | 3.3 |
| 2040 | PHF | 1 | 28.3 | 0.3 | 3.8 | 11.0 | 0.3 | 11.3 |
| 2040 | PHF | 2 | 28.3 | 0.9 | 3.5 | 6.0 | 0.2 | 6.2 |
| 2040 | PHF | 3 | 28.3 | 0.9 | 3.4 | 4.0 | 0.2 | 4.2 |

The approach velocity in the screen channel does not achieve the minimum recommended velocity of 1.3 ft/sec to resuspend solids at the 2025 or 2040 AADFs or PHFs with any of the configurations listed in **Table 2.28** above. A tradeoff exists between minimum approach velocity and maximum flow-through velocity. Achieving the former may result in exceeding the latter. Solids accumulation upstream of the screens can be addressed by an installed diffused air system to provide solids buoyancy. The consequences of debris pass-through are more significant and far-reaching than solids accumulation upstream of screen, thus, velocity through the screen is considered the limiting velocity design criteria.

It is recommended to limit surcharging of the existing 42-inch diameter Mud Creek outfall during peak flows to prevent sanitary sewer overflows (SSOs) in the collection system as much as possible. Assuming that the bottom elevation of the proposed screen channel under this Alternative 2 is at the same elevation as invert of the 42-inch outfall, two duty screens at the 2025 PHF and three duty screens at the 2040 PHF will limit the surcharging of the 42-inch outfall to less than 1-foot. To reduce the number of duty screens to one for the 2025 PHF and two for the 2040 PHF and to maintain the same level of surcharge, the screen channels would need be approximately 5 feet deeper. This additional excavation and concrete would add significant capital costs to the project. It is recommended that three identical screens be installed for the 2025 design conditions, two duty and one standby, and a fourth screen be added for the 2040 design condition. It is recommended that the final number, type and size of screens be verified during the detailed design phase.

Advantages of this alternative include:

- Prevents excessive wear and ragging of influent pumps
- Significantly higher screenings discharge capacity
- Low head loss
- Provides room for expansion for 2040 and beyond
- Lower capital cost, smaller footprint, and shorter construction schedule than Alternative 3
- Existing influent pumps and screening system can remain in place and in operation during construction
- Eliminates the existing manual bar rack and keeps operating staff out of below grade channels at the influent pump station as much as possible

Disadvantages of this alternative include:

- Additional maintenance for screening channel isolation gates
- Limited construction room and site layout flexibility due proximity to floodway and Balfour Road
- Requires a significant amount of fill to raise screening structure top slab elevation out of floodplain
- High capital cost

The conceptual cost opinion for Alternative 2 is included below in **Table 2.29.**

*Table 2.29 Estimated Screening Alternative 2 Capital Costs*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $896,000 |
| 2 | Mechanical | $180,000 |
| 3 | Electrical | $180,000 |
| 4 | Instrumentation | $90,000 |
| 5 | Structural | $1,328,000 |
| 6 | Civil | $50,000 |
| 7 | Mobilization & Demobilization | $109,000 |
| 8 | Indirect Costs | $129,000 |
| 9 | General Conditions & Contractor Markup | $623,000 |
| 10 | 30% Contingency | $1,076,000 |
| 11 | Engineering, Legal, & Administration | $864,000 |
| | **Total Cost Opinion** | **$5,525,000** |

**Table 2.30** and **Table 2.31** show the annual O&M costs for current and future production, respectively.

*Table 2.30 Screening Alternative 2 - Annual O&M Costs - 2021*

| Item | Annual Cost |
|---|---|
| Maintenance | $18,000 |
| Electricity | $1,000 |
| **TOTAL** | **$19,000** |

*Table 2.31 Screening Alternative 2 - Annual O&M Costs - 2040*

| Item | Annual Cost |
|---|---|
| Maintenance | $18,000 |
| Electricity | $1,000 |
| **TOTAL** | **$19,000** |

#### 2.3.2.3 *Alternative 3 – Construct New Screening Facility Upstream of New IPS*

This alternative includes the construction of a new screening facility south of Balfour Road and west of the existing 42-inch outfall as shown previously in **Figure 2.6**. The screening facility would be upstream of and adjacent to a new IPS also located south of Balfour Road and would require extending the 10-inch outfall from the north side of the WWTF and the 24-inch WWTF in-plant gravity sewer that currently flows into the west side of the existing wet well. These sewer lines would converge in a manhole just north of Balfour Road and combine into a common 30-inch sewer that would extend to the new screening facility via trenchless crossing of Balfour Road as shown in **Figure 2.6** above. The existing 42-inch outfall would be redirected to the new screening facility and abandoned in-place along the existing alignment downstream of the new connection. Flow will pass through the screens to the new IPS wet well by gravity and be pumped to grit removal and the equalization basin at the old plant site. Once the new screening facility, IPS, and grit removal facility are operational, construction would include demolition of the existing IPS and screening and grit removal channels.

The elevation of the existing outfall inverts dictates the required depth of the new screening facility, and the floodplain elevation dictates its finished grade, amount of fill, and overall structure height. These elevations are very similar between screening Alternative 2 and 3; therefore, the preliminary screening selection conducted for Alternative 2 also holds true for Alternative 3. It is therefore recommended that two duty screens and one spare be installed for the 2025 PHF condition and a third duty screen be added for the 2040 PHF condition with the qualifier that additional channel depth may reduce the number of channels needed, and that final screen sizing and selection should be performed in the detailed design phase.

Advantages of this alternative include:

- Prevents excessive wear and ragging of influent pumps
- Significantly high screenings discharge capacity
- Low head loss
- Provides room for expansion for 2040 and beyond
- Eliminates the existing manual bar rack and keeps operating staff out of below grade channels at the influent pump station as much as possible
- High degree of site layout and design flexibility
- Existing influent pumps and screening system can remain in place and in operation during construction

Disadvantages of this alternative include:

- Additional maintenance for screening channel isolation gates
- Higher capital cost, larger footprint, and longer construction schedule than Alternative 2
- Greenfield site will require tree clearing, potential environmental impact study, and extension of utilities (e.g., gas, electricity, plant water, etc.)
- Additional permitting requirements to cross NCDOT road
- Requires a significant amount of fill to raise screening structure top slab elevation out of floodplain

Cost opinions and annual O&M costs for screening alternative 3 are equivalent to screening alternative 2 since the layout and equipment required for both alternatives are the same. Grading costs associated with each site are captured in the cost estimates for the influent pumping station alternatives. Therefore the conceptual cost opinion and annual O&M costs for alternative 2, shown in **Table 2.29**, **Table 2.30**, and **Table 2.31** apply to alternative 3.

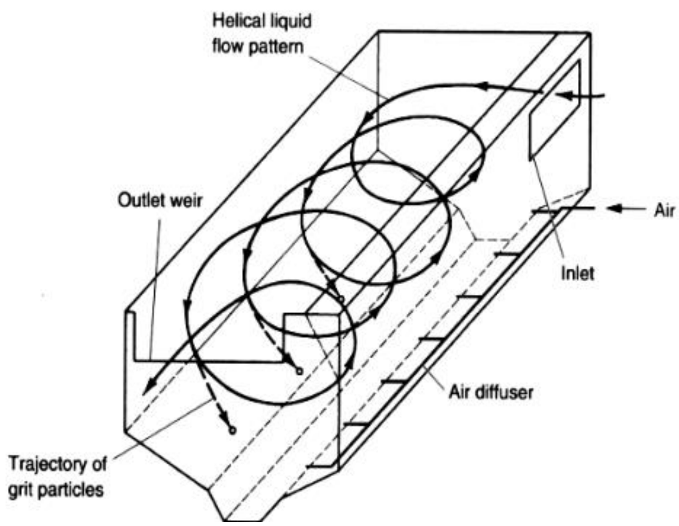
## 2.4 Grit Removal

### 2.4.1 Current Capacity Analysis

The capacity of the existing aerated grit removal system is based on the detention time, channel length-to-width ratio, grit removal quantities, and air volume. Capacity was evaluated assuming one of the grit removal channels is out of service.

Aerated grit chambers operate by introducing air along one side of a rectangle tank to create a spiral flow pattern perpendicular to the flow through the tank as shown in **Figure 2.11**. Heavier grit particles settle to the bottom of tank while lighter, mostly organic particles remain in suspension and pass through the tank.

*Figure 2.11 - Aerated Grit Chamber Flow Schematic*




It was noted in Technical Memorandum No. 1 that significant grit buildup was observed in the aeration basins when taken offline for diffuser replacement in March of 2020. Grit carryover from aerated grit chambers results from the spiral velocity being too high for the larger grit particles to settle out. There are several possible causes of poor performance in aerated grit chambers, the most common being improper chamber geometry. Some other causes include improper baffling, inconsistent air supply to diffusers resulting in irregular roll patterns, and improper diffuser location.

It is recommended that aerated grit chambers have a minimum detention time of 3 minutes at PHF. If the detention time drops below 3 minutes, the flow velocity will be too high, and grit may be carried out of the chamber and into the downstream processes. Assuming a detention time of 3 minutes, one of the existing aerated grit chambers (Volume = 2,464 $ft^3$) has the capacity for a peak hourly flow of 8.85 MGD. Per Technical Memorandum No. 3, it is recommended that grit removal be located upstream of the EQ basin and therefore must be sized to handle the non-equalized flows shown in **Table 2.32**. The capacity of the grit removal process at the WWTF is based on the peak hour flow capacity with both grit removal units in service. Grit removal is not defined as an "essential treatment unit" per 15A NCAC 02T .0103(16), therefore only dual components are required to ensure reliability. Based on this, the existing grit chambers are adequately sized up to a PHF of 17.7 MGD to maintain a minimum 3 minute detention time. However, while not required, it is preferred that a redundant unit be provided if feasible.

*Table 2.32 - Non-equalized Flow Projections*

| Year | 2-Year Storm Peak Flow – Non-equalized (MGD) |
|---|---|
| Base (2017)* | 17.4 |
| 2025 | 22.5 |
| 2040 | 28.3 |

**Base year established by SSAIA Master Plan report*

It is recommended that aerated grit chambers have length-to-width (L:W) ratios between 3:1 and 8:1. The existing grit removal chambers at the WWTF are 32'-0" long and 7'-0" wide, resulting in an acceptable L:W ratio of 4.6:1. The effective length to width ratio can be adjusted by baffles as needed. The recommended width to depth ratio is 1:1 to 1.5:1. The existing aerated grit chambers have a depth of 11 feet based on static liquid level, resulting in a W:D ratio of 0.64. This is less than the typical range for aerated grit chambers and may cause an irregular roll pattern, resulting in less efficient grit capture.

The capacity of the grit removal process can also be limited by the capacity of the grit conveyor equipment, which removes the grit from the basin, dewaters, and transports it to a bin for disposal. The existing grit conveyor is a JDV Equipment Corporation Shaftless Screw Grit Conveyor with a rated capacity of 35 $ft^3$/hr at a constant speed of 4.9 RPM. No grit quantity data for the WWTF has been collected, however, industry texts such as Metcalf & Eddy and WEF MOP 8 have documented average grit removal quantities at domestic wastewater treatment facilities using empirical data. Grit quantities removed by aerated grit chambers typically range from 0.5 to 20 $ft^3$/MG, with an average of 5 $ft^3$/MG as reported by WEF MOP 8.

*Table 2.33 - Grit Removal Quantities*

| Flow Description | Flow (MGD) | Grit Removed (in $ft^3$/hr) | Exceeds Capacity of Existing Screw Auger? |
|---|---|---|---|
| 2020 PHF (non-equalized) | 17.4 | 3.6 | No |
| 2025 PHF (non-equalized) | 22.5 | 4.7 | No |
| 2040 PHF (non-equalized) | 28.3 | 5.9 | No |

**Table 2.33** above shows the projected volume of grit removed per hour assuming the average grit removal rate of 5.0 $ft^3$/MG, indicating there are no capacity concerns for the existing grit removal screw conveyor for future design conditions.

It is recommended that the air supply per unit length for aerated grit chambers is between 3 and 8 CFM per foot of chamber length. This corresponds to a range of 96 to 256 CFM/ft for one of the WWTF's 32-foot long aerated grit chambers. Currently, there are three (3) existing blowers dedicated to both aeration basins and both aerated grit chambers. The aerated grit chambers are currently served by a 6-inch air header that extends off the 12-inch air header that serves all the air diffusers in Aeration Basin No. 1. The 6-inch air header splits into two branches, one for each grit removal chamber, each with a butterfly valve upstream of the air header that drops to the 4-inch diffuser drop leg and coarse bubble diffuser headers in the bottom of the aerated grit chambers. This arrangement provides little control over the actual air flowrate delivered to each aeration basin, which is suspected to be a primary cause for potential poor performance.

The only point of air flow measurement is a venturi meter on the 12-inch aeration header in Aeration Basin No. 1 that is connected to a differential pressure transmitter to allow air flow monitoring via SCADA. This arrangement precludes the ability to accurately monitor and optimize air flow to the grit removal chambers. It is recommended that one or more air flow meters and automated control valves be implemented to improve process monitoring and control if the existing aerated grit chambers are to remain.

In summary, the existing aerated grit chambers are undersized for future peak hour flows based on detention time and are exhibiting grit carryover performance issues likely related to insufficient process control and a less than ideal width-to-depth ratio. The length-to-width ratio of the chambers is satisfactory and the existing grit collectors and screw conveyor have sufficient capacity for future flows, however, improvements to process air control are required. It is recommended that the grit removal capacity at the WWTF be increased in the near term and improvements be made to improve process air control.

### 2.4.2 Alternatives Evaluation

The three most common types of grit removal processes are: 1) horizontal flow grit chambers, 2) aerated grit chambers, and 3) vortex grit chambers. Horizontal flow grit chambers have a relatively large footprint, exert excessive wear on chains, flights, and bearings, and make maintaining target velocity difficult which can lead to removing large amounts of organic material from the flow stream. For these reasons and others, horizontal flow grit chambers have fallen out of favor in new installations compared to the aerated and vortex grit chamber options.

Aerated grit chambers are currently used at the WWTF and the City's staff is familiar with their operation. Although performance issues with the existing aerated grit chambers have been previously identified and described, adjustments to the existing chambers and proper design of additional aerated grit removal chambers have potential to mitigate poor performance issues.

Vortex grit chambers on the other hand are an entirely different grit removal technology, with three primary subcategories: 1) mechanically induced, 2) hydraulically induced, and 3) stacked-tray. The first two subcategories of vortex grit chambers are classified by whether the vortex is created by mechanical means (e.g., paddles/impellers) or by the force of the incoming flow. Hydraulically induced vortex grit removal units handle up to 8 MGD per unit and require pumped influent at a controlled rate and velocity. Although grit removal is proposed to be immediately downstream of the influent pumps, maintaining a controlled flow rate and velocity at the grit removal inlet would require additional pumps downstream of the influent pump force mains.

Mechanically induced and stacked-tray grit removal systems are the most common vortex grit removal systems installed at WWTF's throughout the southeast US. They have the common advantages of having a small footprint, large capacity, high removal efficiency at a wide range of flows, simple operation, and no submerged moving parts requiring maintenance.

For the reasons described above, horizontal flow grit chambers and hydraulically induced vortex grit removal chambers are not considered in this evaluation. The three alternatives evaluated to address the grit removal capacity and performance concerns at the WWTF include 1) expanding the existing aerated chain and bucket grit chambers, 2) constructing a new stacked tray vortex grit removal system, and 3) constructing a new mechanically induced vortex grit removal system.

#### 2.4.2.1 *Alternative 1 – Expand Existing Aerated Chain and Bucket Grit Chambers*

Aerated grit chambers have a long history of use at major WWTFs with well-established design criteria. Operation of these chambers was described above in the Current Capacity Analysis section. Expansion of the existing chambers would involve the construction of new chambers adjacent to the existing ones as shown in **Figure 2.10** above and would require the replacement of the following equipment in the existing grit chambers due to extended use and wear:

- Entire chain and bucket mechanisms including wear shoes, shafts, chains, buckets, etc.
- Fiberglass baffles
- Air headers and diffusers
- Drive mechanisms and shaftless screw conveyors

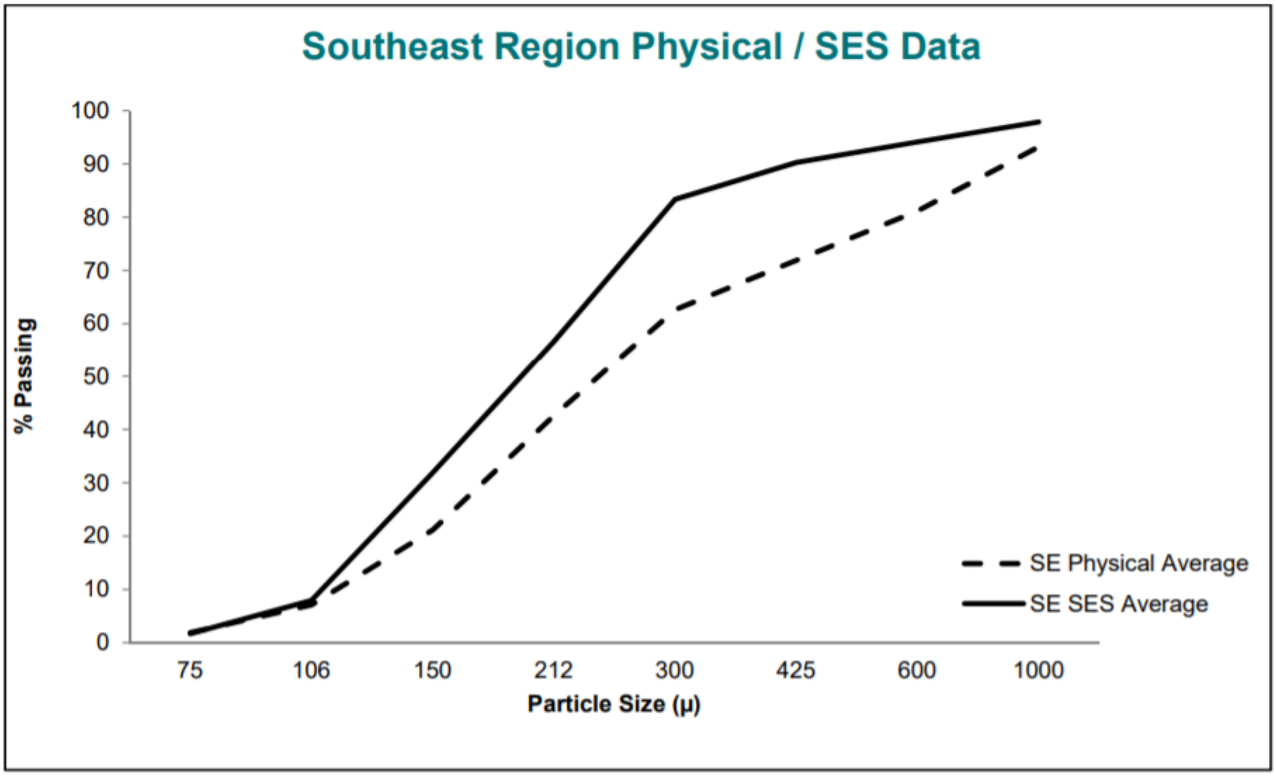
As discussed in the current capacity analysis section, the existing grit removal chambers at the WWTF are experiencing grit carryover likely due to improper chamber geometry and limited control of air supplied to the process. Assuming the new grit chambers are of equal size (volume = 2,464 $ft^3$) to the existing chambers, **Table 2.34** shows that a third grit chamber will be required to accommodate the 2025 PHF and a fourth grit chamber will need to be added for the 2040 PHF.

*Table 2.34 - Aerated Grit Removal Chamber Expansion Criteria*

| Parameter | Flow (MGD) | Design Year | |
|---|---|---|---|
| | | 2025 | 2040 |
| Peak hour flow, un-equalized | MGD | 22.5 | 28.3 |
| Peak hour flow, un-equalized | $ft^3$/sec | 34.8 | 43.8 |
| Minimum Detention time | minutes | 3.0 | 3.0 |
| Grit Removal Chamber Volume | $ft^3$ | 2,464 | 2,464 |
| Number of Duty Grit Removal Chambers Required | - | 3 | 4 |

Aerated grit chambers are typically designed to remove 210 micron grit and can potentially remove nearly 100% of design grit with proper adjustment. Based on the Southeast USA Regional Grit Gradation Data shown in **Figure 2.12**, the Sand Equivalent Size (SES) of 210 micron physical particle size is approximately 170 microns. Based on this regional data, approximately 40% of all grit in the influent wastewater has the potential to be carried over into downstream processes. For example, to achieve roughly 90% grit removal in the Southeast, the design grit particle size should be 106 microns. It should be noted that actual grit gradation data can vary widely throughout the southeast. Detailed design of grit removal improvements should consider implementation of a grit characterization study to select an appropriate design removal target.

*Figure 2.12 - Southeast Region Grit Gradation Data*[1]




[1]Hydro International. (2016, October 3). *Southeast USA Regional Grit Gradation Data.* Advanced Grit Management. https://www.advancedgritmanagement.com/resource/southeast-usa-regional-grit-gradation-data

Advantages of this alternative include:

- Grit removed is typically already well washed with low organic content, eliminating the need for a grit washer/classifier which further lowers equipment costs
- Consistent grit removal efficiency over a wide flow range
- Operator familiarity
- Capable of achieving grit removal efficiencies exceeding 95% for 210 micron particles
- Does not require demolition during construction

Disadvantages of this alternative include:

- Existing location within the facility is not ideal for implementation of future flow equalization facilities to provide grit removal prior to EQ storage
- Increased maintenance compared to other grit removal alternatives due to more moving parts (i.e., chain and bucket system)
- Lower overall grit removal efficiencies than other grit removal alternatives

- Large footprint
- High energy use and operational cost due to continuous air supply
- Grit removal performance can be difficult to fine tune without adequate process control
- Existing aerated grit removal chambers at WWTF have experienced grit carryover

The conceptual cost opinion for grit removal alternative 1 is included in the cost opinion for screening alternative 1 since these structures are connected and would be constructed at the same time if implemented. The conceptual cost opinion for screening alternative 1 was summarized above in **Table 2.25**. Estimated annual O&M costs for grit removal alternative 1 are summarized in **Table 2.35** and **Table 2.36** below for current and future design conditions. O&M cost estimates assumed that three aerated grit chambers would be required for the current condition, and four aerated grit chambers would be required for the 2040 design condition. Air supply to the aerated grit chambers was assumed to be continuous and horsepower requirements were estimated based on an air flow rate of 5 CFM per foot of chamber length at site conditions.

*Table 2.35 – Grit Removal Alternative 1 – Annual O&M Costs – 2021*

| Item | Annual Cost |
|---|---|
| Maintenance | $31,000 |
| Electricity | $13,000 |
| **TOTAL** | **$44,000** |

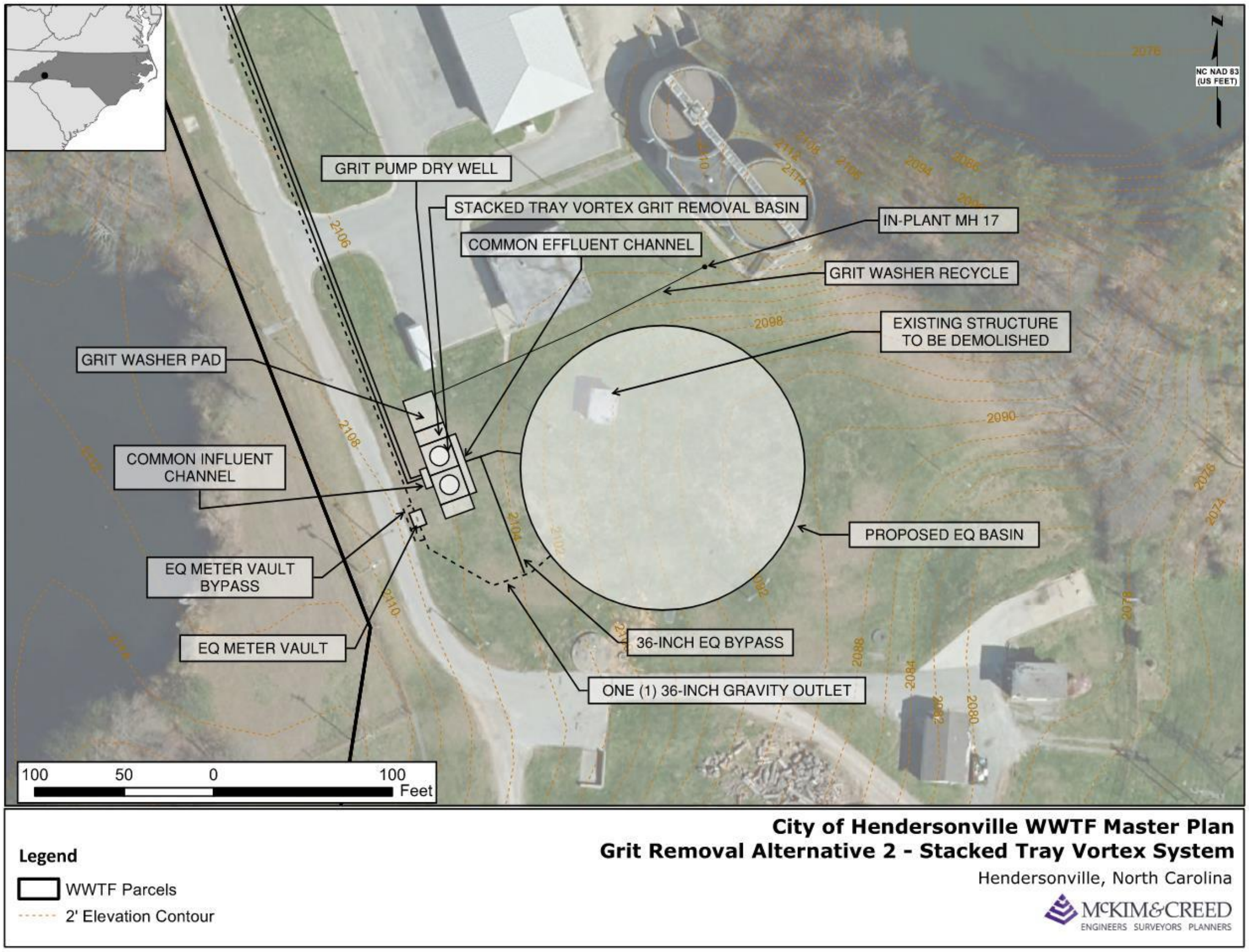
*Table 2.36 – Grit Removal Alternative 1 – Annual O&M Costs - 2040*

| Item | Annual Cost |
|---|---|
| Maintenance | $31,000 |
| Electricity | $18,000 |
| **TOTAL** | **$49,000** |

#### 2.4.2.2 *Alternative 2 – Construct New Stacked Tray Vortex Grit Removal System*

This alternative consists of the demolition of the current screening and grit removal system, and the construction of two new stacked tray vortex grit separators upstream of and adjacent to the EQ basin at the old plant site south of Balfour Road as shown in **Figure 2.13**. It is recommended to locate grit removal upstream of the EQ basin to prevent deposition and buildup of grit in the basin and other downstream processes. Locating the grit removal adjacent to the EQ basin minimizes the depth of excavation for the grit removal process as locating it at the headworks upstream of the IPS would require much deeper excavation to maintain gravity flow. Under this alternative, flow is pumped from the IPS to the stacked tray vortex grit separators after which it flows by gravity to the EQ basin.

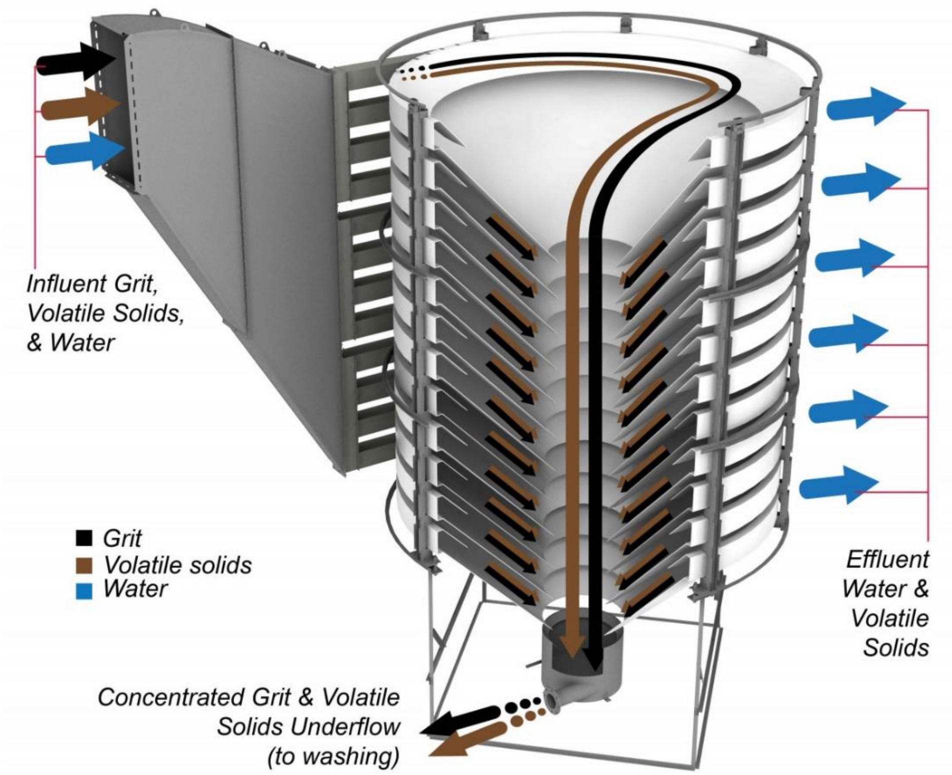
*Figure 2.13 - Grit Removal Alternative 2 Proposed Layout*




In a stacked tray vortex grit separator, a high efficiency flow-distribution header is used for evenly distributing influent over multiple conical trays as shown in **Figure 2.14** below. A tangential feed establishes a vortex flow pattern where solids settle into a boundary layer on each tray and are swept down to a center underflow collection chamber. The settled solids are continuously removed for grit washing and dewatering. The stacked trays create a large, concentrated surface area and short settling distances. The stacked tray vortex grit separator is a proprietary product of Hydro International, marketed as the HeadCell® grit separator.

Two 12-foot diameter 8-tray units provide the grit removal capacity required at the 2040 PHF with a proposed loading rate of 10.8 gpm per square foot of tray surface area. However, the number of trays can be increased to provide greater surface area and therefore greater grit removal capacity, provided there is sufficient basin depth. Stacked tray grit separators come in 4, 6, 9, and 12-foot diameters. For example, assuming the same loading rate of 10.8 gpm/ft$^2$ of tray surface area, two 9-foot diameter 12-tray units could be installed to handle the current and 2025 PHFs, with a third similar unit installed later to handle the 2040 PHF.

*Figure 2.14 Stacked Tray Vortex Grit Removal System*

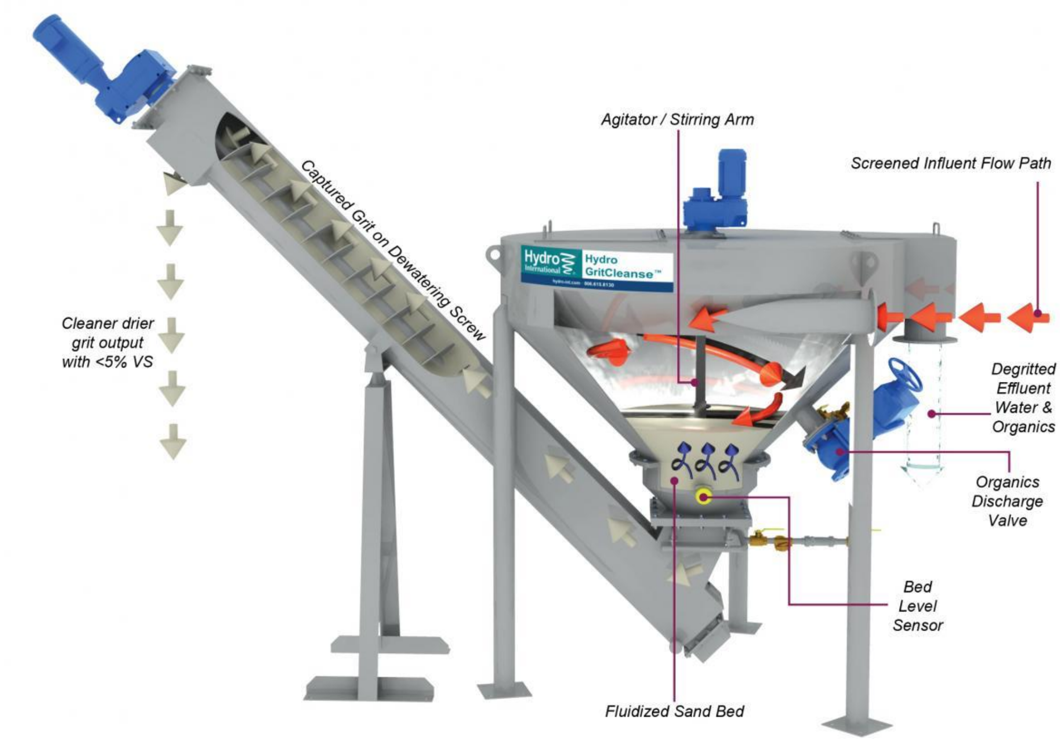



It is recommended that detailed sizing of the stacked tray vortex grit separators be performed during the detailed design phase, accounting for cost/benefit ratios and future expansion either through addition of adjacent grit separators (larger footprint) or addition of trays into existing stacked tray basins (deeper excavations and taller walls).

This proprietary design removes 95% of all grit 75 microns and larger at average flows. Based on the Southeast Grit Gradation Data in **Figure 2.12** above, this capture rate applies to nearly 100% of all grit in the flow stream.

Because solids settled from the unit contain grit plus some remaining organics bound to it, the grit separator system requires an accompanying grit washer/classifier and dewatering unit. It is recommended to separate the organics from the grit via washing/classifying to reduce odors. The washed/classified grit must then be dewatered to achieve a suitable moisture content for landfill disposal and to reduce disposal costs. It was assumed that a grit washer with a conical clarifier, similar to **Figure 2.15** below, would be installed to accompany the stacked tray and mechanically induced vortex grit separators evaluated in alternatives 2 and 3. It is recommended that the benefits and costs of grit washing & dewatering system alternatives be further weighed in the detailed design of grit removal alternatives if vortex-type grit separator equipment is to be installed.

*Figure 2.15 - Grit Washer with Conical Clarifier Diagram[1]*




*[1]Hydro GritCleanse, Hydro International, https://hydro-int.com/en/products/hydro-gritcleanse*

Advantages of this alternative include:

- Compact footprint
- Low head loss
- No submerged bearings or moving parts in vortex basin
- Higher grit removal and energy efficiency compared to aerated grit chambers and mechanically induced vortex grit removal alternatives
- Large degree of capacity flexibility as tray sizes and number of trays can be modified to meet site's specific flow and performance requirements

Disadvantages of this alternative include:

- Requires a grit washer unit to remove excess organics from the grit
- High capital cost
- Potential of collecting rags between stacked trays
- Deeper excavations compared to other grit removal alternatives
- Grit sump tends to clog and requires agitation or high pressure water jets or air to fluidize and loosen compacted grit

The conceptual cost opinion for Alternative 2 is included below in **Table 2.37**.

*Table 2.37 Grit Removal Alternative 2 Cost Opinion*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $925,000 |
| 2 | Mechanical | $185,000 |
| 3 | Electrical | $185,000 |
| 4 | Instrumentation | $93,000 |
| 5 | Structural | $283,000 |
| 6 | Civil | $298,000 |
| 7 | Demo | $50,000 |
| 8 | Mobilization & Demobilization | $81,000 |
| 9 | Indirect Costs | $95,000 |
| 10 | General Conditions & Contractor Markup | $462,000 |
| 11 | 30% Contingency | $798,000 |
| 12 | Engineering, Legal, & Administration | $641,000 |
| | **Total Cost Opinion** | **$4,096,000** |

**Table 2.38** and **Table 2.39** show the annual O&M costs for current and future production, respectively.

*Table 2.38 Grit Removal Alternative 2 - Annual O&M Costs - 2021*

| Item | Annual Cost |
|---|---|
| Maintenance | $19,000 |
| Electricity | $1,000 |
| **TOTAL** | **$20,000** |

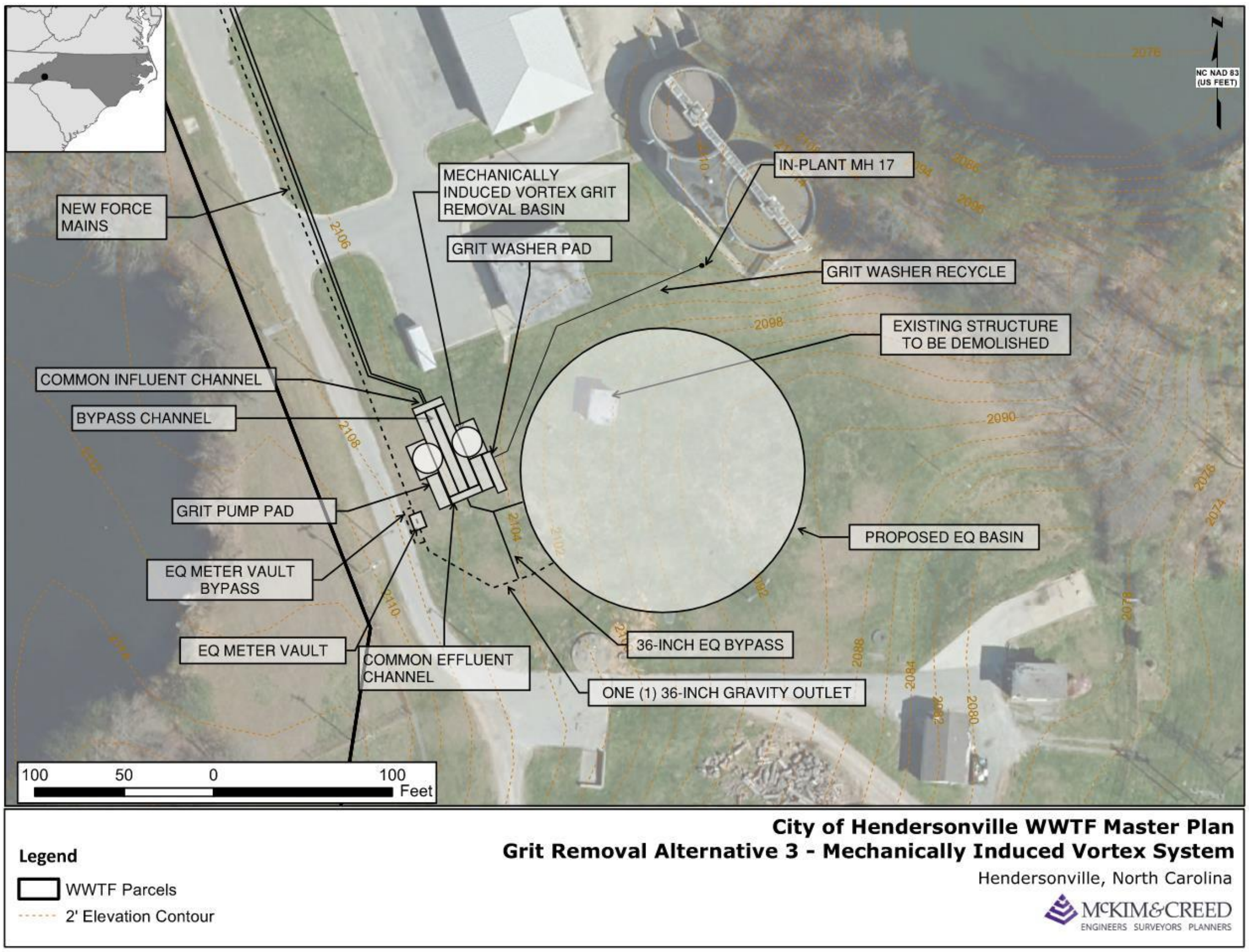
*Table 2.39 Grit Removal Alternative 2 - Annual O&M Costs - 2040*

| Item | Annual Cost |
|---|---|
| Maintenance | $19,000 |
| Electricity | $1,000 |
| **TOTAL** | **$20,000** |

### 2.4.2.3 *Alternative 3 – Construct New Mechanically Induced Vortex Grit Removal System*

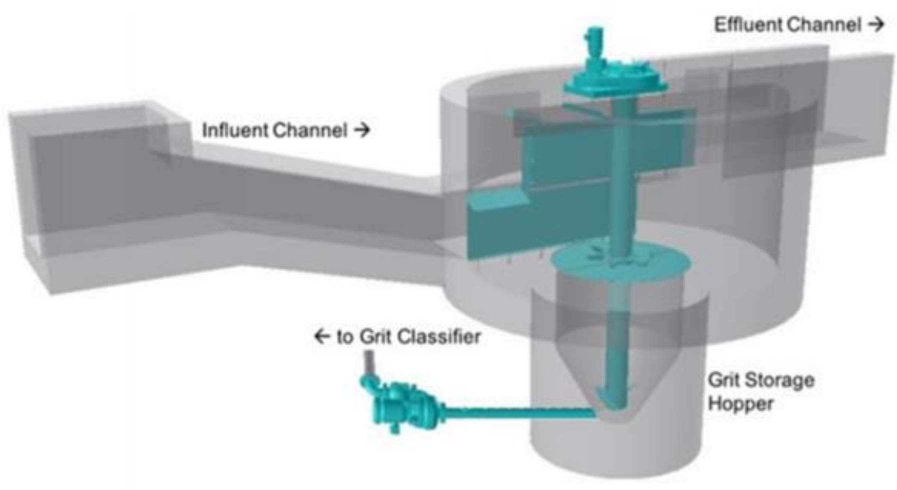
This alternative consists of the demolition of the current screening and grit removal system and the construction of two new mechanically induced vortex grit removal systems. Like Alternative 2, under this alternative, the grit removal system will be located upstream of and adjacent to the EQ basin as shown in **Figure 2.16** below. It is recommended to locate the grit removal upstream of the EQ basin to prevent deposition and buildup of grit in the basin.

*Figure 2.16 - Grit Removal Alternative 3 Proposed Layout*




Mechanically induced vortex grit separators combine the hydraulic forces associated with tangential incoming flow with mechanical paddles to create a vortex pattern. The system is comprised of two circular chambers: (1) an upper chamber where the flow enters allowing for the vortex creation that separates and settles the grit, and (2) a lower chamber where the settled grit is stored until it is removed as shown in **Figure 2.17**. The tanks are circular so that grit solids are forced to the side walls where boundary layer effects cause lower velocities. Near the walls, the vortex and gravity continue to move the captured grit solids into the lower chamber, which is then pumped by dedicated grit slurry pumps to the grit handling equipment for further processing.

*Figure 2.17 Mechanically Induced Vortex Grit Separator*




An example of the mechanically induced vortex grit separator, the Smith & Loveless PISTA Vortex Grit Removal System, is designed for 95% capture of grit 105 microns and larger. Based on the Southeast Grit Gradation Data in **Figure 2.12** above, this capture rate applies to approximately 90% of all grit in the flow stream.

Advantages of this alternative include:

- High grit removal efficiency
- Requires significantly less energy - for the estimated 2040 peak flow rate of 28.3 MGD, compared to an aerated grit chamber, the energy consumption of the Mechanically Induced Vortex Grit Removal System is approximately 1/7$^{th}$ of the energy consumed by an aerated grit chamber.
- Small footprint
- There are no submerged bearings or parts that require maintenance
- Head loss through a vortex system is minimal, typically 6 mm (0.25 in)

Disadvantages of this alternative include:

- Requires a grit washer unit to remove excess organics from the grit
- Paddles tend to collect rags

- Grit sump tends to clog and requires agitation or high pressure water jets or air to fluidize and loosen compacted grit

The conceptual cost opinion for Alternative 3 is included below in **Table 2.40.**

*Table 2.40 Estimated Grit Removal Alternative 3 Capital Costs*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $672,220 |
| 2 | Mechanical | $135,000 |
| 3 | Electrical | $135,000 |
| 4 | Instrumentation | $68,000 |
| 5 | Structural | $254,000 |
| 6 | Civil | $262,000 |
| 7 | Demo | $50,000 |
| 8 | Mobilization & Demobilization | $64,000 |
| 9 | Indirect Costs | $75,000 |
| 10 | General Conditions & Contractor Markup | $361,000 |
| 11 | 30% Contingency | $623,000 |
| 12 | Engineering, Legal, & Administration | $501,000 |
| | **Total Cost Opinion** | **$3,201,000** |

**Table 2.41** and **Table 2.42** show the annual O&M costs for current and future production, respectively.

*Table 2.41 Grit Removal Alternative 3 - Annual O&M Costs - 2021*

| Item | Annual Cost |
|---|---|
| Maintenance | $14,000 |
| Electricity | $2,000 |
| **TOTAL** | **$16,000** |

*Table 2.42 Grit Removal Alternative 3 - Annual O&M Costs - 2040*

| Item | Annual Cost |
|---|---|
| Maintenance | $14,000 |
| Electricity | $2,000 |
| **TOTAL** | **$16,000** |

## 2.5 Recommendations

In summary, each of the existing preliminary treatment process units – the influent pump station, screening system, and grit removal system – have limited capacity that will need to be addressed to accommodate expected 2025 and 2040 design conditions. All preliminary treatment process units are proposed to be upstream of the inline EQ basin recommended in Technical Memorandum No. 3, and therefore should be sized to handle non-equalized projected future PHFs. Six possible combinations of the various alternatives for each process unit are considered and listed below in **Table 2.43**.

*Table 2.43 - Possible Headworks Alternatives Combinations*

| Alternative Combination | Influent Pump Station Alternative | Screening Alternative | Grit Removal Alternative |
|---|---|---|---|
| **1** | 1 – Baseline | 1 – Baseline | 1 – Baseline |
| **2** | 2 – Expand Existing IPS | 1 – Baseline | 1 – Baseline |
| **3** | 2 – Expand Existing IPS | 2 - New Screening Facility Upstream of Expanded IPS | 2 – Stacked Tray |
| **4** | 2 – Expand Existing IPS | 2 - New Screening Facility Upstream of Expanded IPS | 3 – Induced Vortex |
| **5** | 3 – New IPS | 3 - New Screening Facility Upstream of New IPS | 2 – Stacked Tray |
| **6** | 3 – New IPS | 3 - New Screening Facility Upstream of New IPS | 3 – Induced Vortex |

A summary of the capital costs, annual O&M costs, and total NPV for each combination of headworks alternatives is provided in **Table 2.44** below.

*Table 2.44 - Headworks Alternatives Combinations Cost Estimate Summary*

| Alternative Combination | Capital Costs | 2025 Annual O&M Cost | 2040 Annual O&M Cost | Total 20-Year NPV |
|---|---|---|---|---|
| **1** | $15,604,000 | $142,000 | $152,000 | $13,904,000 |
| **2** | $18,698,000 | $136,000 | $146,000 | $17,391,000 |
| **3** | $18,531,000 | $90,000 | $91,000 | $16,228,000 |
| **4** | $17,636,000 | $86,000 | $87,000 | $15,448,000 |
| **5** | $20,546,000 | $118,000 | $122,000 | $18,187,000 |
| **6** | $19,651,000 | $114,000 | $118,000 | $17,406,000 |

As show in **Table 2.44** above, Alternative Combination 1 has the lowest capital cost and total net present value compared to any of the other alternatives. Despite its lower cost, Alternative Combination 1 may not be feasible due to the tight space limitations in the existing influent pump station dry well, and a high likelihood of adverse hydraulic phenomena in the existing undersized wet well. The limited space of the existing dry well coupled with the strong potential for adverse hydraulic conditions in the existing wet well is likely to result in an inefficient pump selection and reduced reliability. Alternative Combination 1 also does not address the recommendations of Technical Memorandum No. 1 to relocate screening ahead of the influent pump station to protect the pumps, improve reliability, and prolong equipment life. The location of the expanded screening and grit removal facilities under Alternative Combination 1 requires the construction of an additional pump station to direct all screened and de-gritted flow to inline flow equalization prior to the aeration basins. As a result, Alternative Combination 1 has the highest annual O&M costs. Based on these considerations, Alternative Combination 1 is not recommended to be implemented.

Based on this comparison, it is recommended that Alternatives Combination 4 be selected, which includes Influent Pump Station Alternative 2 (expanding existing IPS to the south), Screening Alternative 2 (relocating screening ahead of the expanded IPS), and Grit Removal Alternative 3 (mechanically induced vortex system immediately upstream of EQ). This combination of alternatives is recommended for the following reasons:

- It has the lowest capital cost and net present value of combinations 2 through 6
- Improved redundancy and reliability at the influent pump station
- Provides improved protection against equipment wear and may provide prolonged equipment lifespans due to relocation of screening and providing improved grit removal technology
- Provides additional flexibility for facility expansion beyond 2040

## 2.6 References


- Liquid Stream Fundamentals: Grit Removal Fact Sheet, Water Environment Federation, 2017 https://www.wef.org/globalassets/assets-wef/direct-download-library/public/03---resources/wsec-2017-fs-021-mrrdc-lsf-grit-removal_final.pdf#:~:text=grit%20further%20processing.-,Hydraulically%20Induced%20Vortex,%E2%89%A5%20106%20micron%20are%20retained.
- TeaCup Brochure, Hydro International, https://www.hydro-int.com/sites/default/files/teacup_1.pdf
- Weidler, J. (2017, January 10). How to choose between grit washing or grit classification. *Environmental Science & Engineering Magazine,* (December 2016). *https://esemag.com/wastewater/how-to-choose-between-grit-washing-or-grit-classification/*
- https://gritthefacts.com/pdfs/pista_vs_aerated_grit_chamber.pdf

# 3. SECONDARY TREATMENT PROCESS EVALUATION

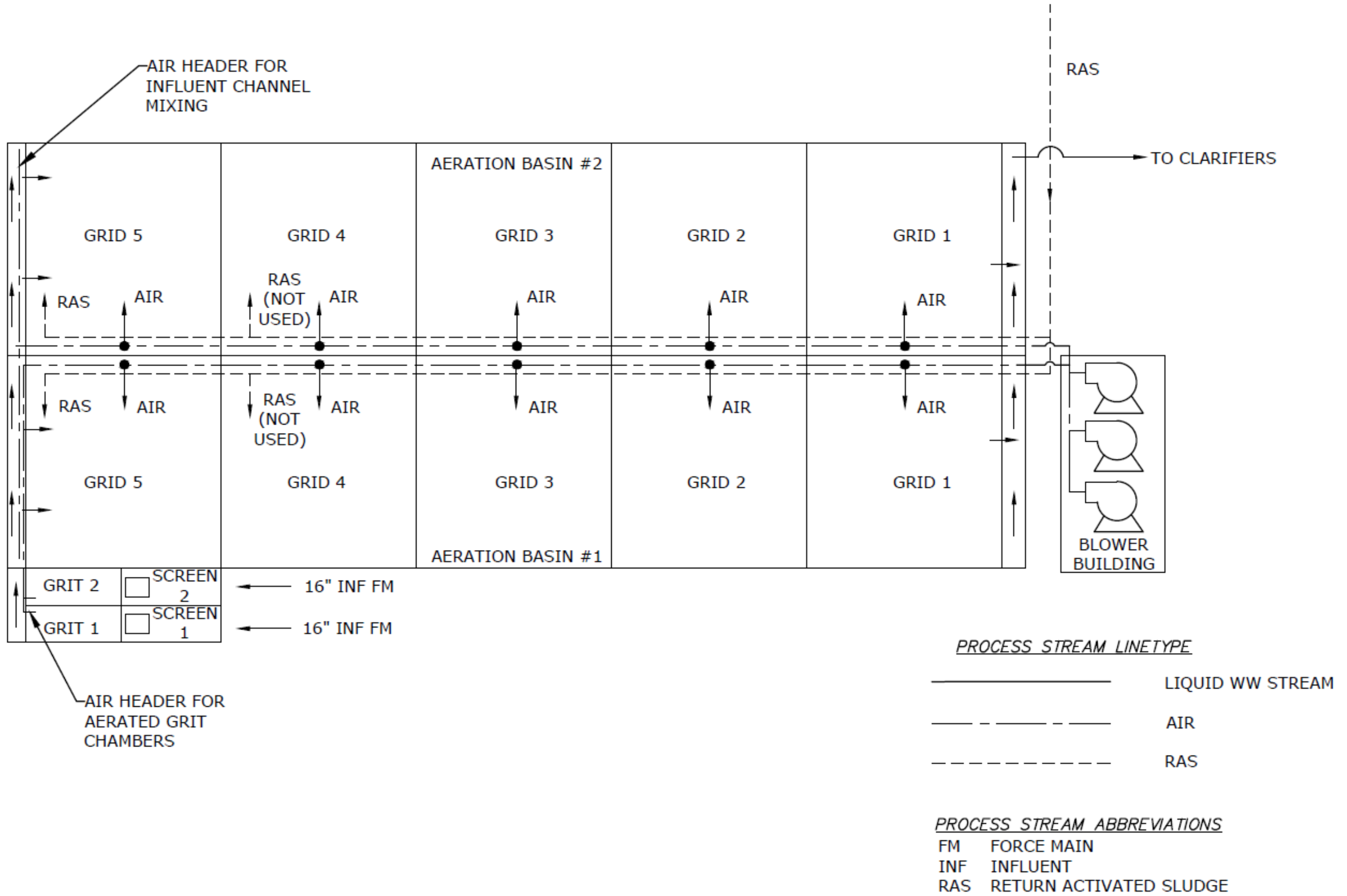
## 3.1 Purpose and Background

After primary treatment, all flow is biologically treated within the existing secondary treatment process. The WWTF is currently permitted to treat up to 4.8 MGD utilizing the existing secondary treatment process, consisting of two cast-in-place extended aeration process trains. Each train consists of five independent aerated grids, as shown in **Figure 3.1** below, equipped with fine bubble membrane diffusers. Each grid has an approximate volume of 465,000 gallons with each train having an active volume of approximately 2,324,000 gallons. Wastewater is currently aerobically treated within Grids 4 through 1 to achieve removal of BOD and nitrification of incoming TKN to meet the current NPDES discharge requirements. Grid 5, at the front of the train, is currently operated under anoxic conditions (performed by throttling air flow) to allow for partial denitrification of the return activated sludge (RAS) stream. While denitrification is not required by the existing facility NPDES permit, denitrification is operationally performed to recover alkalinity within the system. This is primarily performed to reduce the reliance of the WWTF on feeding supplemental alkalinity to maintain system pH, which is currently not performed, and increase operational efficiency. From available operating records, the secondary process has been historically operated to maintain a MLSS concentration of approximately 4,540 mg/L (2019 annual average) which is at the upper limit of typical conditions recommended for activated sludge systems utilizing gravity separation. Typically, design MLSS concentrations are closer to 3,500 mg/L but rarely above 4,500 mg/L in extended aeration systems.

After passing through the extended aeration basins, all flow is directed to a system of two 90-ft diameter secondary clarifiers for liquid/solids separation. Historically, it has been noted that the secondary clarifiers frequently become overloaded during high flow events due to the high solids loading rates from the extended aeration process. This results in excess solids passing over the effluent weirs of the clarifiers and loading to the downstream tertiary filters.

Settled solids are pulled off the bottom of the secondary clarifiers by a system of RAS and waste activated sludge (WAS) pumps. The RAS stream is directed back to the head of each aeration basin where it combines with primary effluent wastewater. The WAS stream wastes excess solids from the secondary process to the existing gravity thickener tanks at the dewatering facility across Balfour Road. The RAS pump station is currently operated to flow pace recycle streams as a percentage of the influent flow rate at the WWTF.

*Figure 3.1 – Existing Aeration Basins Diagram*




As outlined above, the City of Hendersonville is currently evaluating alternatives to meet current and future loading conditions through the planning horizon of 2040. This includes an evaluation of the capacity of the existing system and a preliminary evaluation of improvements required to meet future loading conditions. The City of Hendersonville has requested McKim & Creed to provide recommendations on potential facility configurations and technologies to allow the City to budget for future improvements as part of current master-planning efforts. This evaluation includes high-level assessments of the existing secondary treatment process, consideration of future treatment technologies to meet future effluent limits and approximate facility sizing to evaluate facility layout, and land-use needs with a focus on reuse of the existing WWTF site.

## 3.2 Design Criteria and Assumptions

Influent loading data was obtained from operational records from 2014 through 2019 to evaluate current loading conditions at the Hendersonville WWTF. Data was obtained for influent biochemical oxygen demand ($BOD_5$) and total suspended solids (TSS) which are collected regularly utilizing the existing composite sampler installed upstream of the mechanical bar screen equipment. Quarterly samples are currently collected for ammonia ($NH_3$-N) and total phosphorus (TP). Data was not readily available for influent volatile suspended solids (VSS), total Kjeldahl nitrogen (TKN) or alkalinity. While it is not possible to currently quantify future loading conditions at the WWTF, conservative estimates have been made based on current loading data and general assumptions consistent with typical medium strength domestic wastewater to provide a basis for estimating future treatment capacity needs. The assumed value for alkalinity represents the default value utilized within BioWin. In the event influent alkalinity is significantly lower than this, additional supplemental alkalinity may be required to maintain pH during normal operations. Influent wastewater characteristics utilized to evaluate the secondary treatment process as part of this evaluation are outlined below in **Table 3.1**.

*Table 3.1 – Average Influent Wastewater Concentrations for Process Modeling*

| Parameter | Units | Value |
|---|---|---|
| $BOD_5$ | mg/L | 219 |
| TSS | mg/L | 223 |
| VSS | mg/L | 156 |
| TKN | mg/L | 45 |
| TP | mg/L | 7 |
| Alkalinity | mmol/L | 6 |

It should be noted that the provided influent data does not allow for a reliable determination of the actual loading to the secondary process as the current influent sampling location is upstream of primary treatment. Primary treatment, including grit removal, results in a decrease in inorganic grit loading and to a lesser degree organic loading (BOD) prior to the secondary treatment process. The impact of primary treatment on the influent wastewater characteristics is currently not captured by the available influent data. Utilizing the available data when estimating capacity of the existing treatment process and evaluation of future secondary process improvements will be a more conservative approach for preliminary evaluation and equipment sizing. Prior to any detailed design efforts, it is recommended that the City perform additional influent wastewater characterization to get a more accurate measurement of loading rates to the secondary treatment process. It is recommended that regular samples be collected for typical wastewater parameters including $cBOD_5$, TSS, VSS, ammonia, TKN, TP, and alkalinity downstream of grit removal and prior to introducing any recycle streams.

As part of the capacity analysis exercise, it is necessary to estimate the level of treatment that will be required by the facility to ensure the treatment process will reliably meet projected effluent limits. Currently, the WWTF is only required to remove BOD and nitrify incoming TKN to $NO_3/NO_2$ as part of the facility's existing NPDES permit. As noted above, the City currently operates Grid 5 of each train as an anoxic zone by controlling air splitting to this Grid to achieve partial denitrification to improve process efficiency by reducing the need for supplemental alkalinity addition. However, the existing extended aeration process is not designed with the intent to achieve significant nutrient removal to meet more stringent effluent limits.

The WWTF currently discharges treated effluent to Mud Creek which is a tributary to the French Broad River. To estimate future effluent discharge limits, preliminary research was performed to identify any existing conditions which may require the WWTF to meet biological nutrient removal (BNR) limits in the future. Mud Creek has been documented to be a biologically impaired surface water body, with the primary source of impairment noted to be loading from agricultural and urban non-point sources. However, Total Maximum Daily Loads (TMDLs) have not been developed for the Mud Creek basin or the French Broad River basin which could result in limiting point source discharges within the watershed.

The largest NPDES facility within the French Broad River basin is the French Broad River WRF currently operated by the Metropolitan Sewerage District of Buncombe County. This facility is rated to treat up to 40 MGD and is located just north of Asheville, NC. Currently, the French Broad River WRF does not have limits for $NO_3$/$NO_2$, TN, or TP. This facility is designed only to achieve BOD removal and nitrification utilizing rotating biological contactors and will not achieve significant BNR as currently configured.

Based on existing conditions, it is not likely that BNR will be required for the Hendersonville WWTF within the planning horizon of this evaluation and therefore has not been included as part of the evaluation documented in this report. However, the general regulatory trend has been to include increasingly more stringent nutrient discharge limits on POTWs and therefore the impacts of potential future regulation should be further considered prior to proceeding with detailed design. To better estimate future discharge limits, the City should proceed with requesting updated speculative limits for the discharge into Mud Creek at the future design loading prior to proceeding with detailed design efforts.

For this evaluation, secondary treatment technologies have been limited to processes designed to achieve BOD removal and nitrification of incoming TKN with additional evaluation performed to include pre-anoxic treatment of wastewater for TN removal and alkalinity recovery. However, meeting more stringent nutrient limits was not considered further as part of this study. In the event BNR becomes necessary at the Hendersonville WWTF, additional evaluation, beyond the scope of this study, will be necessary to refine secondary process alternatives to meet more stringent effluent limits. However, as noted in the following section, this evaluation has considered the flexibility of secondary processes to be adapted to meet potential future BNR treatment requirements.

The planning level evaluation performed as part of this study was completed utilizing BioWin v6.2 as developed by EnviroSim (Ontario, Canada). Modeling efforts were limited to steady state simulations which provide the level of detail necessary to complete a master planning evaluation of this level. Dynamic simulations were not prepared as part of this evaluation as dynamic modeling is not required to reliably estimate the sizing of the activated sludge process and therefore is beyond the scope of this evaluation. Dynamic simulations require a significantly higher degree of understanding of system loading and are usually only necessary when evaluating system performance resulting from diurnal loading patterns; trouble shooting of existing systems; evaluating operational changes or preparing the detailed design of new systems. Prior to detailed design, a more significant sampling campaign will be required at the WWTF to better capture operating conditions with the expanded data set utilized to develop dynamic simulations of the treatment system.

It should be noted, the loading conditions considered in this evaluation are theoretical maximum loading rates and the likelihood of seeing such high loading rates (especially sustained) is extremely low. Therefore, results obtained from the below BioWin modeling exercise should be utilized with care when making decisions for future planning. This is recommended for several reasons:

1. Actual loading conditions will likely be lower than what has been assumed as part of this evaluation as no reduction in wastewater strength due to primary treatment has been included to remain conservative for planning purposes as it is not possible to accurately predict future loading conditions. Therefore, worst case conditions have been assumed.
2. It has been assumed that average wastewater strength and max day flow occur at the same time during max day loading at the design minimum temperature. This is extremely unlikely to occur as max day flow typically coincides with a significant rainfall event resulting in a significant amount of dilution due to I/I within the older collection system. This will result in a lower loading rate than that assumed as part of this evaluation.
3. The calculated design SRT utilized in this evaluation also assumes a lower DO level within the reactors (0.75 mg/L in summer and 1.0 mg/L in winter). Operating the system at a higher DO level during periods of extreme cold can increase the nitrification kinetics to better protect from nitrifier washout occurring.
4. Under extreme hydraulic loading events beyond what has been considered in this evaluation, it may become necessary to protect the secondary process when utilizing gravity separation by operating in a high-flow/stormwater mode. This operation would include turning off the blowers within the secondary process to allow the MLSS to separate and settle from the liquid phase in the bioreactors. Solids which settle to the bottom of the trains will be conserved within the system to allow for a quick startup of the secondary process after the peak hydraulic loading event has passed.

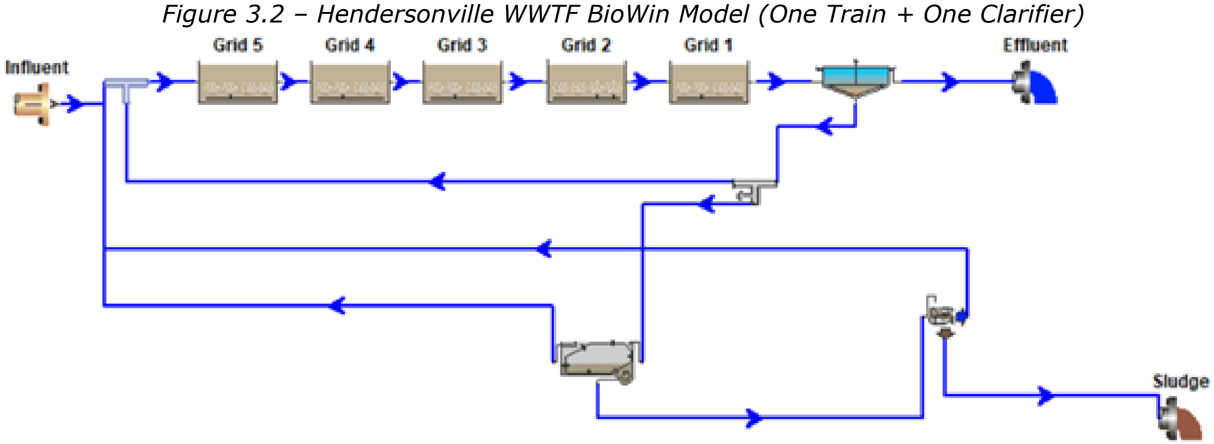
5. It should be further noted that extreme loading conditions considered in this evaluation would likely only occur at buildout (if at all) when loading to the plant is at its maximum. If the plant ever reaches this type of sustained loading, it will likely be decades out and an alternative wastewater treatment/management plan will need to be in place or brought online prior to reaching this condition.

## 3.3 Current Capacity Analysis

### 3.3.1 Secondary Process

The existing secondary treatment process (extended aeration trains and secondary clarifiers) was evaluated utilizing BioWin to determine the limitations of the existing system to treat current flows and future flows. A BioWin model was developed representing half of the existing secondary process as indicated below in **Figure 3.2**. The system outlined includes one extended aeration train, one secondary clarifier, one gravity thickener, and one belt filter press. Only half of the system was modeled to reduce computational resources and decrease simulation runtimes. The addition of the gravity thickener and belt filter press was included within the model to better predict the effects of internal recycle streams from the thickeners and presses back to the secondary process. Simulations were run under various loading and operating conditions to estimate the available capacity of the existing extended aeration trains and secondary clarifiers.

*Figure 3.2 – Hendersonville WWTF BioWin Model (One Train + One Clarifier)*




The BioWin modeling evaluation outlined in the following sections was performed utilizing the below assumptions:

1. The MLSS concentration within the extended aeration process will not exceed 4,500 mg/L as a primary means of controlling the treatment process and to ensure reliable gravity separation within the secondary clarifiers can occur.

2. For annual average and max month loading conditions, the operating SRT will not drop below the calculated design aerobic SRT. The calculated design aerobic SRTs for summer (20 $^oC$) and winter (10 $^oC$) were estimated to be 11.9 days and 13.8 days, respectively. The design aerobic SRT includes a process factor to account for variable loading conditions, variable MLSS concentrations, and periodic wasting from the process which is going to be most impactful for periods of sustained loading.
3. Under higher loading max day conditions, it may become necessary to operate at an SRT below the design SRT to maintain an MLSS approximately equal to the simulated max month operational MLSS concentration (but always less than 4,500 mg/L). Due to the short duration of these periods of lower SRT, it is anticipated that the biological system should recover quickly after the higher loading event has passed through the system. Simulations run under these higher loading conditions were performed to ensure that loss of nitrification was not observed to be significant within the simulation results. Process modeling utilizing BioWin is a powerful tool which can be utilized to estimate system performance under these types of loading conditions as the included process models consider many underlying processes which can result in significant interactions which would not be readily understood utilizing less sophisticated methods. Therefore, BioWin modeling will provide the best possible insight into what the impacts of operating for a short interval at an SRT lower than the design SRT will likely be.
4. Aeration requirements will never drop below the minimum rate required to maintain mixing of the activated sludge process.
5. The secondary clarifiers will not overload under any simulated normal operating conditions with all clarifiers in service.
6. Gravity thickener underflow and belt filter press cake solids concentrations were targeted at 3.36% and 17% TS, respectively. These values were previously developed as part of a mass balance exercise included in the Solids Management Plan Evaluation prepared for the City of Hendersonville by McKim & Creed.

#### 3.3.1.1 *Currently Permitted Maximum Month Average Daily Loading to Two Trains (2 Existing) and Two Clarifiers (2 Existing)*

The currently permitted capacity of 4.8 MGD was evaluated utilizing the BioWin model developed for the existing secondary treatment process utilizing the above outlined influent wastewater characteristics in **Table 3.1** and the above outlined assumptions. The BioWin simulation results indicated that the existing system would treat the permitted annual average flow without any foreseeable capacity or performance limitations. A summary of the modeling results is included in **Table 3.2** below.

*Table 3.2 – BioWin Simulation Results Summary (Current Permitted Capacity, 4.8 MGD)*

| Parameter | Summer (20 °C) | Winter (10 °C) |
|---|---|---|
| SRT (Days) | 19.2 | 19.2 |
| HRT (Hrs) | 23.2 | 23.2 |
| MLSS (mg/L) | 3,381 | 3,558 |
| MLVSS (mg/L) | 1,957 | 2,114 |

Based on the simulated results, the existing system has sufficient capacity to operate well below the maximum MLSS concentration while maintaining an operating SRT above the design SRT calculated for both summer and winter conditions. Therefore, the existing trains provide adequate capacity for the current permitted conditions when operated at the simulated MLSS concentration. It should be noted that this MLSS concentration is much lower than the historical average MLSS concentration closer to 4,500 mg/L. The City is currently in the process of reducing the operating MLSS of the existing system from 4,500 mg/L closer to the recommended MLSS concentration of 3,500 mg/L.

The existing secondary clarifiers were modeled as part of the simulation utilizing the included ideal clarifier model. The model was utilized to develop State-Point Analysis (SPA) diagrams for the clarifiers based on the simulated operating conditions for the secondary system. The SPA diagrams for simulations representing summer and winter design conditions are included in **Figure 3.3** and **Figure 3.4** below. Results indicated that the clarifiers are sufficiently underloaded where capacity limitations are not anticipated to occur under simulated operating conditions.

*Figure 3.3 – SPA Diagram, 2 Clarifiers (Currently Permitted Capacity = 4.8 MGD, Summer)*

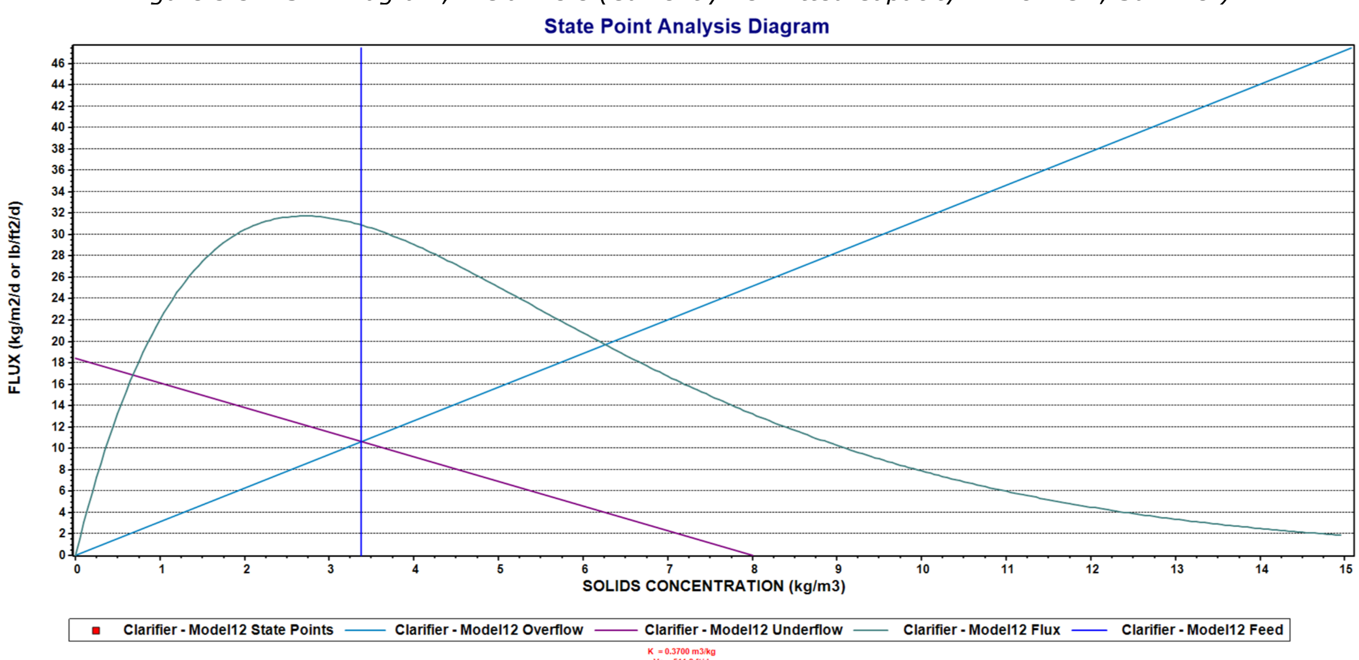
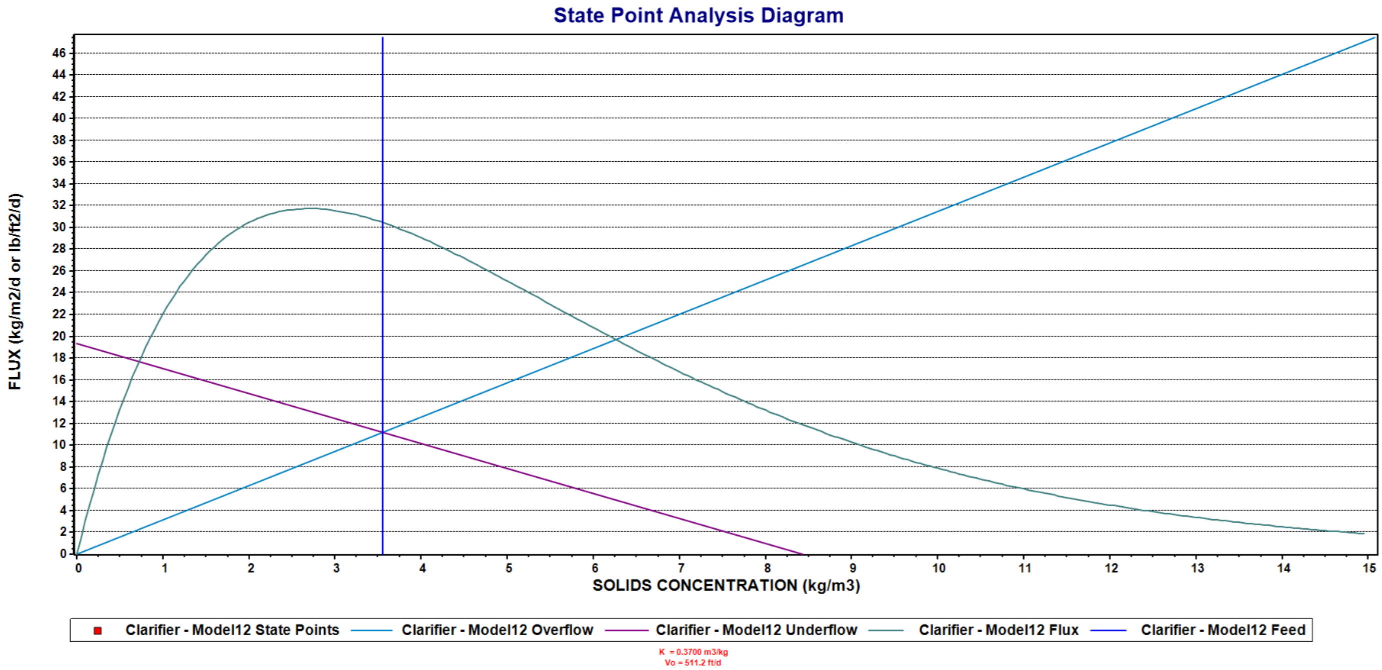


*Figure 3.4 – SPA Diagram, 2 Clarifiers (Currently Permitted Capacity = 4.8 MGD, Winter)*




## 3.3.2 Blower Building

The existing blower building at the WWTF houses three 250 hp Hoffman multistage centrifugal blowers (2 duty, 1 standby) to supply air to the aeration basins. The design information for the existing blowers is summarized in **Table 3.3** below.

*Table 3.3 – Existing Blower Design Information*

| Parameter | Units | Value |
|---|---|---|
| No. of Existing Blowers | - | 3 |
| Rated Design Capacity, each | SCFM | 4,400 |
| Firm Capacity, total | SCFM | 8,800 |
| Inlet Pressure at Rated Capacity | psia | 13.40 |
| Atmospheric Pressure | psia | 13.60 |
| Inlet Air Temperature at Rated Capacity | °F | 100 |
| Relative Humidity at Rated Design Conditions | % | 50% |
| Blower Speed at Rated Capacity | RPM | 3,550 |
| Blower Input Horsepower at Rated Capacity, each | hp | 246.2 |
| Blower Efficiency at Rated Capacity | % | 72% |
| Discharge Pressure at Rated Capacity | psig | 9.00 |
| Discharge Temperature at Rated Capacity | °F | 224.68 |
| Motor Horsepower Rating, each | hp | 250 |
| Electrical Service Voltage | V | 480 |
| Electrical Service Phases | - | 3 phase |
| Motor Drive Type | - | Constant Speed |

Aeration blowers must be sized to accommodate the air demands of the biological process and to provide adequate mixing to prevent solids deposition in the aeration basins. The current capacity of the existing blowers was evaluated using the results of the BioWin modeling for the current capacity analysis along with typical design calculations and assumptions to estimate worst case conditions. The worst case design condition for blower capacity occurs at the peak daily sustained loading during summer conditions. In contrast, the worst case design condition for the blower's motor and driver occurs at the coldest winter conditions. Typical design guidance suggests that peak load conditions should be estimated using a peaking factor of 1.5 to 2.0 times the average BOD and TKN loading (Tchobanoglous, 2014, p. 886). During this evaluation a peak loading factor of 1.5 was applied to the average BOD and TKN loading occurring at the maximum month design flow to estimate the peak daily sustained loading conditions that may occur due to diurnal variations or slug contaminant loads from significant industrial users. The biological air demands were estimated for current (2021), 2025, and 2040 design conditions as shown in **Table 3.4** below to check against the existing blowers' firm capacity rating. The estimated peak brake horsepower requirements associated with the peak oxygen demands are shown in **Table 3.5** to compare against the motor horsepower rating of the existing blowers' motors.

*Table 3.4 – Estimated Peak Oxygen Demands*

| Parameter | Units | Value |
|---|---|---|
| Current (2021) Peak Oxygen Demand | SCFM[1] | 6,100 |
| 2025 Peak Oxygen Demand[2] | SCFM[1] | 7,400 |
| 2040 Peak Oxygen Demand[3] | SCFM[1] | 10,340 |

*[1]Standard air conditions are T = 20 °C, P = 1 atm, RH = 36%*
*[2]2025 peak conditions assume permitted facility capacity of 6.0 MGD through existing trains*
*[3]2040 peak conditions assume permitted facility capacity of 7.8 MGD with the addition of a third like-sized treatment train*

*Table 3.5 – Estimated Peak Blower Brake Horsepower Requirements*

| Parameter | Units | Value |
|---|---|---|
| Current (2021) Peak Blower Brake hp Required | hp | 407 |
| 2025 Peak Blower Brake hp Required | hp | 494 |
| 2040 Peak Blower Brake hp Required | hp | 690 |

*Note: Blower efficiency assumed to match rated efficiency of existing blowers*

As shown above, the existing blowers have sufficient capacity to meet firm capacity requirements up to the 2025 design conditions for a permitted facility capacity of 6.0 MGD. The estimated peak brake horsepower requirements at the 2025 design conditions approach the limits of the existing blower motors. However, energy requirements for aeration under the 2025 design conditions are not expected to exceed the capacity of the blower building's existing electrical service. The existing blowers do not have sufficient capacity for the 2040 design conditions for a permitted facility capacity of 7.8 MGD. It was assumed that a third biological treatment train would be required under the 2040 design conditions, so this capacity limitation is not unexpected. Based on this information, the existing blowers have sufficient capacity to continue to serve the two existing process trains. An additional blower building is recommended to be constructed for 2040 design conditions to serve a third treatment train. This would allow the existing blower building to continue to serve the two existing aeration basins in perpetuity, and aeration demands for the future third aeration basin would be supplied independently.

As noted in Technical Memorandum No. 1 of this master plan, the three existing multistage centrifugal blowers are reaching the end of their useful life and are oversized for the current aeration capacity requirements with no capability for reduced speed operation. It is recommended that the existing centrifugal blowers be replaced with new turbo blowers with VFDs sized for the 2025 design conditions. The 2025 design conditions are expected to result in the greatest oxygen demands for the two existing treatment trains.

### 3.3.3 Recycle Pumping Station

The existing recycle pumping station at the WWTF houses both RAS and WAS pumping systems for the two existing treatment trains. The design information for the existing RAS and WAS pumps is summarized in **Table 3.6** and **Table 3.7** below.

*Table 3.6 – Existing RAS Pumps Design Information*

| Parameter | Units | Value |
|---|---|---|
| No. of Existing RAS Pumps | - | 2 |
| Rated Design Capacity, Each | gpm | 2,500 |
| TDH at Rated Capacity | ft | 18 |
| Speed at Rated Capacity | RPM | 585 |
| Pump Full Speed | RPM | 890 |
| Estimated Capacity at Full Speed | gpm | 4,300 |
| Impeller Diameter | inch | 17 |
| Motor Horsepower Rating | hp | 50 |
| Motor Maximum Speed | RPM | 900 |
| Electrical Service Voltage | V | 480 |
| Electrical Service Phases | - | 3 |
| Motor Drive Type | - | VFD |

*Table 3.7 – Existing WAS Pumps Design Information*

| Parameter | Units | Value |
|---|---|---|
| No. of Existing WAS Pumps | - | 2 |
| Rated Capacity at 1st Condition Point, Each | gpm | 400 |
| TDH at 1st Condition Point | ft | 78 |
| Speed at 1st Condition Point | RPM | 880 |
| Rated Capacity at 2nd Condition Point, Each | gpm | 825 |
| TDH at 2nd Condition Point | ft | 75 |
| Speed at 2nd Condition Point | RPM | 880 |

| Parameter | Units | Value |
|---|---|---|
| Pump Full Speed | RPM | 1180 |
| Impeller Diameter | inch | 18 |
| Motor Horsepower Rating | hp | 60 |
| Motor Maximum Speed | RPM | 900 |
| Electrical Service Voltage | V | 480 |
| Electrical Service Phases | - | 3 |
| Motor Drive Type | - | VFD |

RAS pumping systems are typically designed to provide flowrates of 20% to 100% of the average facility design flow and up to 150% of the average design flow for smaller treatment facilities. McKim & Creed recommends that the RAS pumping system for the City's WWTF be designed to provide a firm capacity of approximately 110% of the design maximum month flow to ensure sufficient operational flexibility for peak loading conditions and wet weather operation. The existing recycle pump station currently houses two RAS pumps, with one duty pump and one standby pump. As shown in the table above, the firm capacity of the existing RAS pumps is approximately 4,300 gpm when one pump is operated at full speed. This equates to a firm capacity of approximately 6.2 MGD, or 129% of the 4.8 MGD permitted capacity of the existing WWTF. The firm capacity of the existing RAS pumps exceeds the requirements for current design.

When compared to the 2025 design conditions, the existing RAS pumps provide a firm capacity of 103% of the assumed 6.0 MGD permitted capacity. The permitted capacity of the facility is based on the maximum month conditions, therefore the firm capacity of the existing RAS pumps is just below the recommended firm capacity of 110% of the 6.0 MGD design maximum month flow. However, the firm capacity of the existing RAS pumps is 147% of the projected 2025 design average flow of 4.23 MGD, which significantly exceeds the typical recommended range based on average design flow. Therefore, no modifications are necessary to the existing RAS pumping system until the 2025 design conditions are exceeded. It is assumed that a third treatment train would be required to meet the 2040 design conditions. It is recommended that expansion of the WWTF to meet 2040 design conditions include the construction of a second RAS/WAS pump station to serve a third treatment train independently from the existing two treatment trains. This would allow the existing recycle pumping station to continue to serve the two existing treatment trains in perpetuity.

The firm capacity of WAS pumping systems is recommended to be designed based on the maximum day loading conditions. Similar to the RAS pumps, the existing recycle pump station currently houses two WAS pumps, with one duty pump and one standby pump. As shown in **Table 3.7** above, the firm capacity of the existing WAS pumps is 400 gpm at the first condition point. The first condition point corresponds to the existing system curve of a single 8-inch diameter waste sludge force main in service discharging to one gravity thickener at its maximum side water depth. The existing WAS pumps are significantly oversized to accommodate an intermittent sludge wasting schedule. As noted in Technical Memorandum No. 1, the City currently wastes sludge approximately every other day for 8 to 10 hours per day. Sludge wasting is recommended to be operated on a more continuous basis once thickened WAS holding facilities are constructed as recommended previously. For reference, the 2040 maximum day WAS production rate predicted from the BioWin modeling was approximately 260,000 gpd. This equates to a wasting rate of 181 gpm if continuous sludge wasting is provided. This wasting rate is well below the firm capacity of 400 gpm at the first condition point for the existing WAS pumps. As noted above regarding the RAS pumps, no improvements to the existing WAS pumps are necessary to continue to serve the two existing treatment trains in perpetuity. It is recommended that a second RAS/WAS pump station be constructed to serve a third treatment train to meet the 2040 design conditions.

## 3.4 Alternatives Screening

Based on the future influent flow projections that were presented in Technical Memorandum No. 1 of this master plan, it is expected that the City will need to expand the existing WWTF in the near future to continue to meet the wastewater treatment needs of the City's service area. Prior to evaluating alternatives for future facility expansion, a list of potential treatment technology alternatives was identified for preliminary screening. The list of alternatives was developed considering the limitations of the existing WWTF site. The existing WWTF is located on a large 53.64 acre property, however most of the land area available for future expansion of the existing WWTF is located within the FEMA 100-year floodplain of Mud Creek. Development within the existing FEMA 100-year floodplain would require significant earthwork costs, potential impacts to jurisdictional waters, and increased environmental permitting requirements. Based on these limitations, it was prudent to consider treatment technology alternatives that could offer increased treatment capacity with reduced space requirements compared to traditional activated sludge alternatives. The following treatment technology alternatives were considered during preliminary screening:

- Alternative 1: Extended aeration (baseline alternative, expansion of existing process)
- Alternative 1(a): Modified Ludzack-Ettinger (MLE) (modification to Alternative 1)
- Alternative 2: BioMag Ballasted Activated Sludge
- Alternative 2(a): BioMag with MLE (modification to Alternative 2)
- Alternative 3: Membrane Bioreactor (MBR)
- Alternative 4: Integrated fixed-film activated sludge (IFAS)

- Alternative 5: New WWTF at new site (process technology undetermined)

It should be noted that the above list of potential treatment technology alternatives does not include granular activated sludge. Granular activated sludge is a relatively new treatment technology in the US that offers a significant increase in treatment capacity within the existing footprint of the WWTF due to the rapid settling rate of the granular sludge which in-turn allows higher biomass concentrations in the system. This treatment technology is a breakthrough in the conventional activated sludge process and has a promising future in municipal wastewater treatment applications. However, the granular activated sludge process is currently a proprietary process owned by Aqua-Aerobic Systems, Inc. and marketed under the trade name of AquaNereda®. This technology has very few installations within the US, requires specialized start-up procedures to initiate the granulation process or seeding from another AquaNereda® facility, and is typically implemented in a sequencing batch reactor (SBR) process configuration. The AquaNereda® process was not considered in the preliminary alternatives screening process due to its early stage of implementation in the US and the potential requirement for a complete reconfiguration of the existing WWTF's processes to an SBR style process. However, this technology should continue to be considered in the future as it matures and is implemented in more flow-through processes similar to the City's WWTF. Detailed designs for future WWTF expansions should continue to consider this process prior to final design.

The goal of the preliminary screening was to identify the most feasible treatment technology options to be evaluated in more detail. To do this, each of the preliminary alternatives were compared based on benchmark capital and operational cost expectations as well as non-cost criteria. The non-cost criteria were:

- Adaptability of the technology to future effluent limit restrictions
- Land area required (land use)
- Expected energy efficiency
- Expected maintenance intensity
- Expected chemical requirements
- Expected sludge quantity
- Expected sludge quality
- Personnel requirements
- Operator familiarity with the technology
- Constructability

- Impacts to streams, wetlands, or other environmental impacts

The relative importance and weighting of the evaluation criteria was established using a pair-wise comparison weighting process. City staff were asked to complete a pair-wise comparison worksheet to provide their rankings of the relative importance of each evaluation criterion. The rankings from City staff were then normalized and the normalized rankings were averaged to determine the percent weight for each criterion. The finalized pair-wise comparison weighting matrix is included at the end of this Technical Memorandum in **Appendix A**

With the evaluation criteria weighting established, each alternative was scored based on the cost and non-cost criteria. The scoring for each alternative was totalized and the total scoring for each alternative was compared to identify the three highest ranked alternatives for further evaluation. The results of the preliminary alternatives screening scoring is summarized in **Table 3.8** below, and the full spreadsheet is included at the end of this Technical Memorandum in **Appendix A**

*Table 3.8 – Preliminary Alternatives Screening Scoring Summary*

| Alternative # | Alternative Name | Total Weighted Score |
|---|---|---|
| 1(a) | Baseline + MLE | 7.10 |
| 1 | Baseline | 6.71 |
| 2(a) | BioMag + MLE | 5.51 |
| 2 | BioMag | 5.36 |
| 4 | IFAS | 5.31 |
| 5 | New WWTF at New Site | 4.69 |
| 3 | MBR | 3.50 |

As shown in the table above, the top three alternatives were 1(a), 1, and 2(a). In review of this ranking order with the City, it was noted that alternatives 1(a) and 2(a) may be viewed as an extension of alternatives 1 and 2 respectively, and that the top four alternatives actually represents only two larger categories of alternatives. Based on this discussion, it was recommended that alternative 1(a), 2(a), and 4 be selected for further evaluation to ensure a wider variety of alternatives are considered for future expansion. Therefore, the recommended alternatives for further evaluation are summarized in **Table 3.9** below.

*Table 3.9 – Treatment Technology Alternatives Recommended for Further Evaluation*

| Alternative # | Alternative Name | Total Weighted Score |
|---|---|---|
| 1(a) | Baseline + MLE | 7.10 |
| 2(a) | BioMag + MLE | 5.51 |
| 4 | IFAS | 5.31 |

## 3.5 Alternatives Evaluation

As part of this evaluation, consideration has been given to meeting projected 2040 loading rates. Per the flow projections previously performed by others in the SSAIA Master Plan Report, an annual average flow of 5.9 MGD is expected to be reached in 2040. The previous flow projections and the influent wastewater characteristics listed in **Table 3.1** were utilized as the basis for estimating future treatment needs at the City's WWTF. As part of evaluating the treatment needs of the secondary treatment process, it is necessary to estimate the 2040 max month and max day loading conditions. Historical flow data from 2016 through 2018 was utilized to update estimated peaking factors for the max month and max day hydraulic loading rates at the WWTF. It was estimated that the max month and max day loading conditions have associated peaking factors of 1.42 and 1.80, respectively, compared to the annual average loading conditions. Therefore, these peaking factors have been utilized to estimate 2040 max month and max day loading conditions. The 2040 max month and max day hydraulic loading rates were estimated to be 8.39 MGD and 10.65 MGD, respectively. Wastewater characteristics utilized for BioWin simulations are as indicated above in **Table 3.10**. A summary of flow conditions evaluated is summarized in Table 3.10 below.

*Table 3.10 – Estimated 2040 Loading Conditions*

| Parameter | Flow (MGD) |
|---|---|
| 2040 Annual Average | 5.9 |
| 2040 Max Month | 8.39 |
| 2040 Max Day | 10.65 |

### 3.5.1 Alternative 1(a): Extended Aeration/Secondary Clarification

As noted above, the City of Hendersonville's existing WWTF utilizes an extended aeration activated sludge process for biological treatment of incoming wastewater. Solids separation is provided by gravity separation utilizing existing secondary clarifiers. Expanding the existing WWTF utilizing in-kind technologies has been considered as the baseline scenario to manage 2040 loading conditions.

#### 3.5.1.1 *2040 Annual Average Loading to Two Trains (2 Existing) and Two Clarifiers (2 Existing)*

The existing two train and two clarifier system was evaluated at the projected 2040 annual average loading condition utilizing the developed BioWin model. A summary of the simulation results is indicated below in **Table 3.11**.

*Table 3.11 – BioWin Simulation (2 Trains + 2 Clarifiers) Results Summary (2040 Annual Average Loading, 5.9 MGD)*

| Parameter | Summer (20 °C) | Winter (10 °C) |
|---|---|---|
| SRT (Days) | 16 | 16 |
| HRT (Hrs) | 18.9 | 18.9 |
| MLSS (mg/L) | 3,580 | 3,781 |
| MLVSS (mg/L) | 2,106 | 2,285 |

Based on the results from the BioWin simulation, it was determined that the existing extended aeration trains will provide adequate capacity to meet the 2040 annual average loading condition without any observed capacity or treatment limitations. As noted in **Table 3.11**, the SRT simulated exceeded the calculated design SRT noted above. In addition, the MLSS concentration remained well below the 4,500 mg/L maximum recommended concentration.

Evaluation of the existing secondary clarifiers was also performed utilizing the ideal clarifier model SPA. The SPA diagrams are included below in **Figure 3.5** and **Figure 3.6**. The SPA diagrams indicated that under the simulated 2040 annual average loading conditions, the existing clarifiers will remain underloaded and are therefore capable of processing the projected 2040 annual average loading without compromising clarifier performance.

*Figure 3.5 – SPA Diagram, 2 Clarifiers (2040 Annual Average Loading = 5.9 MGD, Summer)*

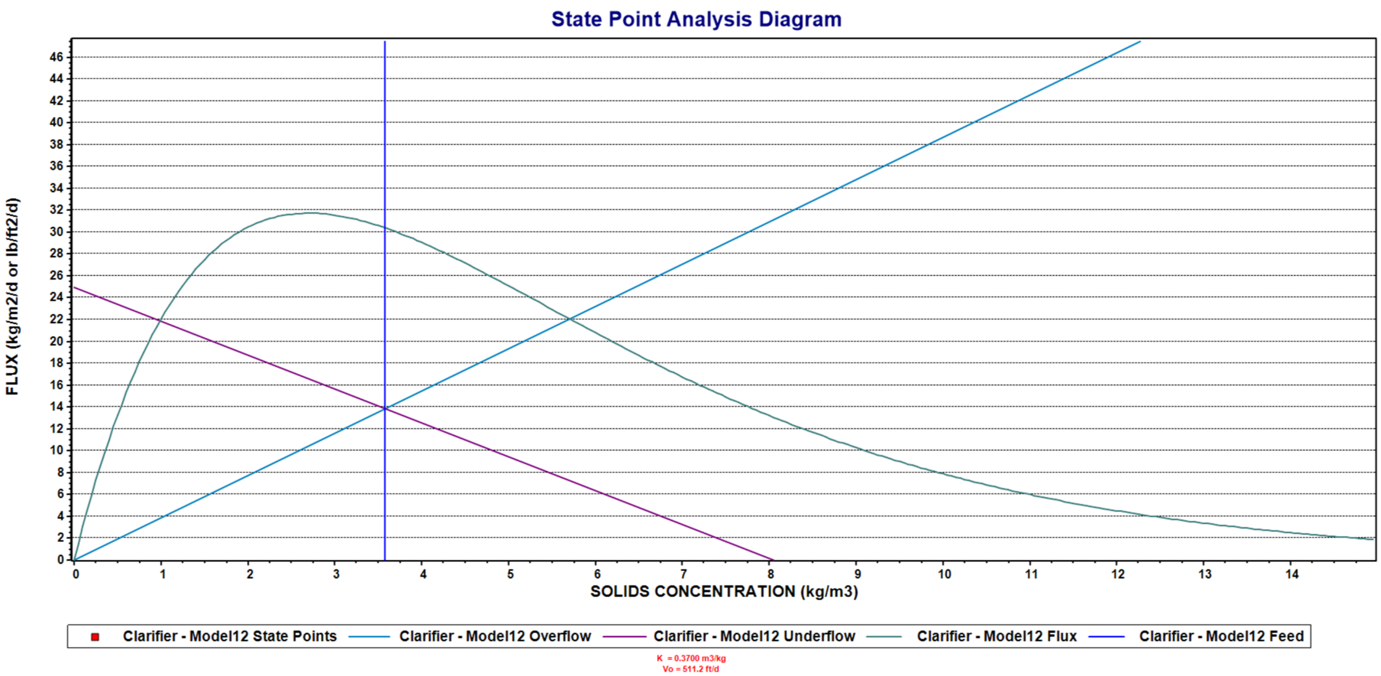
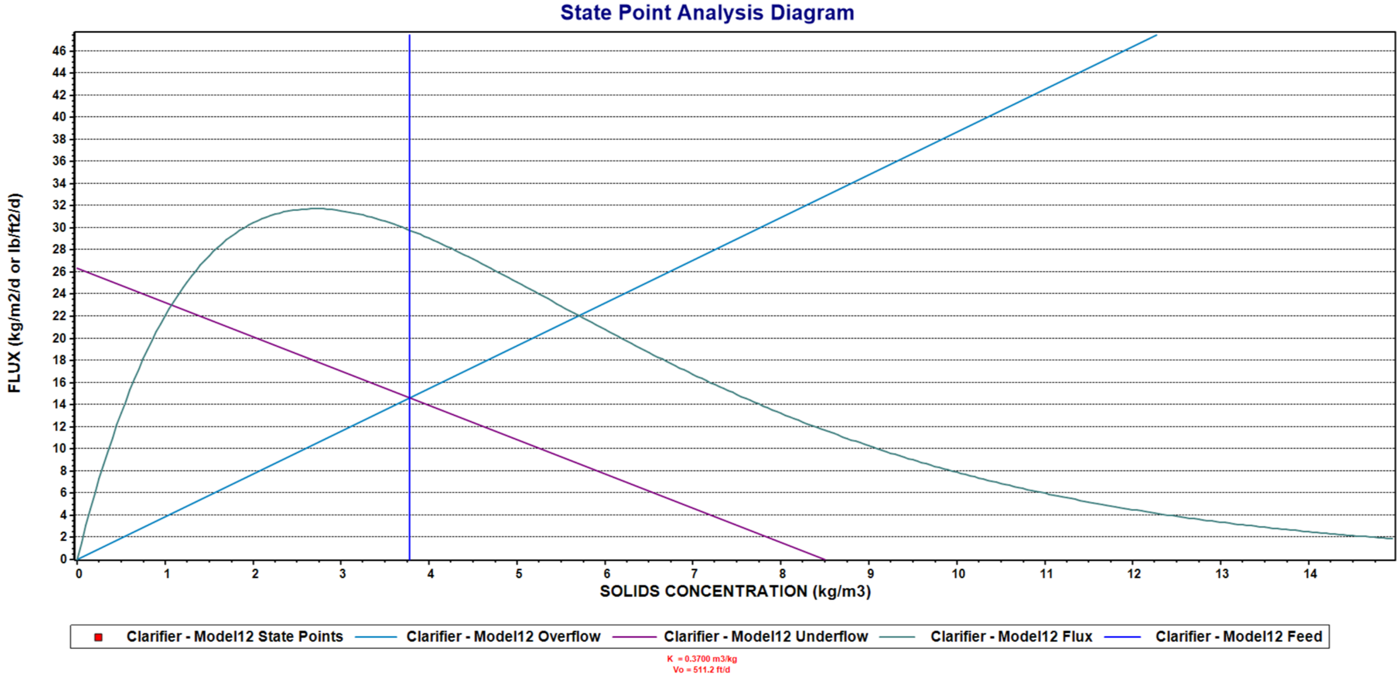


*Figure 3.6 – SPA Diagram, 2 Clarifiers (2040 Annual Average Loading = 5.9 MGD, Winter)*




### 3.5.1.2 *2040 Max Month Loading to Two Trains (2 Existing) and Two Clarifiers (2 Existing)*

The existing two train and two clarifier treatment system was further evaluated at the projected 2040 max month loading condition utilizing the developed BioWin model. A summary of the simulation results is indicated below in **Table 3.12**.

*Table 3.12 – BioWin Simulation (2 Trains + 2 Clarifiers) Results Summary (2040 Max Month Loading, 8.39 MGD)*

| Parameter | Summer (20 °C) | Winter (10 °C) |
|---|---|---|
| SRT (Days) | 12 | 13.1 < 13.8[1] |
| HRT (Hrs) | 13.3 | 13.3 |
| MLSS (mg/L) | 4,007 | 4,561 > 4,500[2] |
| MLVSS (mg/L) | 2,419 | 2,808 |

*[1]Minimum design winter SRT of 13.8 days*
*[2]Maximum design MLSS of 4,500 mg/L.*

Based on the results from the BioWin simulation, it was determined that the existing extended aeration trains will have adequate capacity to handle the 2040 max month loading during summer. The SRT was simulated at the calculated summer design SRT of 11.9 days. At the design SRT, the operating MLSS was simulated to be 4,007 mg/L which is less than the 4,500 mg/L recommended maximum MLSS concentration. While the existing system was simulated to have adequate capacity during summer operations at 2040 max month loading, it was determined from the simulation results that the existing extended aeration trains will not have capacity to handle 2040 max month winter loading. As documented in **Table 3.5**, the 13.1 day SRT simulated was below the calculated design SRT of 13.8 days noted above. At this condition, the possibility of washout becomes more likely to occur under a period of extended loading (like the maximum month loading period). The MLSS concentration of 4,561 mg/L at the simulated operating conditions had already exceeded the recommended maximum of 4,500 mg/L. Therefore, it is not possible to operate at a higher MLSS without significantly exceeding the 4,500 mg/L MLSS maximum concentration. Based on the simulation results, to accommodate the 2040 max month loading utilizing extended aeration technology, a third extended aeration train will be required.

As part of this evaluation, the existing clarifiers were modeled utilizing the ideal clarifier model and SPA method. The SPA diagrams are included below in **Figure 3.7** and **Figure 3.8**. Based on the SPA diagrams, the existing two clarifier system is approaching a critically loaded condition at the 2040 max month loading at the simulated winter operations. At the current simulated loading condition, an additional clarifier would be recommended. However, with the addition of a third train, the system will be able to operate at a lower MLSS concentration which will result in a lower solids loading rate to the existing secondary clarifiers. Therefore, it is necessary to further evaluate the impacts of a third train on the performance of the existing secondary clarifiers at the 2040 loading conditions. This will be addressed in the following sections.

*Figure 3.7 – SPA Diagram, 2 Clarifiers (2040 Max Month Loading = 8.39 MGD, Summer)*

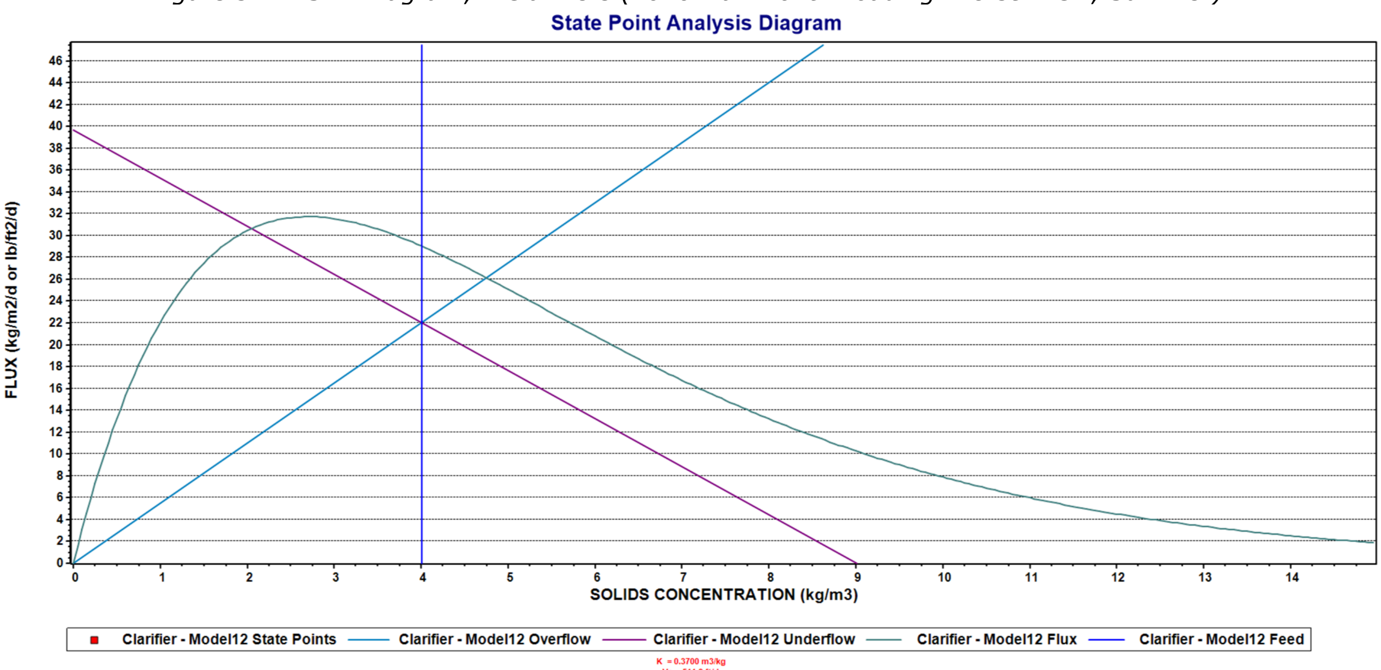
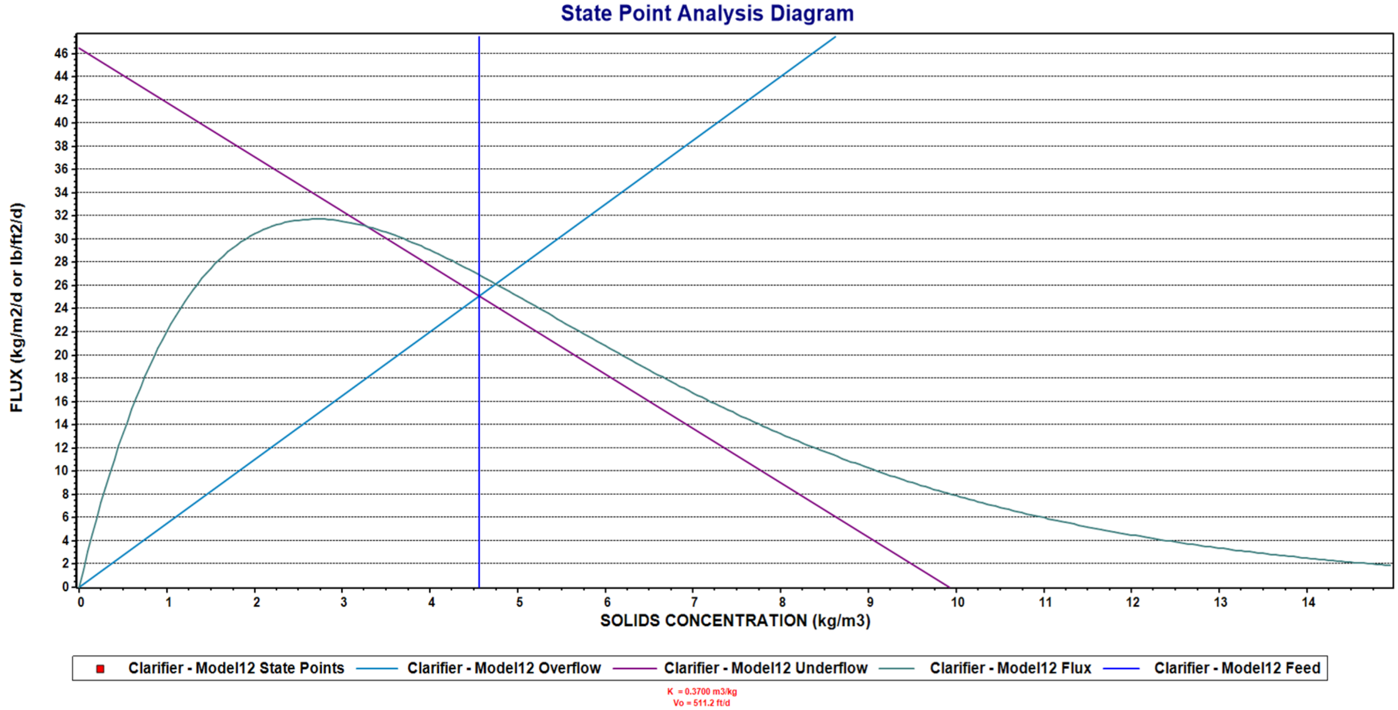


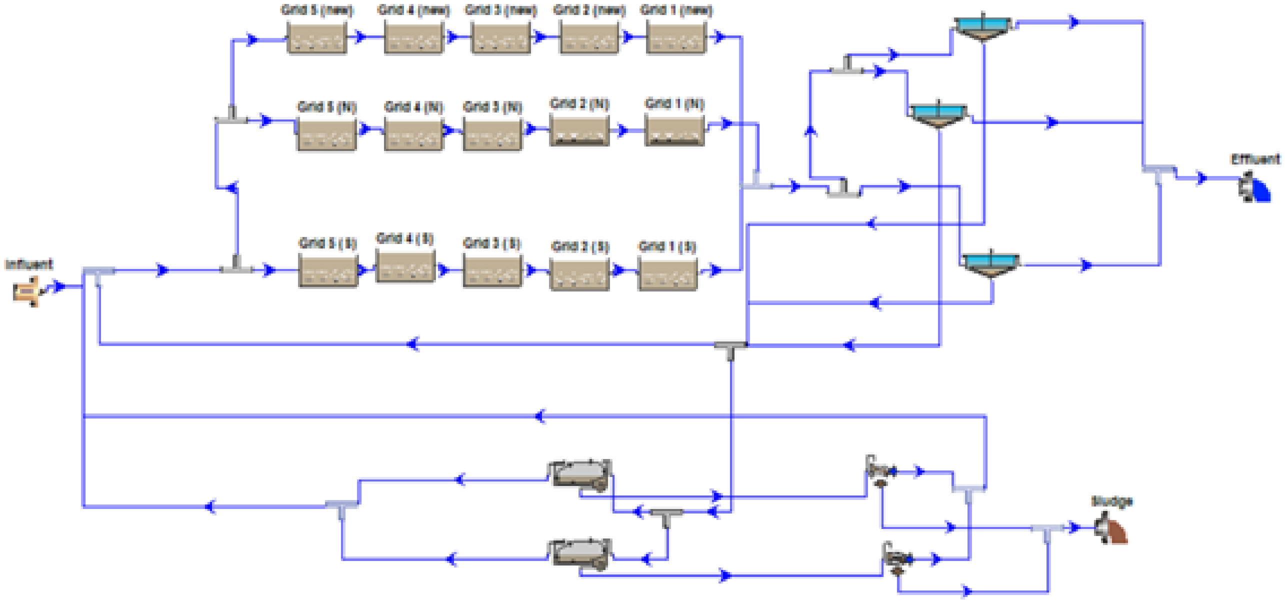
*Figure 3.8 – SPA Diagram, 2 Clarifiers (2040 Max Month Loading = 8.39 MGD, Winter)*




### 3.5.1.3 *2040 Max Month Loading to Three Trains (2 Existing + 1 New) and Two Clarifiers (2 Existing)*

The BioWin model was modified to include the addition of a third identical train and a third identical clarifier to evaluate addressing the limitations noted with the 2040 max month loading conditions. The outline of the updated BioWin model is depicted in **Figure 3.9**.

*Figure 3.9 – City of Hendersonville WWTF BioWin Model (3 Trains + 3 Clarifiers)*




The initial simulation utilizing the updated model included evaluating the addition of one additional extended aeration train to the existing system. To accommodate this, one of the three clarifiers within the updated BioWin model was turned off by adjusting flow splitting within the model. The model was run to simulate the above outlined winter 2040 max month loading conditions to confirm the addition of a third train will address secondary capacity limitations. The simulation results are outlined in **Table 3.13** below.

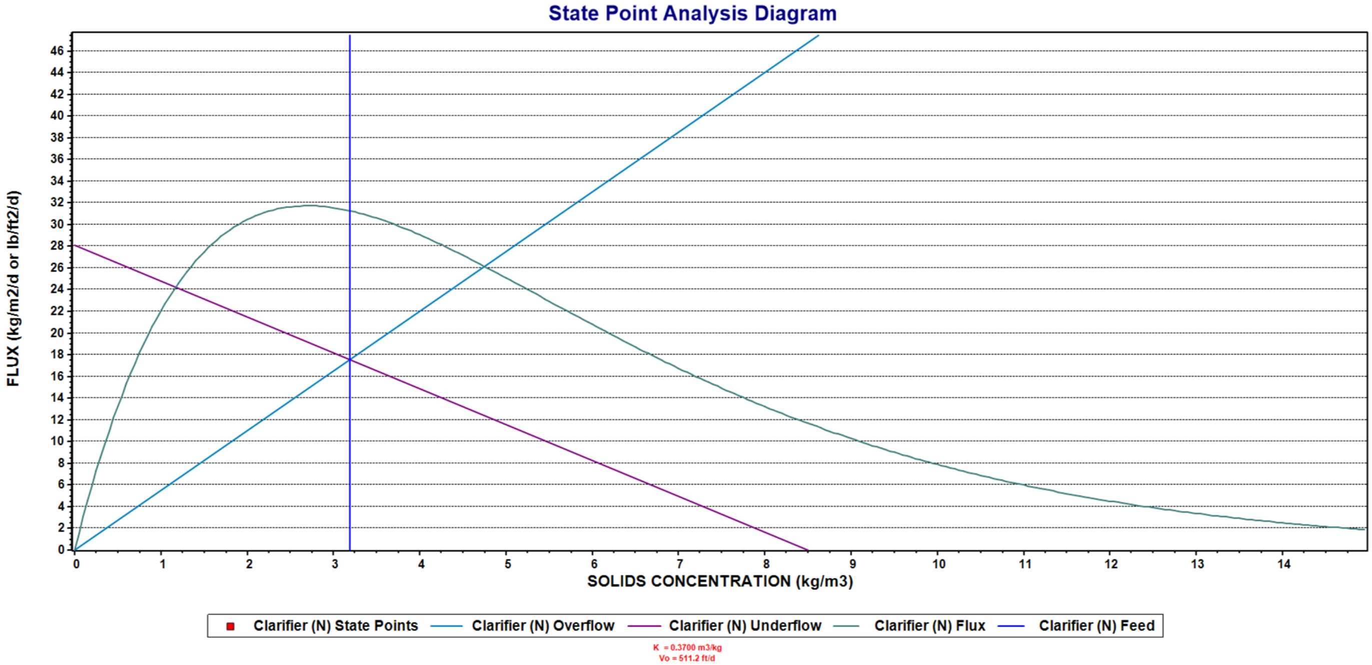
*Table 3.13 – BioWin Simulation (3 Trains + 2 Clarifiers) Results Summary (2040 Max Month Loading, 8.39 MGD)*

| Parameter | Winter (10 ºC) |
|---|---|
| SRT (Days) | 14 |
| HRT (Hrs) | 19.9 |
| MLSS (mg/L) | 3,191 |
| MLVSS (mg/L) | 1,948 |

The BioWin simulation results indicated that the addition of a third extended aeration train addressed capacity limitations observed with the existing two train system. At the 2040 max month loading conditions, the three train system was able to meet the winter design SRT requirement of 13.8 days without exceeding the recommended MLSS concentration. At the 2040 max month loading conditions simulated, the MLSS was estimated to be 3,191 mg/L. Therefore, it will be appropriate to add an additional train to meet 2040 max month loading conditions.

Evaluation of the two existing secondary clarifiers was performed as part of this analysis. The SPA diagram is depicted in **Figure 3.10**. The SPA diagram indicates that the existing two clarifiers will handle the 2040 max month loading condition as simulated with the addition of a third train. The observed improvement in clarifier performance is the result of the decreased operating MLSS and resulting lower solids loading rate to the clarifiers after the addition of the third train.

*Figure 3.10 – SPA Diagram, 2 Clarifiers (2040 Max Month Loading = 8.39 MGD, Winter)*




### 3.5.1.4 *2040 Max Day Loading to Three Trains (2 Existing + 1 New) and Two Clarifiers (2 Existing)*

The three train and two clarifier system was evaluated at the projected 2040 max day loading conditions utilizing the updated BioWin model. A summary of the simulation results is indicated below in **Table 3.14**.

*Table 3.14 – BioWin Simulation (3 Trains + 2 Clarifiers) Results Summary (2040 Max Day Loading, 10.65 MGD)*

| Parameter | Summer (20 °C) | Winter (10 °C) |
|---|---|---|
| SRT (Days) | 11.95 | 11.05 < 13.8[1] |
| HRT (Hrs) | 15.7 | 15.7 |
| MLSS (mg/L) | 3,374 | 3,351 |
| MLVSS (mg/L) | 2,034 | 2,088 |

*[1]Minimum design winter SRT of 13.8 days*

Based on the results from the BioWin simulation, it was determined that the three extended aeration trains will provide adequate treatment capacity to meet the 2040 max day loading condition without any observed capacity or treatment limitations. However, under max day loading, clarifier capacity becomes a concern requiring additional simulation.

To ensure the secondary clarifiers are not overloaded at the simulated operating conditions, SPA was performed to confirm clarifier performance at the 2040 max day loading conditions did not negatively impact effluent quality. The resulting SPA diagrams are included below in **Figure 3.11** and **Figure 3.12**. The SPA diagrams indicated that under the simulated 2040 max day loading conditions the existing two clarifiers will remain underloaded while maintaining an MLSS concentration approximately equal to the 2040 max month simulated conditions. As noted in **Table 3.14**, the operating SRT simulated for 2040 winter conditions was allowed to drop below the design SRT as operating at the design SRT resulted in an excessive solids loading rate to the existing clarifiers and compromised performance. It is anticipated that operating at an SRT below the design SRT for short periods (such as max day) will not have a significant impact on secondary treatment performance of the extended aeration process. The simulation performed at the lower operating SRT at 10 °C did not result in an observed loss of nitrification indicative of nitrifier washout occurring. Therefore, to maintain good clarifier performance under 2040 max day winter loading conditions, it will become necessary to waste additional biomass from the system to maintain a reliable operating MLSS concentration until the max day loading condition has subsided.

*Figure 3.11 – SPA Diagram, 2 Clarifiers (2040 Max Day Loading = 10.65 MGD, Summer)*

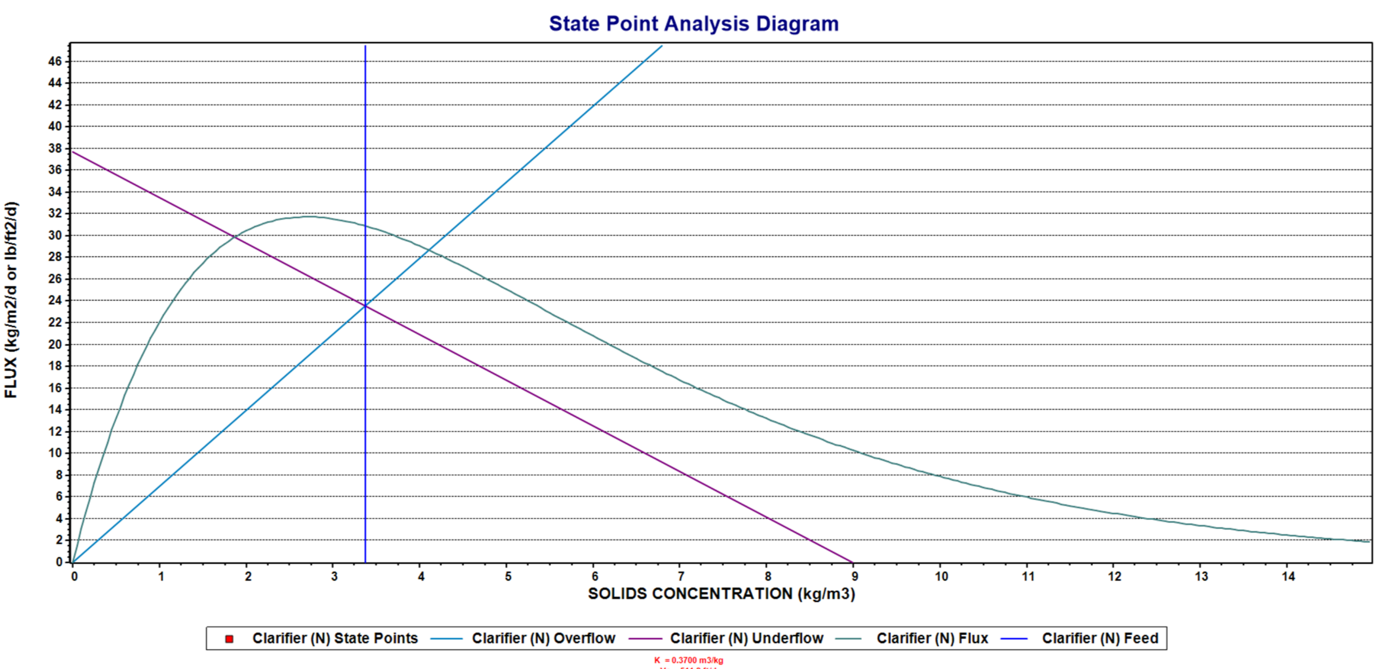
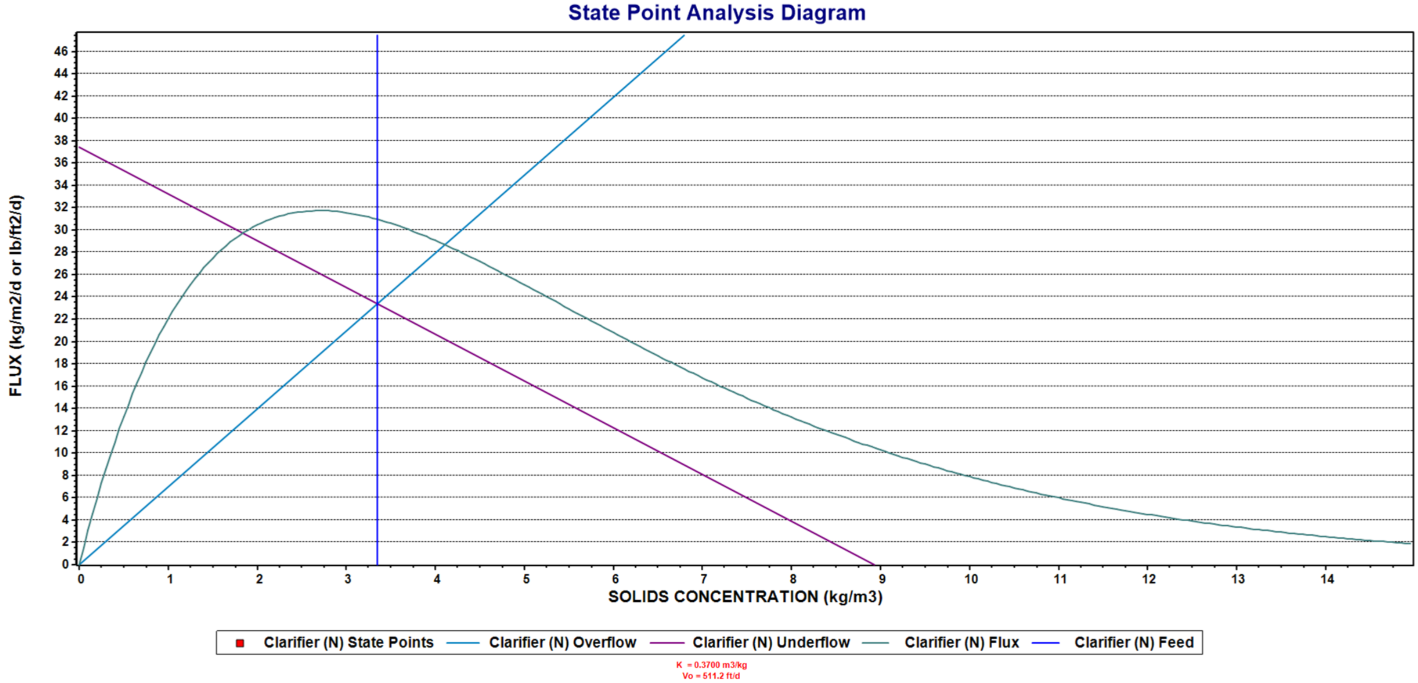


*Figure 3.12 – SPA Diagram, 2 Clarifiers (2040 Max Day Loading = 10.65 MGD, Winter)*




## 3.5.2 Alternative 1(b): Modified Ludzack-Ettinger (MLE) Process/Secondary Clarification

As noted in the previous sections, the City currently operates Grid 5 within each of the extended aeration trains at a low dissolved oxygen (DO) level to achieve simultaneous nitrification/denitrification to recover alkalinity for pH stabilization of the secondary process. However, this mode of operation is not as efficient as it could be and is not recommended as a method to achieve significant nitrogen removal. System denitrification efficiency and alkalinity recovery rates can be significantly improved by converting the existing extended aeration process to an MLE process by adding a dedicated anoxic zone at the front of the extended aeration process. This can most readily be accomplished by converting Grid 5 to a dedicated anoxic zone with mechanical mixing and including an internal recycle pump at the end of the extended aeration train. The addition of the internal recycle will allow recycling nitrate rich wastewater back to the anoxic zone for improved denitrification performance. This would allow for significant TN removal to meet potential future TN limits. However, enhanced biological phosphorus removal will not be achieved in this arrangement. To achieve TP removal, chemical precipitation can be performed. Chemical phosphorus removal has not been evaluated as part of this study. As noted in the previous extended aeration evaluation, it will be necessary to provide three extended aeration trains to meet the design aerobic SRT.

#### 3.5.2.1 *2040 Max Month Loading to Three Trains (2 existing + 1 new) and Two Clarifiers (2 existing)*

System performance was evaluated at the design 2040 Max Month loading condition. The updated BioWin model is shown below in **Figure 3.13**. The model includes three trains as noted above to meet 2040 loading conditions and three clarifiers. For this simulation, flow splitting was adjusted to have two clarifiers in service during the simulations to evaluate the two existing clarifiers. Modeling results indicated no observed limitations in meeting the design aerobic SRT and MLSS concentrations as outlined in **Table 3.15** below. In addition, the secondary clarifiers exhibited no observed performance issues as shown by the SPA diagrams in **Figure 3.14** and **Figure 3.15**.

*Figure 3.13 – City of Hendersonville WWTF BioWin Model (Three MLE Trains + Three Clarifiers)*

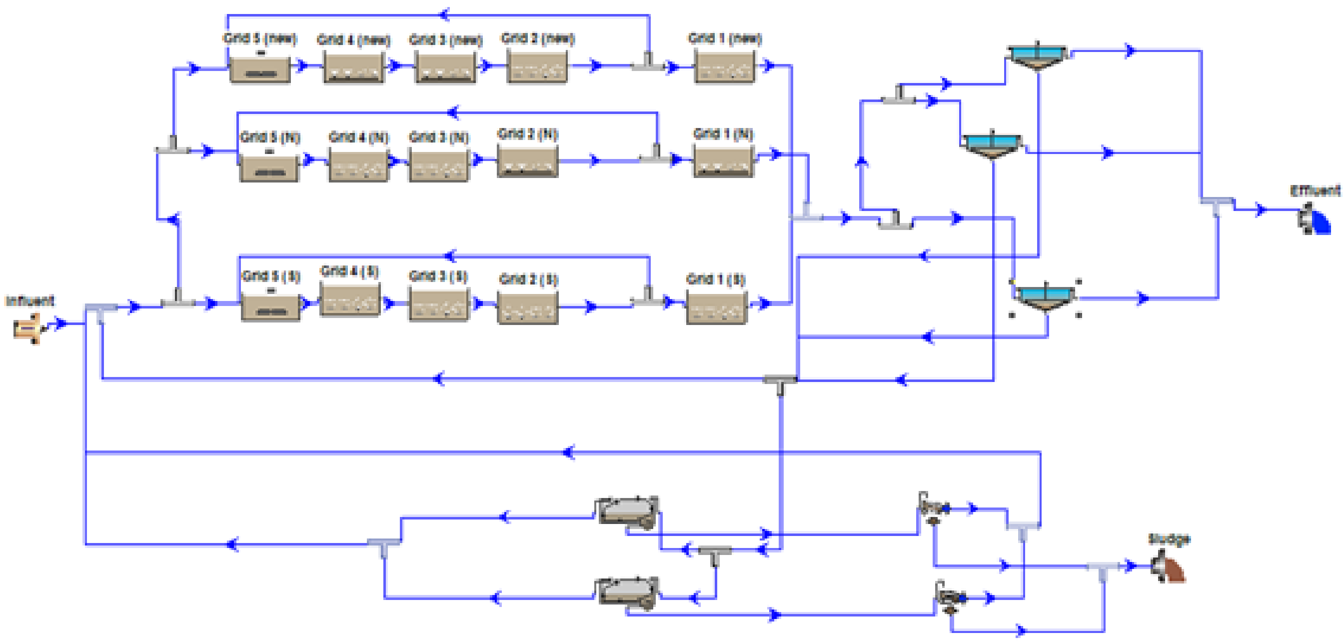



*Table 3.15 – BioWin Simulation (3 MLE Trains + 2 Clarifiers) Results Summary (2040 Max Month Loading, 8.39 MGD)*

| Parameter | Summer (20 °C) | Winter (10 °C) |
|---|---|---|
| Aerobic SRT (Days) | 11.94 | 13.83 |
| HRT (Hrs) | 19.9 | 19.9 |
| MLSS (mg/L) | 3,233 | 3,908 |
| MLVSS (mg/L) | 1,921 | 2,342 |

*Figure 3.14 – SPA Diagram, 2 Clarifiers (2040 Max Month Loading = 8.39 MGD, Summer)*

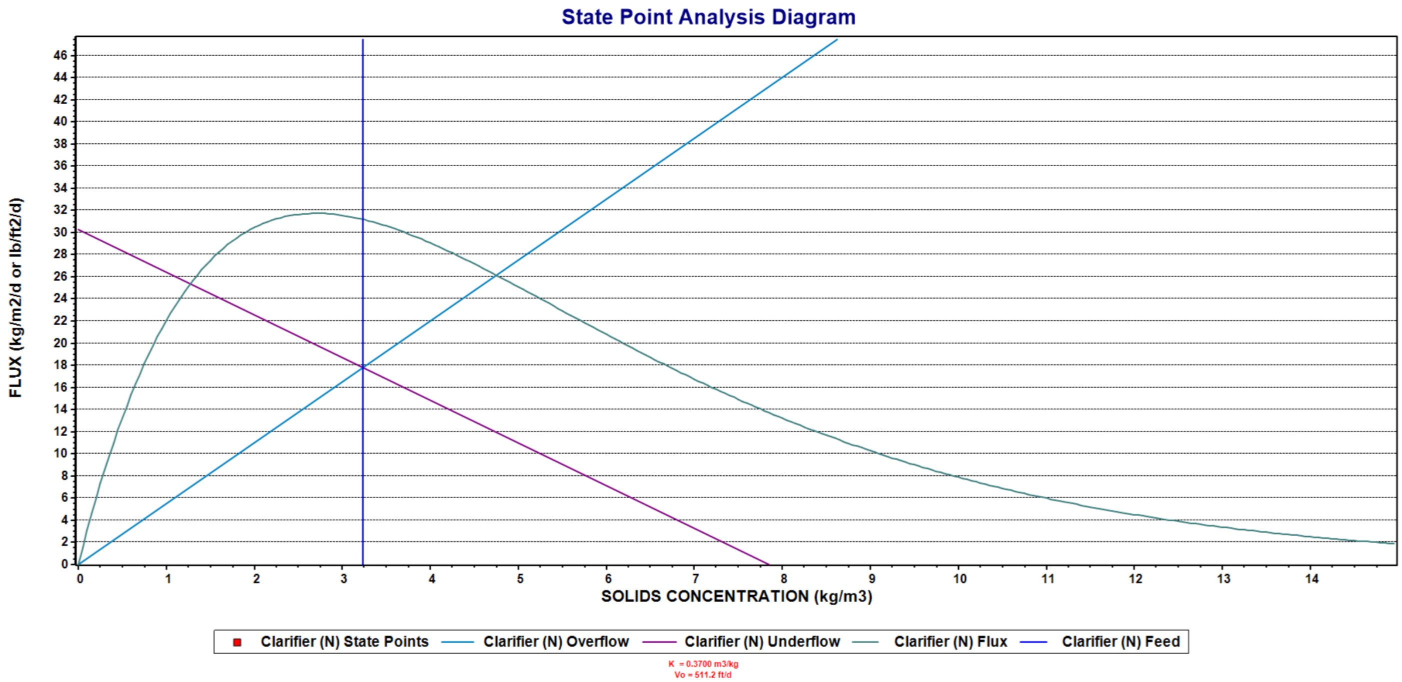
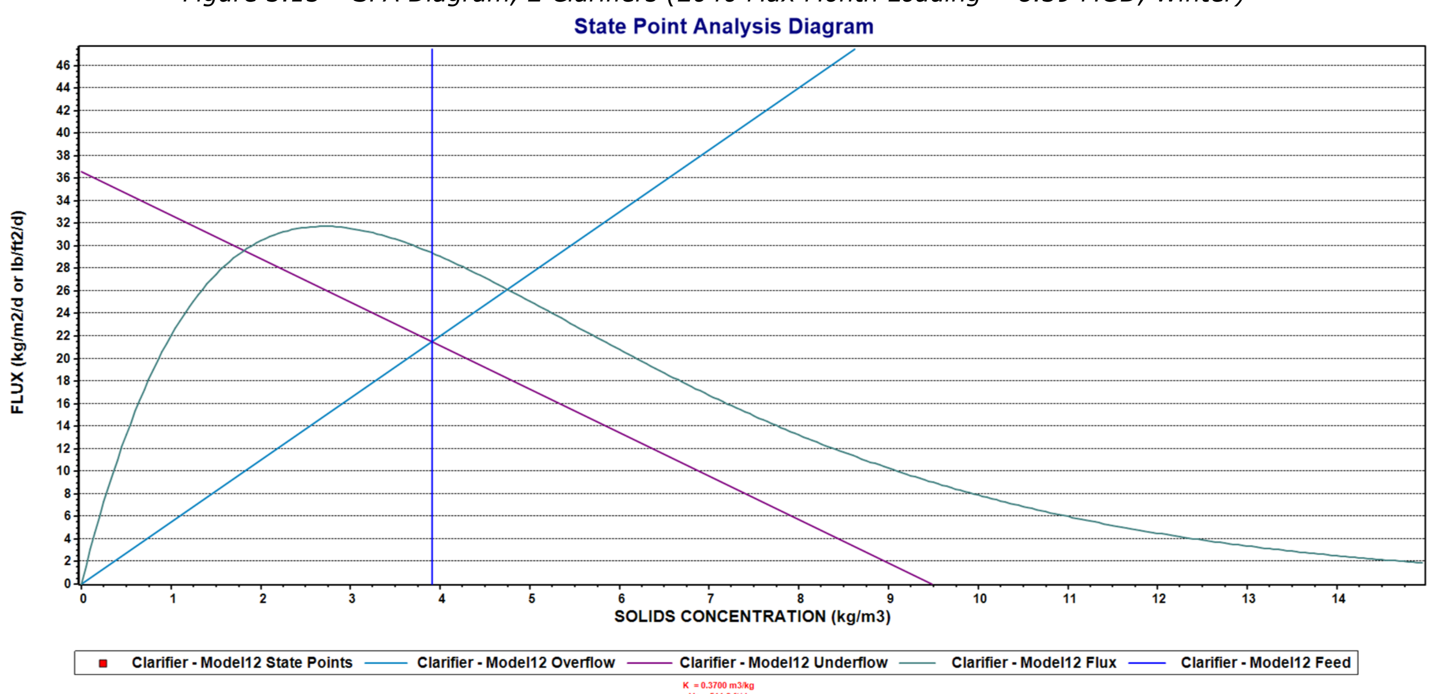



*Figure 3.15 – SPA Diagram, 2 Clarifiers (2040 Max Month Loading = 8.39 MGD, Winter)*




The addition of the anoxic selector and internal recycle resulted in a significant increase in the simulated amount of denitrification achieved by the system. Preliminary simulation results indicated that TN limits in the range of <12 mg/L may be achievable by converting to an MLE process resulting in TN removal rates approximately 30% higher compared to the extended aeration process alone. Furthermore, the simulation results indicated an approximately 10% decrease in aeration demands when compared to the extended aeration process without the dedicated anoxic zone.

### 3.5.2.2 *2040 Max Day Loading to Three Trains (2 existing + 1 new) and Two Clarifiers (2 existing)*

System performance was evaluated at the design 2040 max day loading condition. The same BioWin model utilized for the max month loading and presented in **Figure 3.13** was utilized to simulate max day loading. The model includes three trains as noted above to meet 2040 loading conditions and three clarifiers. For this simulation of the existing two clarifiers only, flow splitting was adjusted to have two clarifiers in service during these simulations.

It was necessary to decrease the simulated aerobic SRT below the design SRT to ensure overloading of the clarifiers did not occur. Simulated operating conditions are outlined in **Table 3.16** below. Under the summer operating conditions, the simulated aerobic SRT was 9.24 days which is significantly lower than the 11.9 day design aerobic SRT. The simulation results showed no observed loss of nitrification. Therefore, operating for an intermittent period (max day) should not result in a significant decrease in treatment performance of the biological treatment system. The secondary clarifiers exhibited no observed performance issues under the summer operations as shown by the SPA diagram in **Figure 3.16**.

Under the winter operating conditions, the simulated aerobic SRT was 10.48 days which is significantly lower than the 13.8 day design aerobic SRT. The operational MLSS under max day conditions was maintained to be approximately equal to the operational MLSS simulated for the 2040 max month loading condition. No significant loss of nitrification was observed at the max day loading condition. The corresponding SPA diagram is depicted below in **Figure 3.17**. The SPA indicates that the existing secondary clarifiers become critically loaded under the 2040 max day loading condition and are likely to become overloaded due to diurnal loading rates. Therefore, it is recommended that an additional clarifier be included to meet 2040 max day loading conditions.

*Table 3.16 – BioWin Simulation (3 MLE Trains + 2 Clarifiers) Results Summary (2040 Max Day Loading, 10.65 MGD)*

| Parameter | Summer (20 °C) | Winter (10 °C) |
|---|---|---|
| Aerobic SRT (Days) | 9.24 <11.9 | 10.48 <13.8 |
| HRT (Hrs) | 15.7 | 15.7 |
| MLSS (mg/L) | 3,326 | 3,961 |
| MLVSS (mg/L) | 2,021 | 2,424 |

*Figure 3.16 – SPA Diagram, 2 Clarifiers (2040 Max Day Loading = 10.65 MGD, Summer)*

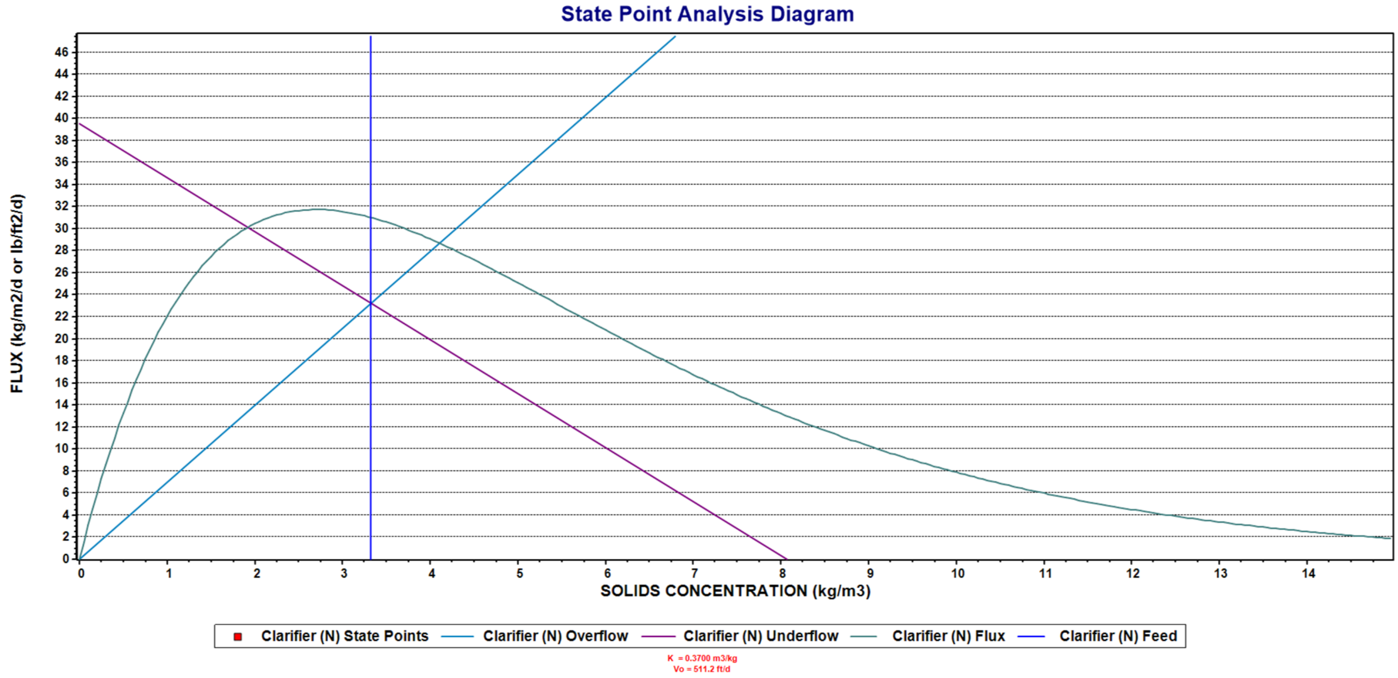
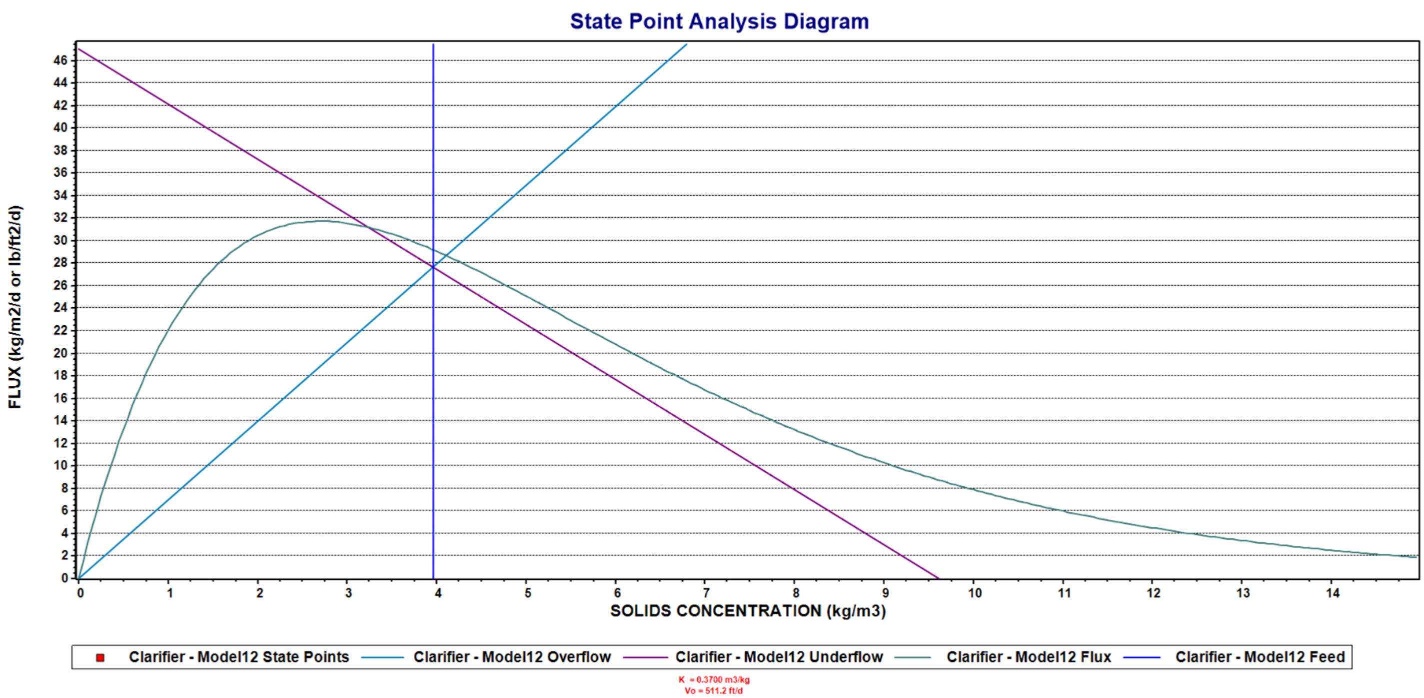



*Figure 3.17 – SPA Diagram, 2 Clarifiers (2040 Max Day Loading = 10.65 MGD, Winter)*



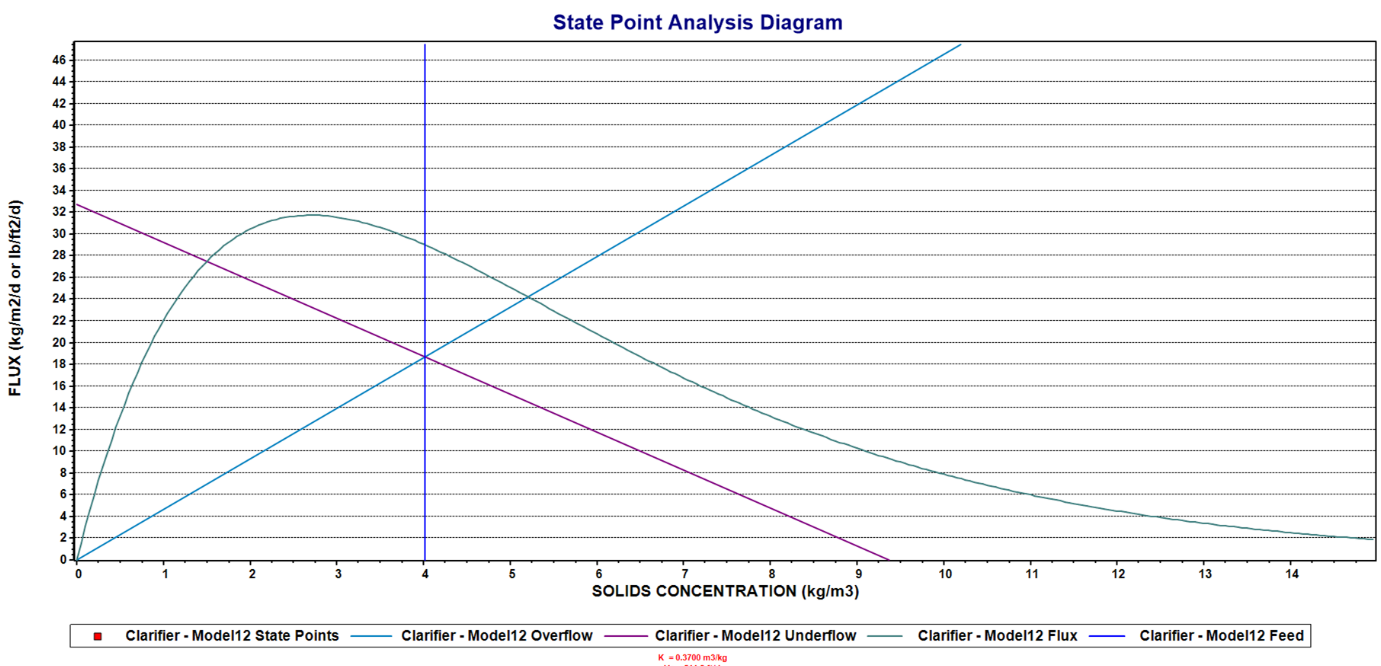
### 3.5.2.3 *2040 Max Day Loading to Three Trains (2 existing + 1 new) and Three Clarifiers (2 existing + 1 new)*

To address capacity limitations associated with the existing clarifiers, the BioWin model flow splitting was updated to include the addition of a third clarifier when simulating the 2040 max day winter loading condition utilizing the model shown in **Figure 3.13**. Operational conditions are outlined in **Table 3.17** below. The associated SPA diagram is included in **Figure 3.18** below. As noted in the below operational conditions, the simulated aerobic SRT of 10.67 days dropped below the design aerobic SRT of 13.8 days. The 2040 max day operational MLSS concentration was maintained to be close to the max month simulated MLSS concentration. Under these simulated operating conditions, no significant decrease in nitrification performance was observed. The SPA diagram indicates that the addition of the third clarifier addressed previously observed capacity issues with the existing clarifiers. Therefore, it is further confirmed that upgrade of the existing biological process from an extended aeration system to an MLE process will require the addition of a third train and a third clarifier.

*Table 3.17 – BioWin Simulation (3 MLE Trains + 3 Clarifiers) Results Summary (2040 Max Day Loading, 10.65 MGD)*

| Parameter | Winter (10 ºC) |
|---|---|
| Aerobic SRT (Days) | 10.67 <13.8 |
| HRT (Hrs) | 15.7 |
| MLSS (mg/L) | 4,017 |
| MLVSS (mg/L) | 2,457 |

*Figure 3.18 – SPA Diagram, 2 Clarifiers (2040 Max Day Loading = 10.65 MGD, Winter)*



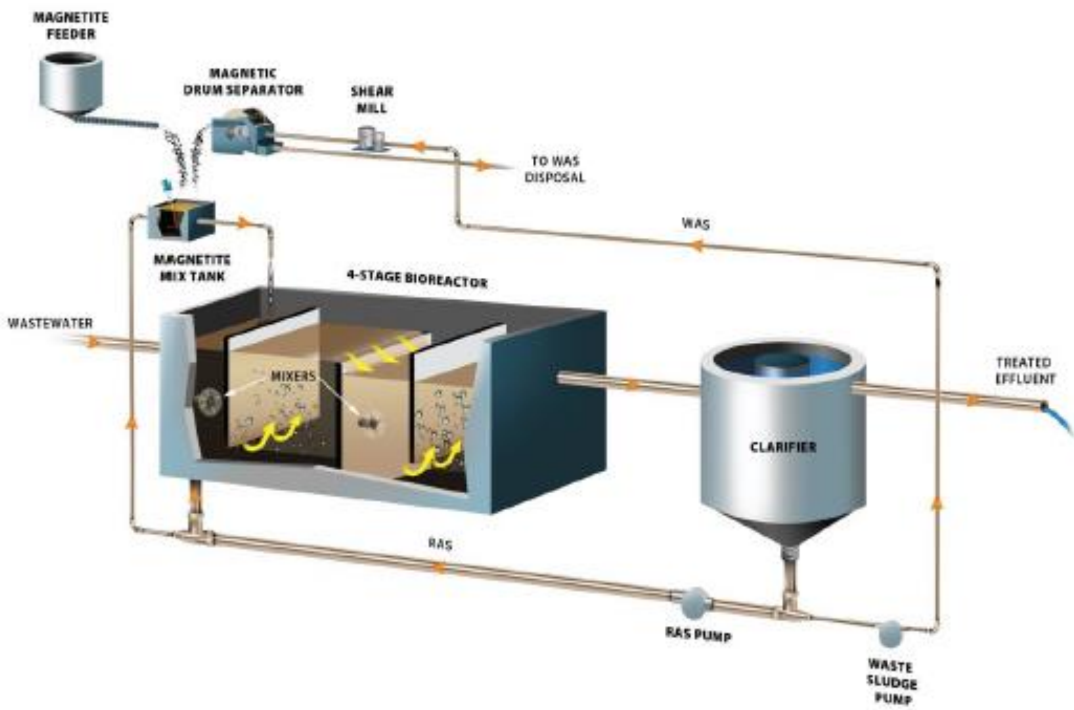
### 3.5.3 Process Intensification Alternatives

As technologies have continued to improve, commercially available intensification processes have become more prevalent in full scale applications as a means to increase treatment capacity within existing biological treatment processes. These systems are primarily employed to treat higher loading rates within a smaller footprint where land availability may be limited. As outlined in the above sections, expansion of the existing facility to meet projected 2040 loading condition utilizing traditional activated sludge technologies will require a significant increase in the facility footprint including the addition of a third train and third clarifier. Usable land area at the WWTF is limited and will require significant filling of adjacent wetlands to accommodate the larger footprint. As an alternative, process intensification has been considered as part of this evaluation. Technologies evaluated include ballasted activated sludge (BioMag) and Integrated Fixed Film Activated Sludge (IFAS) as outlined in the following sections.

### 3.5.4 Alternative 2: BioMag Process/Secondary Clarification

Ballasted activated sludge utilizes a magnetite media of high density to augment the flocculated biomass within the activated sludge process. The magnetite media is readily assimilated into the biomass and acts to significantly increase the density of the biomass resulting in an increase in the settling velocity of the biomass. In general, the biomass floc has an approximate specific gravity of 1.03 which is only slightly greater than water. Operationally, this results in a slow settling process within the secondary clarifiers. The magnetite media has a specific gravity of 5.2 which significantly increases the specific gravity of the biomass during operation to enhance settling velocity. An outline of the BioMag process is shown in **Figure 3.19** below.

*Figure 3.19 – BioMag Process Flow Diagram*




*(Evoqua Water Technologies)*

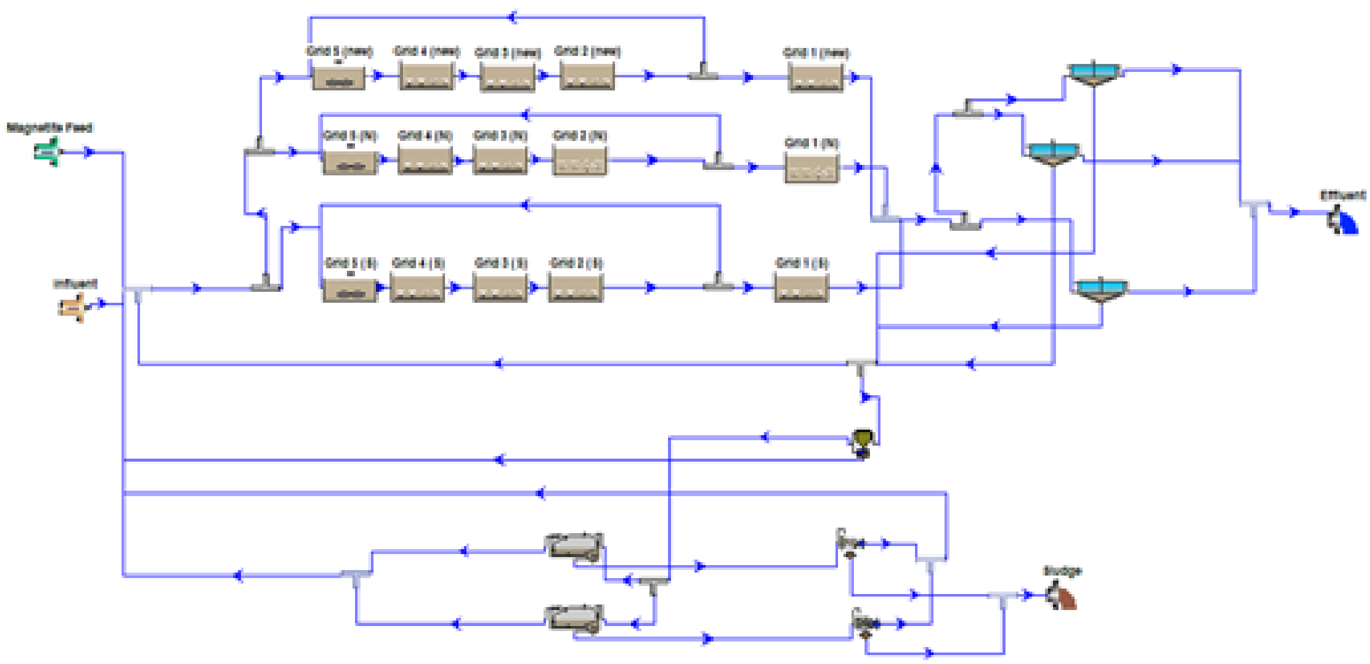
BioMag is fed to the secondary process through blending with the RAS stream at the front of the secondary treatment process within a dedicated reactor utilizing a magnetite feed system. Magnetite is blended with the MLSS at a rate of approximately 1:1 within the secondary train. As RAS flows are pulled from the bottom of the secondary clarifiers, much of the magnetite is sent back to the head of the treatment process. Some magnetite is wasted from the process as part of the WAS stream. Additional magnetite separation equipment is installed prior to the WWTF's solids handling facilities to separate the magnetite from the rest of the WAS stream. The WAS stream is run through a shear mill to separate the magnetite media from the WAS. After the shear mill, the magnetite is collected on a magnetic drum separator and the remainder of the WAS stream continues to the solids handling facilities at the WWTF. The magnetite separation process has a high collection efficiency, usually greater than 95%. The collected magnetite media can be fed back to the secondary process along with a small stream of new media to replace the media lost as part of collection inefficiencies.

In general, the increased settling performance of the BioMag process has been marketed as a technology to significantly increase capacity of existing secondary clarifiers by a factor of 2 or more in some applications. While BioMag is typically recommended primarily to increase clarifier capacity, the observed increase in clarifier capacity has also been marketed as a technology to increase the capacity of the existing biological treatment system as a result of the ability to increase the operating MLSS concentration. BioMag has also been utilized to allow for the addition of BNR within existing systems without the need for additional tanks. As noted in the previous sections, the existing two train extended aeration process would become exceeded under the 2040 max month loading condition and require one additional extended aeration train to meet loading conditions. In addition, when converting to the extended aeration process to an MLE process a third MLE train and third clarifier would become required.

As reported in the previous section, the evaluation of the use of conventional activated sludge was considered with a maximum MLSS concentration of 4,500 mg/L. Published literature indicates that the BioMag process can be employed to achieve an MLSS up to 10,000 mg/L. However, as MLSS concentrations exceed 7,500 mg/L complications are likely to be observed with respect to oxygen transfer efficiency, RAS/WAS pumping, reactor mixing and treatment reliability. Therefore, it is not recommended that BioMag be utilized to achieve a design MLSS concentrations significantly above 7,500 mg/L. The magnetite media constitutes a significant fraction of the MLSS and does not provide any treatment capacity. When operating at a 1:1 mass ratio between the MLSS and magnetite media concentrations, this would limit the maximum mixed liquor concentration to approximately 3,750 mg/L with an additional 3,750 mg/L of magnetite for a combined MLSS of 7,500 mg/L. This mixed liquor concentration of 3,750 mg/L is significantly lower than the 4,500 mg/L limit recommended for conventional activated sludge indicating that the use of BioMag will decrease the biological treatment capacity of the previously evaluated three train system when compared to traditional activated sludge processes. Therefore, BioMag is not a feasible means to eliminate the third train. As BioMag has the potential to reduce treatment capacity within the three train system under ideal conditions, it is necessary to further consider the impacts of BioMag operations on the three train system. The worst case design condition for the secondary trains evaluated above occurred for the MLE process at the 2040 max month winter loading.

To investigate the performance of BioMag on system operations at 2040 loading conditions, the previously developed BioWin model was updated to simulate the addition of BioMag to the MLE process. This was primarily achieved by adjusting settling parameters within the clarifier model to capture the improved settling performance provided by the magnetite media, implementing a state variable feed to represent magnetite feed and implementing a selective cyclone model to simulate the magnetite recovery process. The updated BioWin model is shown below in **Figure 3.20**.

*Figure 3.20 – City of Hendersonville WWTF BioWin Model (Three MLE Trains + Three Clarifiers with BioMag Addition)*




To consider how the use of BioMag would impact capacity of the three train system, an additional simulation was run at the 2040 max month winter loading condition of the three train system. Flow splitting was set up to have two clarifiers in service during the simulation. The simulation targeted meeting a design SRT of 27.6 days to account for the additional magnetite solids within the system at a 1:1 ratio and to limit MLSS concentration within the trains. A summary of the operational parameters is shown below in **Table 3.18**.

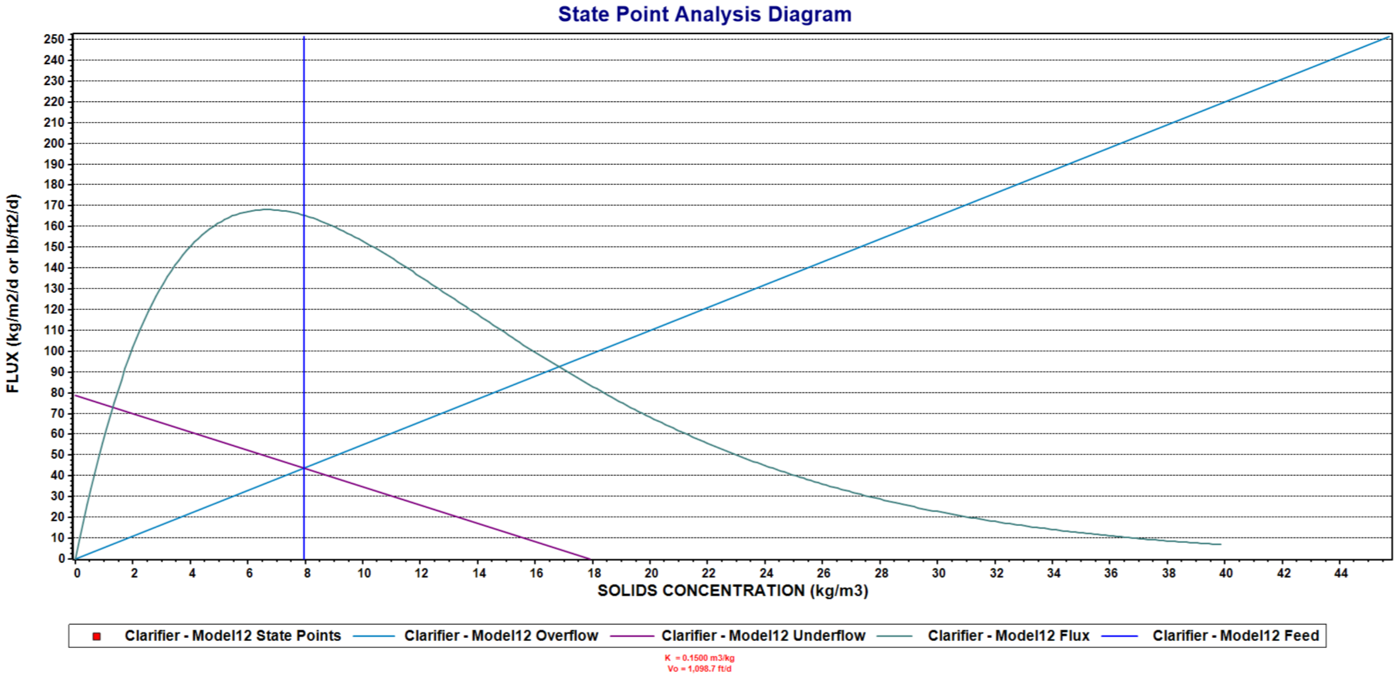
*Table 3.18 – BioWin Simulation (3 MLE Trains + 2 Clarifiers with BioMag) Results Summary (2040 Max Month Loading, 8.39 MGD)*

| Parameter | Winter (10 °C) |
|---|---|
| Aerobic SRT (Days) | 27.7 |
| HRT (Hrs) | 19.9 |
| MLSS (mg/L) | 7,963 |
| Magnetite (mg/L) | 4,022 |
| MLVSS (mg/L) | 2,367 |

Based on the operational conditions outlined, to meet the design aerobic SRT within the three train system utilizing BioMag, it will be necessary to increase the MLSS concentration close to 8,000 mg/L. The increased operating MLSS will result in additional operational concerns associated with reactor mixing, pumping, aeration requirements and overall treatment process reliability.

Although BioMag does not eliminate the need for a third train, the improved settling characteristics were noted in the 2040 max month winter evaluation as can be seen in **Figure 3.21** below when compared to the SPA prepared for the MLE process in **Figure 3.15** above. Settling performance within the two existing clarifiers indicated that reliable solids separation at the higher loading rate could be achieved by converting to a BioMag process.

*Figure 3.21 – SPA Diagram, 2 Clarifiers (2040 Max Month Loading = 8.39 MGD, Winter)*



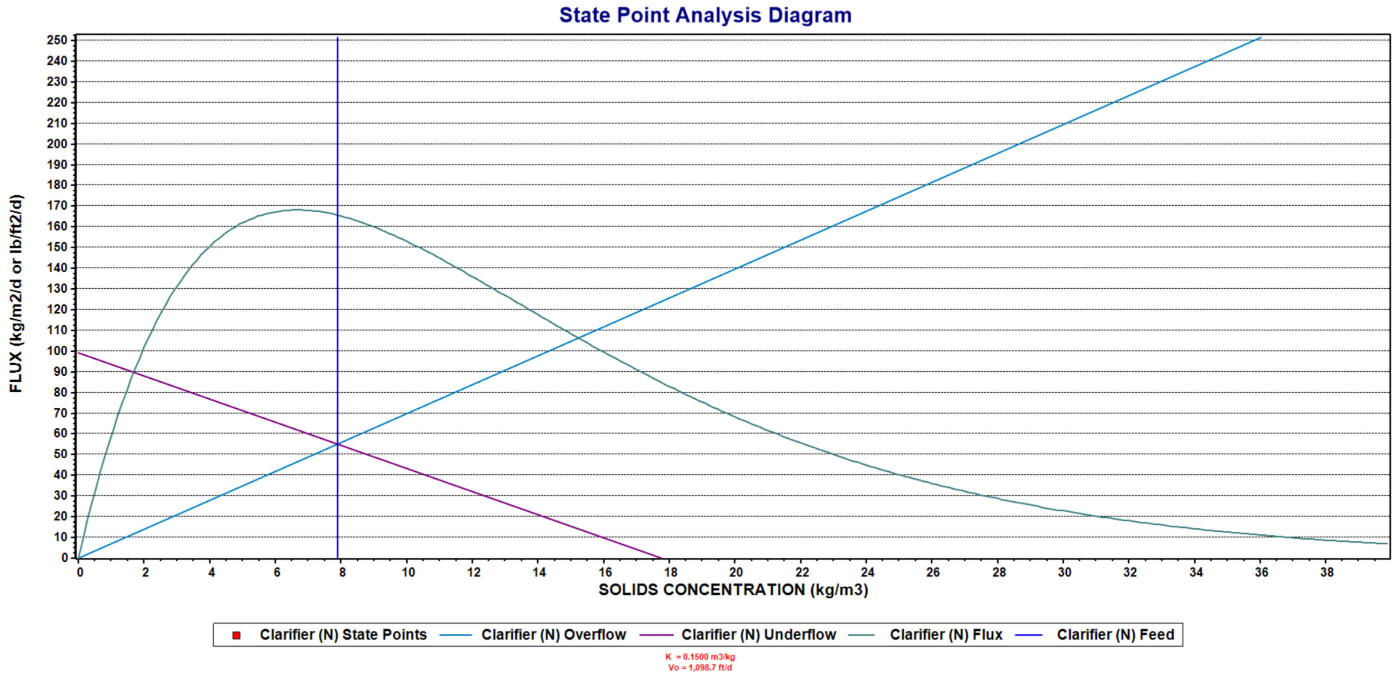

Based on the observed improvements in settling performance in the 2040 max month winter simulation, it is likely that the enhanced settling performance would eliminate the need for an additional third clarifier under the 2040 max day winter loading condition when converting to the MLE process. Therefore, the 2040 max day winter loading condition was simulated utilizing the developed BioWin model of the MLE process with BioMag outlined in this section. The simulation was modeled to maintain an MLSS concentration of approximately 7,900 mg/L which is consistent with the simulated 2040 max month winter simulation. Simulated operating conditions are outlined in **Table 3.19** below. Simulated conditions indicate that the operating aerobic SRT will drop below the design SRT. However, due to the short duration of the max day loading condition, it is anticipated the treatment process would recover quickly after peak loading conditions subside. The associated SPA diagram for the 2040 max day loading simulation is included in **Figure 3.22** below. Based on the SPA diagram, the existing two clarifiers should have sufficient capacity to manage the estimated 2040 loading conditions without requiring an additional third clarifier.

*Table 3.19 – BioWin Simulation (3 Extended Aeration Trains with BioMag + 2 Clarifiers) Results Summary (2040 Max Day Loading, 10.65 MGD)*

| Parameter | Winter (10 °C) |
|---|---|
| SRT (Days) | 20.1 <27.6[1] |
| HRT (Hrs) | 15.7 |
| MLSS (mg/L) | 7,898 |
| Magnetite (mg/L) | 3,980 |
| MLVSS (mg/L) | 2,391 |

*[1]BioMag system minimum design winter SRT of 27.6 days (including inert magnetite)*

*Figure 3.22 – SPA Diagram, 2 Clarifiers with BioMag (2040 Max Day Loading = 10.65 MGD, Winter)*

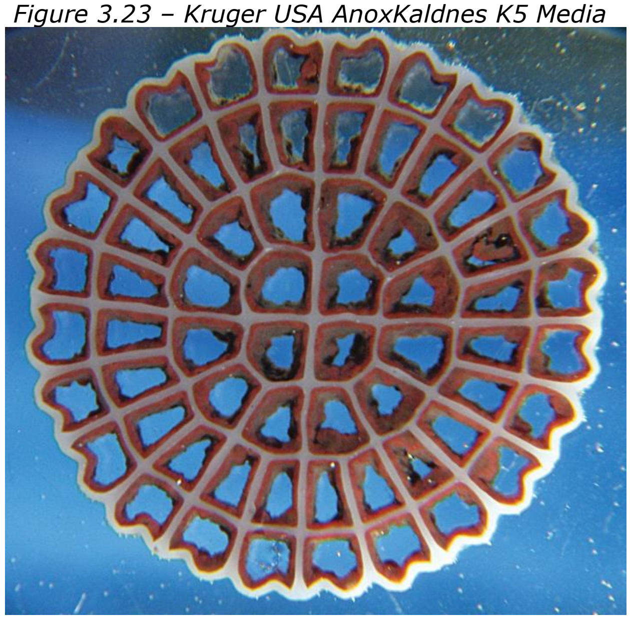



Based on the simulations outlined in this section, BioMag appears to be a technically feasible alternative to meet 2040 loading conditions at the Hendersonville WWTF. However, it will require the addition of a third treatment train to meet treatment needs. Due to the additional capital equipment costs and operational costs associated with the BioMag system with the only significant benefit being the elimination of one additional secondary clarifier, it is not recommended that conversion of the existing process to a ballasted activated sludge process be considered further.

### 3.5.5 Alternative 3: Integrated Fixed Film Activated Sludge (IFAS Process)/Secondary Clarification

Integrated Fixed Film Activated Sludge (IFAS) processes utilize supplemental support media within the secondary process to support the growth of fixed film biomass. The media can be fixed in place (such as sheet plastic trickling filter media) or be free to circulate in the bioreactor (such as net-zero buoyancy plastic media). An example of floating media is included in **Figure 3.23** below.

*Figure 3.23 – Kruger USA AnoxKaldnes K5 Media*



The fixed film biomass is maintained within the secondary treatment process by ensuring the media remains within the reactors while the media is retained within the secondary process by remaining in a fixed location or by physical screening prior to leaving the secondary process. The IFAS process can increase the total mass of biomass within the system by allowing a significant fraction of the biomass to be present within the fixed film thus decreasing the suspended biomass which must be separated by gravity separation within the secondary clarifiers. Benefits of the IFAS process include:

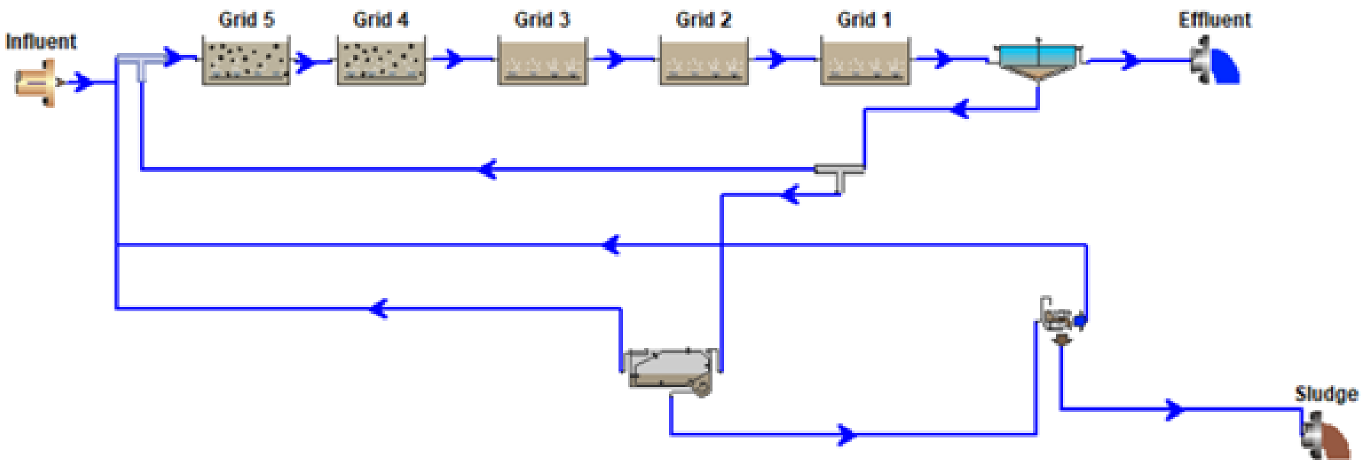
- Stable nitrification even with a more limited suspended growth SRT
- Addition of denitrification within aerobic systems
- Utilizes traditional wastewater treatment equipment
- Suspended biomass provides excellent removal of colloidal and particulate substrate

Drawbacks of the IFAS process include:

- Requires the use of separate liquid-solids separation for media
- Oxygen and mixing requirements are higher than suspended growth processes
- Loading rates are higher than purely suspended growth systems but lower than other attached growth systems
- Process design basis not well established
- Limited full-scale application compared to traditional activated sludge processes

In general, this evaluation considers the addition of net-zero buoyancy media to the existing extended aeration trains within grids 5 and 4. The simulation was performed with 50% of the grid volumes filled with media, which is approaching the upper limit of recommended design range. An updated BioWin model is indicated below in **Figure 3.24**.

*Figure 3.24 – City of Hendersonville WWTF BioWin Model (Two Extended Aeration Trains + Two Clarifiers with IFAS Addition)*




Additional solids separation will be necessary between grids 4 and 3 to ensure media migration does not occur. Aeration and mixing were provided by the aeration system to all grids of the train to ensure complete mixing requirements are maintained. Therefore, the suspended growth phase of the extended aeration train was maintained within fully aerobic conditions. Anoxic/anaerobic conditions were maintained within the fixed film biomass to provide denitrification for alkalinity recovery within the secondary process. In general, the DO levels observed during modeling within the grids ranged from 1.25 mg/L (grid 5) to 3 mg/L (grid 1) which is significantly higher than the modeled extended aeration results above. Therefore, the design aerobic SRT was adjusted for the IFAS process to account for the higher DO levels. Calculated aerobic SRT requirements were determined to be 8.3 and 10.9 days at 20 °C and 10 °C, respectively. Modeling calculated the aerobic SRT based on the suspended biomass only. Fixed film biomass was assumed to be completely anoxic/anaerobic. To evaluate capacity of the existing two train system, additional BioWin simulations were run at 2040 max month loading conditions. Modeling results are outlined in **Table 3.20** below.

*Table 3.20 – BioWin Simulation (2 Extended Aeration Trains + 2 Clarifiers with IFAS addition) Results Summary (2040 Max Month Loading, 8.39 MGD)*

| Parameter | Summer (20 °C) | Winter (10 °C) |
|---|---|---|
| Aerobic SRT (Days) | 8.34 | 10.9 |
| HRT (Hrs) | 13.3 | 13.3 |
| MLSS (mg/L) | 3,354 | 4,477 |
| MLVSS (mg/L) | 2,043 | 2,752 |

Simulation results indicated that the existing two train system has adequate reactor volume to maintain the necessary aerobic suspended biomass required to meet the design aerobic SRT requirements. Under the 2040 max month summer loading conditions, significant denitrification was also observed with effluent TN below 15 mg/L. However, 2040 max month winter loading conditions did not achieve sufficient denitrification with effluent TN values greater than 24 mg/L.

In addition, SPA was performed for the simulated loading conditions. SPA diagrams are included in **Figure 3.25** and **Figure 3.26** for 2040 max month loading conditions. Based on the SPA results, a third clarifier would be recommended for 2040 max month winter loading conditions and would become necessary for 2040 max day loading conditions. Based on these results, it is not recommended that further consideration of facility expansion utilizing IFAS be considered.

*Figure 3.25 – SPA Diagram, 2 Clarifiers (2040 Max Month Loading = 8.39 MGD, Summer)*

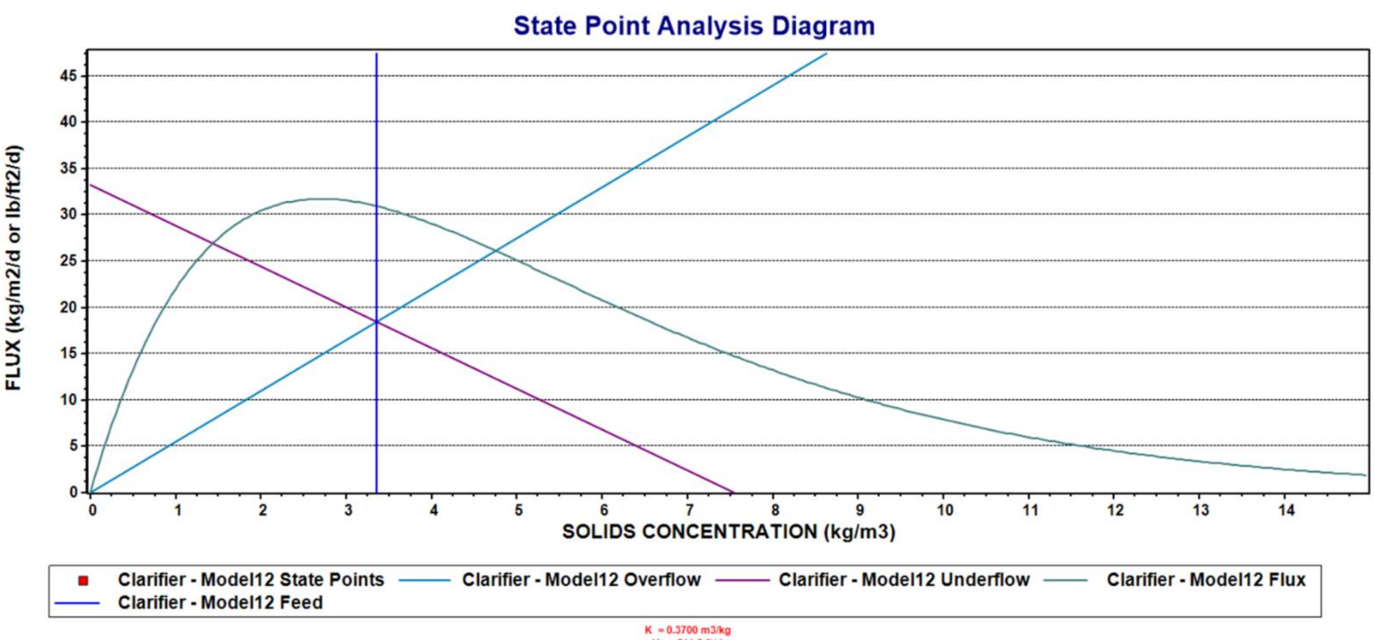
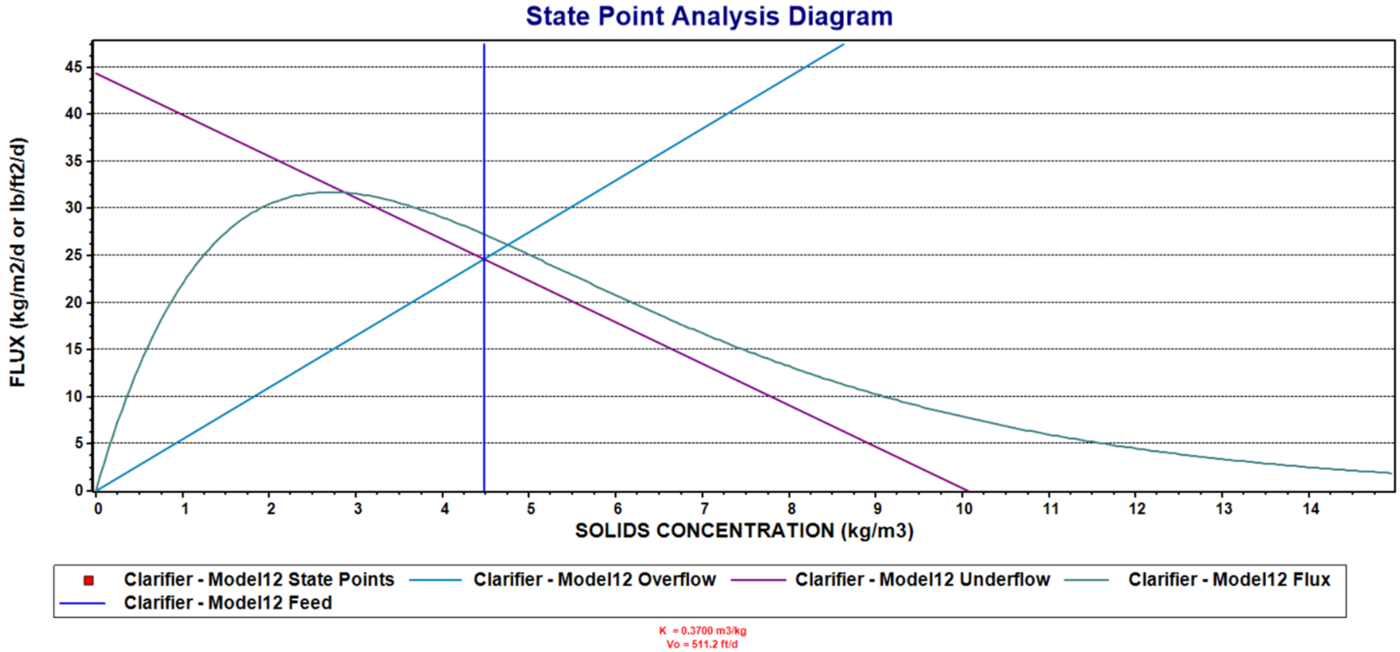


*Figure 3.26 – SPA Diagram, 2 Clarifiers (2040 Max Month Loading = 8.39 MGD, Winter)*




## 3.6 Recommendations

The above sections included evaluating the capacity of the existing secondary treatment process and expanding the existing process to meet projected 2040 loading conditions. The evaluation considered expanding the existing process utilizing the following technologies:

- Extended aeration process with secondary clarification
- Modified Ludzack-Ettinger process with secondary clarification
- Extended aeration process with secondary clarification with BioMag addition
- Modified Ludzack-Ettinger process with secondary clarification with BioMag addition
- IFAS process with secondary clarification

Based on the results outlined in the previous sections, it is anticipated that expansion of the existing secondary process utilizing the Modified Ludzack-Ettinger process with secondary clarification will be the most feasible and provide the best operational flexibility to include significant TN removal and alkalinity recovery. Further, this process will require significantly less aeration when compared to the extended aeration technology. Expansion would include modifying the existing extended aeration trains to include a dedicated anoxic zone with internal recycle to increase denitrification performance and the addition of a third train and third clarifier to meet 2040 loading conditions and provide sufficient system redundancy.

### 3.6.1 Modifications to Existing Aeration Basins

As noted above, it is recommended that the existing extended aeration process be converted to a Modified Ludzack-Ettinger process to continue to achieve nitrogen removal and alkalinity recovery at increased influent loading conditions expected in the future. The conversion to the MLE process will require several modifications to implement a dedicated anoxic zone in each of the two existing aeration basins. Physical separation of the anoxic and aerobic zones in each basin is not expected to be required, however, improvements will be required to provide adequate mixing and nitrified mixed liquor internal recycle flow.

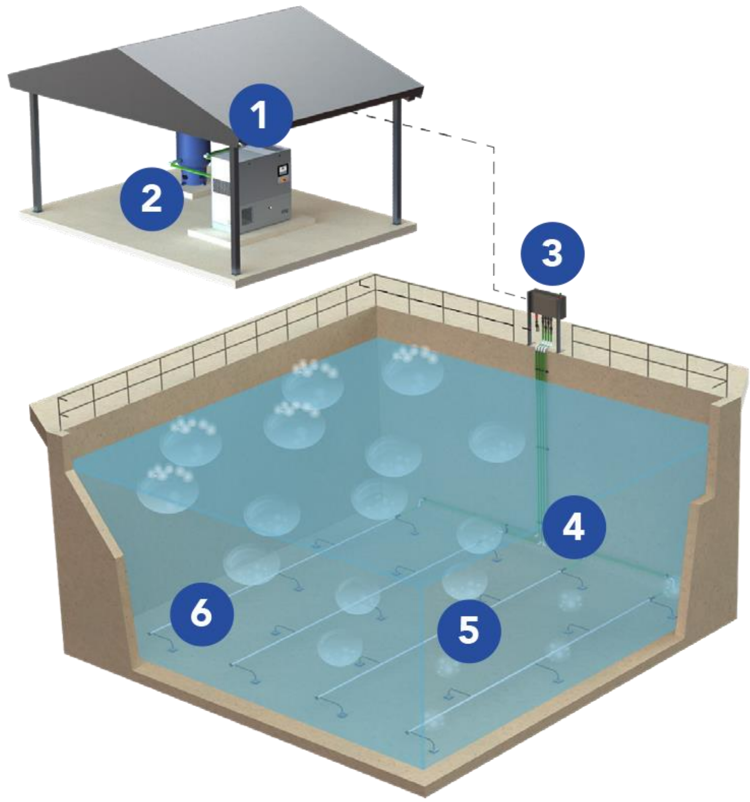
#### 3.6.1.1 *Mixing*

Air flow to diffuser grid 5 in each basin must be shut off to provide anoxic conditions at the head of each existing basin. With aeration in diffuser grid 5 shut off, a separate form of mixing must be provided to prevent deposition of suspended solids and ensure influent loading has sufficient contact with the mixed liquor. There are many different styles of mixing available to accomplish this task. The most common styles include submersible mechanical mixers, floating mechanical mixers, fixed mechanical mixers, jet mixing (without aeration), and compressed gas mixing. It is recommended that the City implement compressed gas mixing in the proposed anoxic zones of the existing basins. Compressed gas mixing is recommended for use in the proposed anoxic zones for the following reasons:

- It is easily implemented within the existing basins without removing the existing diffuser grids
- It has one of the lowest horsepower requirements per cubic foot of basin volume, at approximately 0.13 hp/1000 $ft^3$
- It has no moving parts within the basin and the submerged components are virtually maintenance free
- The large bubble size used results in effectively no oxygen transfer, making it ideal for anoxic zones
- It provides complete mixing with limited to no dead zones, which maximizes the basin volume available for treatment

A typical diagram of a compressed gas mixing system is shown in **Figure 3.27** below. Compressed air for mixing is provided by a compressor (1) and receiver tank (2) located outside of the basin. A valve module (3) located outside of the basin controls the pressure, frequency, duration, and sequence of nozzle (6) firing to ensure complete mixing throughout the basin. The compressor and receiver tank are recommended to be installed within a new compressor building to protect them from the elements. The new compressor building may be constructed adjacent to the existing basins.

*Figure 3.27 – Compressed Gas Mixing System Example Diagram*




*Source: EnviroMix, Inc. (https://enviro-mix.com/technology/)*

The conceptual cost opinion for the recommended mixing improvements is summarized in **Table 3.21** below.

*Table 3.21 – Anoxic Zone Compressed Gas Mixing Cost Opinion*

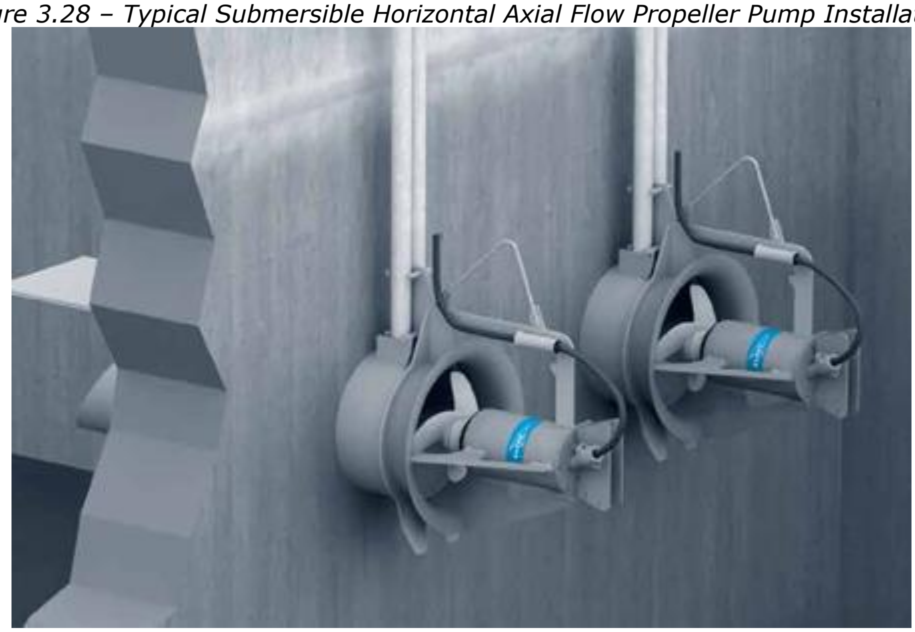
| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $215,000 |
| 2 | Mechanical | $43,000 |
| 3 | Electrical | $43,000 |
| 4 | Instrumentation | $22,000 |
| 5 | Structural | $21,000 |
| 6 | Civil | $0 |
| 7 | Mobilization & Demobilization | $14,000 |
| 8 | Indirect Costs | $18,000 |
| 9 | General Conditions & Contractor Markup | $80,000 |
| 10 | 30% Contingency | $137,000 |
| 11 | Engineering, Legal, & Administration | $110,000 |
| | **Total Cost Opinion** | **$703,000** |

### 3.6.1.2 *Nitrified Internal Recycle*

In the Modified Ludzack-Ettinger process, mixed liquor that is high in nitrate is recycled from the end of the aerobic zone of the basin to the head of the anoxic zone where influent wastewater is introduced to the basin. This nitrified internal recycle (NRCY) flow is shown in **Figure 3.13**Figure  which represents the BioWin model of this process. The nitrate recycled to the head of the anoxic zone is used by facultative heterotrophic microorganisms as the final electron acceptor in lieu of oxygen for the oxidation of soluble organic matter in the wastewater. The NRCY flowrate used in the MLE process typically ranges from 200% to 400% of the influent flow rate. A design NRCY flowrate of 300% of the design maximum month influent flowrate is recommended for the implementation of the MLE process at the City of Hendersonville's WWTF. NRCY flowrates greater than 300% of the influent flowrate typically have diminishing returns on nitrogen removal because it recycles excess oxygen from the aerobic zone and dilutes the influent wastewater, resulting in less efficient denitrification.

A submersible horizontal axial flow propeller pump and recycle pipeline is recommended to be installed in each of the existing aeration basins to provide the NRCY flowrate required. It is recommended that the NRCY pumps and recycle pipeline for the existing aeration basins be sized based on the 2025 design permitted capacity of 6.0 MGD at maximum month conditions. Based on this, a NRCY capacity of 9.0 MGD is required for each existing aeration basin, assuming 3.0 MGD of the influent flowrate is delivered to each basin. These pumps must be driven by VFDs to provide operational flexibility and flow-paced control. An example of a typical submersible horizontal axial flow propeller pump installation is shown in **Figure 3.28** below. The example shown below varies slightly from what is recommended herein, in that the mating flange of the pump would be mounted directly on the discharge pipeline rather than on a wall sleeve as shown.

*Figure 3.28 – Typical Submersible Horizontal Axial Flow Propeller Pump Installation*



*Source: Xylem, Inc. (https://www.xylem.com/siteassets/brand/flygt/flygt-resources/flygt-resources/fb155-431_design_rec_ultra_high_pumps.pdf)*

NRCY pumps similar to Flygt model P 4650 are recommended for this application to provide a maximum flow rate of 9.0 MGD and a minimum flow rate of 3.0 MGD per pump. One pump is recommended per basin, and a spare pump is recommended to be stocked on-site at the WWTF, for a total of three pumps for the existing basins. Each pump is recommended to discharge into a 20-inch diameter internal recycle pipeline for this application to provide a velocity of at least 2.0 fps at a minimum pumping rate of 3.0 MGD, and less than 6.5 fps at the maximum pumping rate. A 20-inch electromagnetic flow meter is recommended to be provided on each 20-inch diameter internal recycle pipeline to provide direct measurement of the internal recycle flowrate. A new electrical building is recommended to constructed adjacent to the existing aeration basins to house the VFDs and control equipment for the NRCY pumps. The conceptual cost opinion for the internal recycle improvements is summarized below in **Table 3.22**.

*Table 3.22 – Internal Recycle Improvement Cost Opinion*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $149,000 |
| 2 | Mechanical | $30,000 |
| 3 | Electrical | $30,000 |
| 4 | Instrumentation | $15,000 |
| 5 | Structural | $49,000 |
| 6 | Civil | $209,000 |
| 7 | Mobilization & Demobilization | $20,000 |
| 8 | Indirect Costs | $25,000 |
| 9 | General Conditions & Contractor Markup | $112,000 |
| 10 | 30% Contingency | $192,000 |
| 11 | Engineering, Legal, & Administration | $154,000 |
| | **Total Cost Opinion** | **$985,000** |

### 3.6.2 Modifications to Existing Blower Building

As noted above in **Section 3.3.2**, the existing Hoffman multistage centrifugal blowers are recommended to be replaced by new turbo blowers to provide variable speed control and improved energy efficiency. Three new turbo blowers rated for a maximum capacity of 5,000 SCFM each are recommended to be installed based on the preliminary estimates of aeration demands at the 2025 design conditions as summarized previously in **Table 3.4**. A more detailed analysis of expected aeration demands is recommended to be completed during detailed design of the blower replacement to verify blower capacity selection. It may be desirable to replace the three existing blowers with a combination of differently sized turbo blowers to ensure they operate near their best efficiency point under normal operation conditions. The blower sizing referenced above is assumed to be conservative for master planning and cost estimating purposes.

It is recommended that the existing blower building be modified to enclose the existing open canopy. Turbo blowers are recommended to be installed indoors to protect the air intakes from airborne dust and other foreign materials. Special considerations for filtered air intakes is recommended during detailed design for the blower building enclosure. The conceptual cost opinion for the blower building improvements is summarized in **Table 3.23** below.

*Table 3.23 – Blower Building Improvements Cost Opinion*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $650,000 |
| 2 | Mechanical | $130,000 |
| 3 | Electrical | $130,000 |
| 4 | Instrumentation | $65,000 |
| 5 | Structural | $155,000 |
| 6 | Civil | $0 |
| 7 | Mobilization & Demobilization | $46,000 |
| 8 | Indirect Costs | $54,000 |
| 9 | General Conditions & Contractor Markup | $259,000 |
| 10 | 30% Contingency | $447,000 |
| 11 | Engineering, Legal, & Administration | $359,000 |
| | **Total Cost Opinion** | **$2,295,000** |

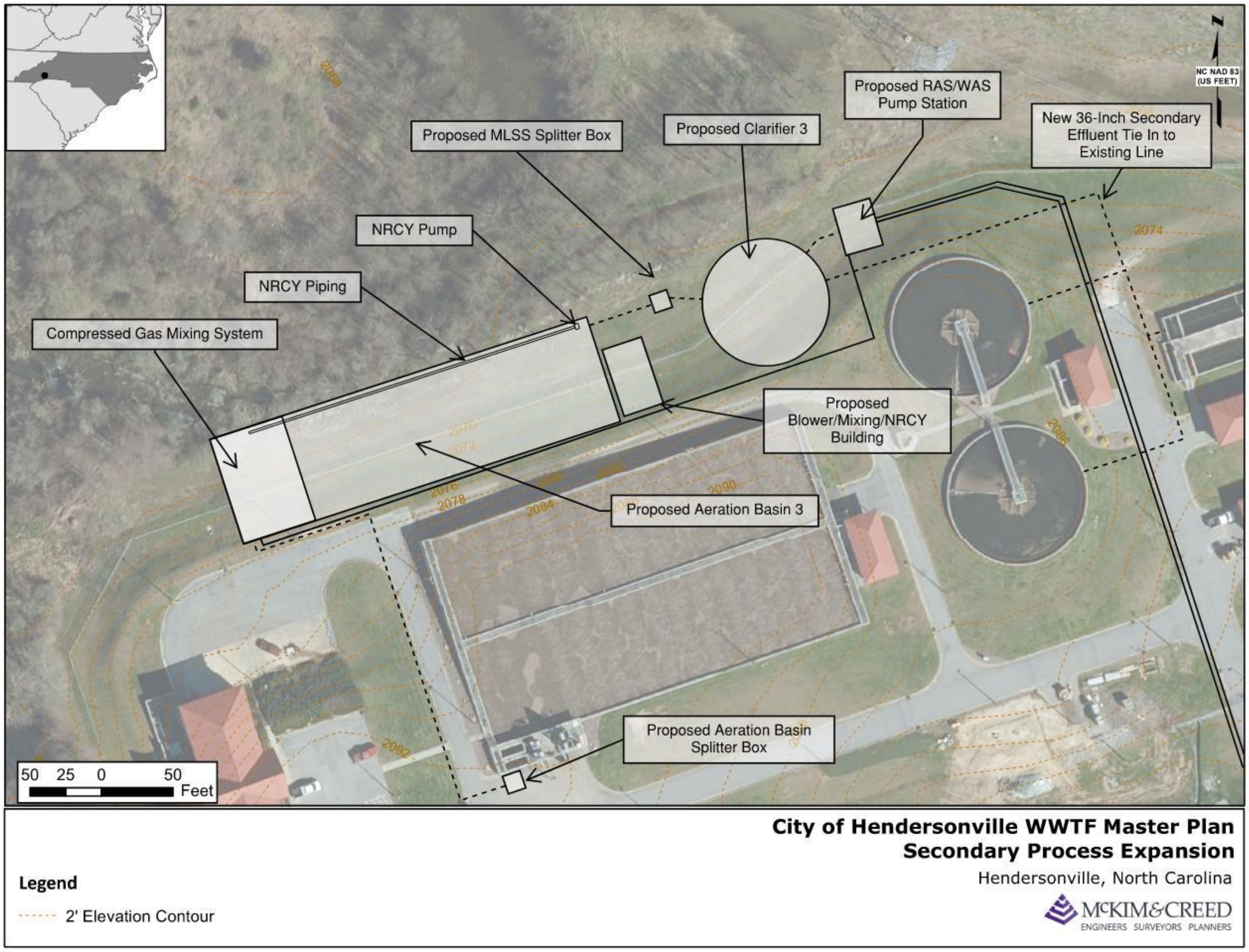
### 3.6.3 Expansion of Third Train

As noted above, the existing WWTF must be expanded to add a third treatment train in order to provide adequate treatment capacity for the anticipated 2040 loading conditions. Expansion of the existing WWTF to add a third treatment train is recommended to include the following:

- One primary effluent splitter box to split flow to the two existing and one new aeration basin
- A new 2.4 MG aeration basin No. 3, to match existing aeration basins No. 1 and No. 2, including a dedicated anoxic zone with compressed gas mixing and a NRCY pump and pipeline
- A new blower building No. 2 to house new turbo blowers, a compressed gas mixing system for the anoxic zone, NRCY pump VFDs, and all associated electrical and control equipment
- A new MLSS splitter box to direct aeration basin No. 3 effluent to a new secondary clarifier No. 3, and provide long-term future expansion capability to include a fourth aeration basin and secondary clarifier
- A new 90-ft diameter secondary clarifier No. 3, to match existing secondary clarifiers No. 1 and No. 2
- A new recycle pumping station No. 2 to include RAS and WAS pumping serving aeration basin No. 3 and secondary clarifier No. 3
- Piping tie-ins to the existing 36” secondary effluent pipeline to tertiary filtration (alternatives for expansion of tertiary filtration are discussed in **Section 4.2**)

The recommended layout of the third treatment train and its incorporation into the existing WWTF is shown in **Figure 3.29** below.

*Figure 3.29 - Recommended Layout for 2040 Secondary Treatment Process Expansion*



The conceptual cost opinion for the expansion of a third secondary treatment process train is summarized in **Table 3.24** below.

*Table 3.24 – 2040 Secondary Treatment Process Expansions Cost Opinion*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $1,557,000 |
| 2 | Mechanical | $312,000 |
| 3 | Electrical | $312,000 |
| 4 | Instrumentation | $156,000 |
| 5 | Structural | $6,938,000 |
| 6 | Civil | $3,374,000 |
| 7 | Mobilization & Demobilization | $506,000 |
| 8 | Indirect Costs | $594,000 |
| 9 | General Conditions & Contractor Markup | $2,888,000 |
| 10 | 30% Contingency | $4,992,000 |
| 11 | Engineering, Legal, & Administration | $4,011,000 |
| | **Total Cost Opinion** | **$25,640,000** |

## 3.7 References


- McKim & Creed, Inc. (2021). (rep.). *City of Hendersonville Solids Management Plan Evaluation*. Charlotte, NC.
- Daigger, G. T. (2010). A Practitioner's Perspective on the Uses and Future Developments for Wastewater Treatment Modelling. *Proceedings of the Water Environment Federation*, *2010*(16), 1026–1026. https://doi.org/10.2175/193864710798158508
- Tchobanoglous, G., Stensel, H. D., Tsuchihashi, R., & Burton, F. L. (2014). *Wastewater Engineering: Treatment and Resource Recovery* (5th ed.). New York, NY: McGraw-Hill Education.
- Evoqua Water Technologies. (n.d.). *The Biomag System*. Waukesha, WI; Evoqua Water Technologies.
- Evoqua Water Technologies. (2017). *The Biomag System for Enhanced Secondary Treatment*. Waukesha, WI.
- Evoqua Water Technologies. (2017). *BIOMAG/COMAG – FREQUENTLY ASKED QUESTIONS*. Waukesha, WI.
- Grady, C. P., Daigger, G. T., Love, N. G., & Filipe, C. D. M. (2011). *Biological wastewater treatment* (3rd ed.). IWA Publ.
- Mandli, C. (2017, September 15). Media - Kruger USA AnoxKaldnes K5 Media. https://www.tpomag.com/g/product-focus/2017/09/media_kruger_usa_anoxkaldnes_k5_media.

- EnviroMix, Inc. (2020). *BioMix Compressed Gas Mixing* [Image]. https://enviro-mix.com/technology/.
- Xylem, Inc. (2015). *Design Recommendations for Installation of Flygt Ultra-Low-Head, High-Flow Pumps* [Image]. https://www.xylem.com/siteassets/brand/flygt/flygt-resources/flygt-resources/fb155-431_design_rec_ultra_high_pumps.pdf.

# 4. TERTIARY FILTERS EVALUATION

## 4.1 Current Capacity Analysis

Clarified effluent is polished through tertiary filtration to remove additional suspend solids and turbidity prior to ultraviolet disinfection. The City currently employs two tertiary filter units, one Aqua-Aerobic Systems Inc. AquaDiamond cloth media traveling bridge filter (tertiary filter No. 1), and one EIMCO traveling hood sand filter (tertiary filter No. 2). Tertiary filter No. 1 used to be an EIMCO traveling hood sand filter matching tertiary filter No. 2, which were installed during the original construction of the current WWTF, approximately 20 years ago. Construction of the AquaDiamond cloth media filter in tertiary filter No. 1 was completed in 2020, and the rated capacity of the AquaDiamond tertiary filter No. 1 is 6.0 MGD at average daily flow, and 15.0 MGD at peak hourly flows. Additional capacity information for the AquaDiamond tertiary filter No. 1 is summarized in **Table 4.1** below.

*Table 4.1 – AquaDiamond Tertiary Filter No. 1 Current Capacity Summary*

| Parameter | Units | Value |
|---|---|---|
| Average Daily Design Flow | MGD | 6.0 |
| Average Hydraulic Loading | $gpm/ft^2$ | 2.60 |
| Peak Hour Design Flow | MGD | 15.0 |
| Peak Hydraulic Loading | $gpm/ft^2$ | 6.51 |
| Average Design Suspended Solids | mg/L | 5.0 |
| Peak Design Suspended Solids | mg/L | 15.0 |
| Number of Diamond Laterals per Unit | - | 8 |
| Length per Diamond Lateral | ft | 50 |
| Total Filter Area provided | $ft^2$ | 1,600 |
| Filter Media Cloth Type | - | OptiFiber PA2-13 |
| Filter Media Cloth Nominal Pore Size | µm (micron) | 10 |

The current tertiary filter No. 2 is an EIMCO traveling hood sand filter with a total filter media surface area of approximately 832 $ft^2$ (16-ft wide by 52-ft long). The original basis of design hydraulic loading rates for the EIMCO traveling hood sand filter were 2.0 $gpm/ft^2$ at average design conditions, and 5.0 $gpm/ft^2$ at peak hourly flow. The original basis of design hydraulic loading rates for filter No. 2 equate to an average design capacity of 2.4 MGD and a PHF capacity of 6.0 MGD. However, following the rise in popularity of this style of traveling hood sand filter and other similar automatic backwash granular media filters, operational experience at many WWTF's have shown that the peak hydraulic capacity of this style of filter degrades over time. Causes of reduced hydraulic capacity in traveling hood sand filters and other similar automatic backwash granular media filters includes solids capture within the filter media, fouling of the filter media, fouling of the filter underdrain system, and clogging of the filter underdrain system.

Actual documented peak hydraulic loading rates collected by the Water Environment Federation for traveling hood sand filters and other similar automatic backwash granular media filters has ranged from 2.0 gpm/ft$^2$ to 4 gpm/ft$^2$. In McKim & Creed's experience, actual peak hydraulic loading rates of 2.0 gpm/ft$^2$ have been most common. Based on this, it is recommended that the peak hydraulic capacity of filter No. 2 be evaluated based on a peak hydraulic loading rate of 2.0 gpm/ft$^2$, which equates to a peak hydraulic capacity of 2.4 MGD.

Past observations of tertiary filter No. 2's performance support expectations of a maximum hydraulic loading rate of approximately 2.0 gpm/ft$^2$. One example of observed operation under increased hydraulic loading occurred during installation of the AquaDiamond filter in tertiary filter No. 1 in the fall and winter of 2019. During this time, tertiary filter No. 2 was the only filter in operation. Following wet weather events when influent flows to the WWTF exceeded average daily conditions of 3.0 MGD, head loss through tertiary filter No. 2 would occasionally back up into the filter's common influent channel and bypass the filters via the overflow weir. It is also noted that the underdrain system in tertiary filter No. 2 has failed in various areas of multiple filter cells with underdrain failure getting progressively worse over time. Failure of the filter underdrain system has likely allowed filter No. 2 to pass higher flows than 2.4 MGD, however, this flow bypasses the filter media and is unfiltered prior to UV disinfection.

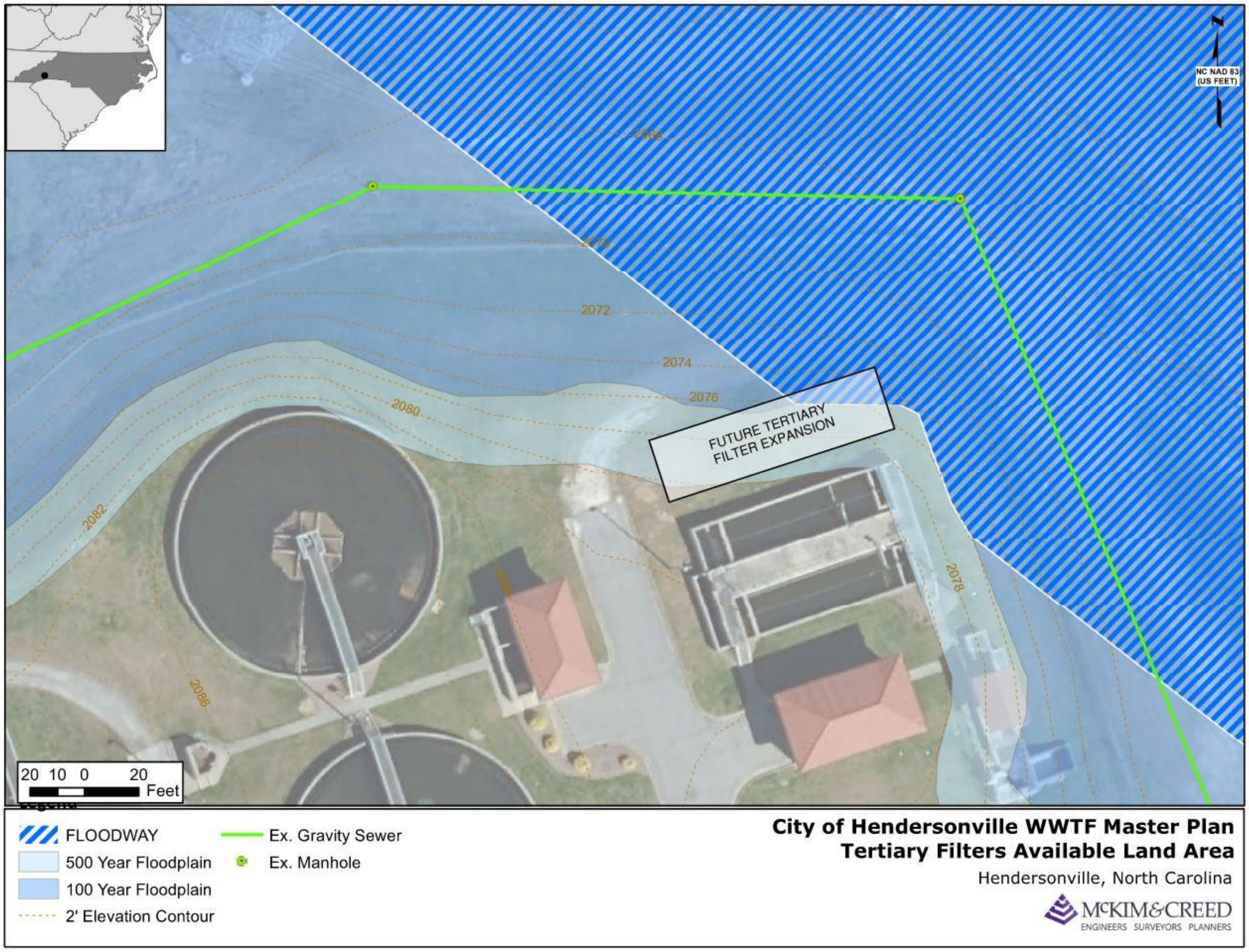
Based on the observed capacity of tertiary filter No. 2 and an assumed peak hydraulic loading rate of 2.0 gpm/ft$^2$, the WWTF's tertiary filtration process has a maximum firm capacity of approximately 2.4 MGD with the largest unit out of service. As a result, the existing tertiary filters do not meet NCDEQ Minimum Design Criteria for NPDES Wastewater Treatment Facilities since tertiary filter No. 2 cannot pass the peak hourly flow to the facility with tertiary filter No. 1 out of service. The existing tertiary filtration process must be expanded to provide adequate firm capacity at current and future peak hourly flows.

The design and construction of the AquaDiamond cloth media filter replacement of tertiary filter No. 1 included provisions to support the future replacement of tertiary filter No. 2. If implemented, the replacement of tertiary filter No. 2 with an equally sized AquaDiamond cloth media filter will increase the firm capacity of the tertiary filtration process to 15.0 MGD at peak hour conditions. It is recommended that the City plan for the near-term replacement of tertiary filter No. 2 with an AquaDiamond cloth media filter matching tertiary filter No. 1. This will maximize the capacity of the existing tertiary filter basins and ensure operator familiarity with the redundant tertiary filter. Future increases in the design peak hourly flow to the tertiary filters above 15.0 MGD will require the installation of additional tertiary filters. Alternatives for the expansion of the tertiary filters to handle future 2040 peak hourly flow conditions are described in more detail below.

## 4.2 Alternatives Evaluation

As indicated above, additional tertiary filtration capacity will be required to provide firm capacity at design peak hour flow conditions above 15.0 MGD. Per the flow projections presented in the SSAIA Master Plan Report, and **Table 2.1** earlier in this report, these conditions are expected to occur between 2025 and 2040, assuming flow equalization facilities are implemented to store wet weather peak flows above the allowable peak flow to the facility. The hydraulic capacity of the WWTF is projected to be 19.5 MGD in 2040, indicating the third tertiary filter should have a capacity of at least 4.5 MGD to handle flows in excess of the firm capacity (15 MGD) of filters No. 1 and 2 by 2040. Tertiary filter No. 3 would be installed to the north of the existing filters as shown in **Figure 4.1** below and would require modifications to the existing 36-inch diameter secondary effluent piping to tie-in the third filter. Tertiary filter No. 3's influent weir should be installed at the same elevation as tertiary filters No. 1 and No. 2 and should be sized to provide flow splitting proportional to the capacity of each filter. Tertiary filter No. 3 should also be provided with isolation plug valves or gates to take it offline when not needed.

*Figure 4.1 – Available Land Area for Tertiary Filtration Expansion*




There is a wide variety of available technologies for tertiary filtration, but they may be summarized in two main categories: granular media filtration and cloth media filtration. Cloth media filtration has gained significant popularity over granular media filtration in the last 20 years due to higher effluent quality, increased capacity in a smaller footprint, minimal head loss, lower backwash rates, and reduced maintenance requirements.

The primary objectives of the alternatives evaluated herein to provide additional filtration capacity are to limit capital and operating costs, limit footprint requirements, limit process head loss, limit maintenance requirements, and maximize operator familiarity. Granular media filtration technologies are limited in their ability to meet these objectives, therefore only cloth media filtration technologies are evaluated below.

The alternatives evaluated below each assume that tertiary filter No. 2 will be converted to an AquaDiamond cloth media filter with equal capacity to tertiary filter No. 1. The opinion of probable project cost for installation of AquaDiamond filter No. 2 is summarized below in **Table 4.2**. The cost opinion detailed below is based on the actual costs incurred during the installation of AquaDiamond filter No. 1, with deductions to account for work that was completed during the installation of AquaDiamond filter No. 1 to prepare for an eventual AquaDiamond filter No.2. The contingency for this project was reduced from the typical 30% for conceptual cost estimates to 15% due to the higher degree of cost certainty based on the previous filter installation. In addition, engineering, legal, and administration costs were reduced for this project due to the reduced scope of work required since electrical, structural, and mechanical designs completed for the tertiary filter No. 1 replacement included most of the design requirements for the tertiary future filter No. 2 replacement.

*Table 4.2 – Estimated AquaDiamond Cloth Media Filter Cost Opinion*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $884,000 |
| 2 | Mechanical | $117,000 |
| 3 | Electrical | $100,000 |
| 4 | Instrumentation | $57,000 |
| 5 | Structural | $30,000 |
| 6 | Demo | $104,000 |
| 7 | Mobilization & Demobilization | $52,000 |
| 8 | Indirect Costs | $62,000 |
| 9 | General Conditions & Contractor Markup | $296,000 |
| 10 | 15% Contingency | $256,000 |
| 11 | Engineering, Legal, & Administration | $246,000 |
| | **Total Cost Opinion** | **$2,204,000** |

**Table 4.3** and **Table 4.4** show the annual O&M costs for current and future operation of AquaDiamond filter No. 2. Operating cost estimates for electricity usage assumed total backwashing durations for 2021 and 2040 would be approximately 4 hours per day and 8 hours per day, respectively.

*Table 4.3 – AquaDiamond Cloth Media Filter - Annual O&M Costs - 2021*

| Item | Annual Cost |
|---|---|
| Maintenance | $18,000 |
| Electricity | $2,000 |
| **TOTAL** | **$20,000** |

*Table 4.4 – AquaDiamond Cloth Media Filter - Annual O&M Costs - 2040*

| Item | Annual Cost |
|---|---|
| Maintenance | $18,000 |
| Electricity | $4,000 |
| **TOTAL** | **$22,000** |

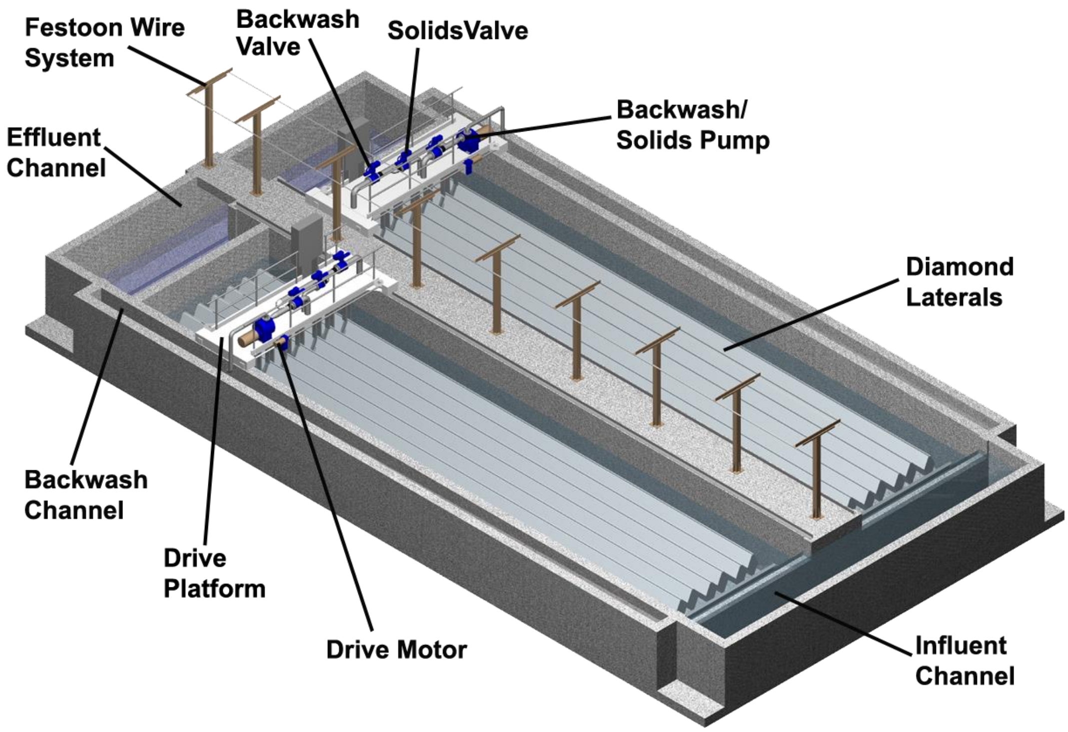
### 4.2.1 Alternative 1 – Additional AquaDiamond Train

This alternative consists of the construction of an additional AquaDiamond train to operate in parallel with AquaDiamond filters No. 1 and No. 2 (once installed). This baseline alternative is based on standardizing on the AquaDiamond cloth media technology to maximize operator familiarity, ensure consistent maintenance procedures among all tertiary filters, and reduce the number of spare parts required to be stored on site.

The AquaDiamond filter, as shown in **Figure 4.2** below, operates by effluent from the secondary clarifiers entering the filter basin over the influent weir and passing through the cloth media from the outside of the diamond, in. The filtrate flows lengthwise inside the diamond laterals (which penetrate the filter basin end wall) and enters the effluent channel where it falls over the effluent weir. As the filters become covered in solids, head loss increases and the water level inside the basin rises until it reaches a set point at which the control system initiates a backwash cycle. The backwash drive platform travels along the length of the filter basin until the entire length of the diamond laterals has been cleaned.

Based on the firm capacity requirements at the 2040 conditions, AquaDiamond filter No. 3 may be sized based on a peak hourly flow rate of 4.5 MGD. This will provide a firm hydraulic capacity of 19.5 MGD at 2040 design conditions when operated in parallel with either AquaDiamond filter No. 1 or No. 2. This sizing allows AquaDiamond filter No. 3 to be smaller than filters No. 1 and No. 2, which may be accomplished using a fewer, shorter AquaDiamond laterals than the other filters. AquaDiamond filter No. 1 was installed with eight 50-foot long AquaDiamond laterals with a filter media surface area of 1,600 $ft^2$, sized based on a peak hydraulic loading rate of 6.51 $gpm/ft^2$. Extending this hydraulic design criteria to tertiary filter No. 3, a filter media surface area of 480 $ft^2$ would be required. The AquaDiamond laterals installed in filter No. 1 have 4 $ft^2$ of filter media surface area per foot of lateral length. It was assumed that AquaDiamond filter No. 3 would use the same type of AquaDiamond lateral, which would require 120 linear feet of AquaDiamond laterals. Therefore, AquaDiamond filter No. 3 was assumed to consist of four 30-foot long AquaDiamond laterals to provide a maximum design flow of 4.5 MGD.

*Figure 4.2 – AquaDiamond Filter Train*




Advantages of this alternative include:

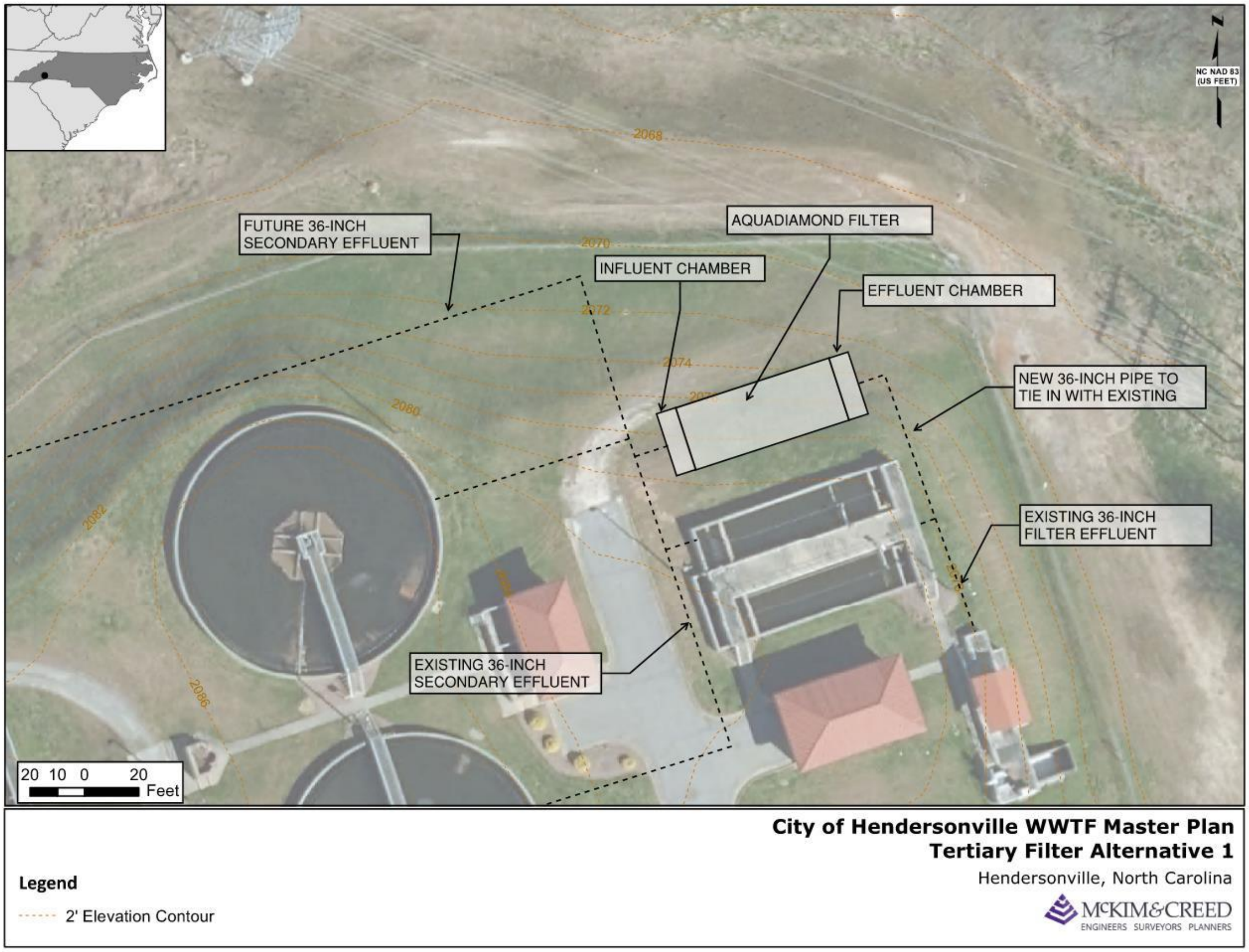
- Operator familiarity
- Standardization of a single filter type for the WWTF, which has multiple benefits including common spare parts, minimal operator training, and identical operation for all filters

Disadvantages of this alternative include:

- Largest footprint of all three alternatives
- Highest Cost
- Solids will deposit in the filter basin, so a solids-capture system is required
- A cover system is not provided by the manufacturer; will require a shelter structure to protect the equipment from weather and prevent algae growth
- Originally intended as a retrofit option for existing traveling bridge sand filters, requires construction of cast-in-place concrete basin

A preliminary layout map for this alternative is shown in **Figure 4.3** below.

*Figure 4.3 - Tertiary Filter Alternative 1 Proposed Layout*



The conceptual cost opinion for Alternative 1 is included below in **Table 4.5.**

*Table 4.5 – Estimated Tertiary Filters Alternative 1 Cost Opinion*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $400,000 |
| 2 | Mechanical | $127,000 |
| 3 | Electrical | $125,000 |
| 4 | Instrumentation | $59,000 |
| 5 | Structural | $192,000 |
| 6 | Civil | $495,000 |
| 7 | Mobilization & Demobilization | $56,000 |
| 8 | Indirect Costs | $67,000 |
| 9 | General Conditions & Contractor Markup | $321,000 |
| 10 | 30% Contingency | $553,000 |
| 11 | Engineering, Legal, & Administration | $444,000 |
| | **Total Cost Opinion** | **$2,839,000** |

**Table 4.6** below shows the annual O&M costs for AquaDiamond filter No. 3 at the 2040 design conditions.

*Table 4.6 – Tertiary Filters Alternative 1 - Annual O&M Costs - 2040*

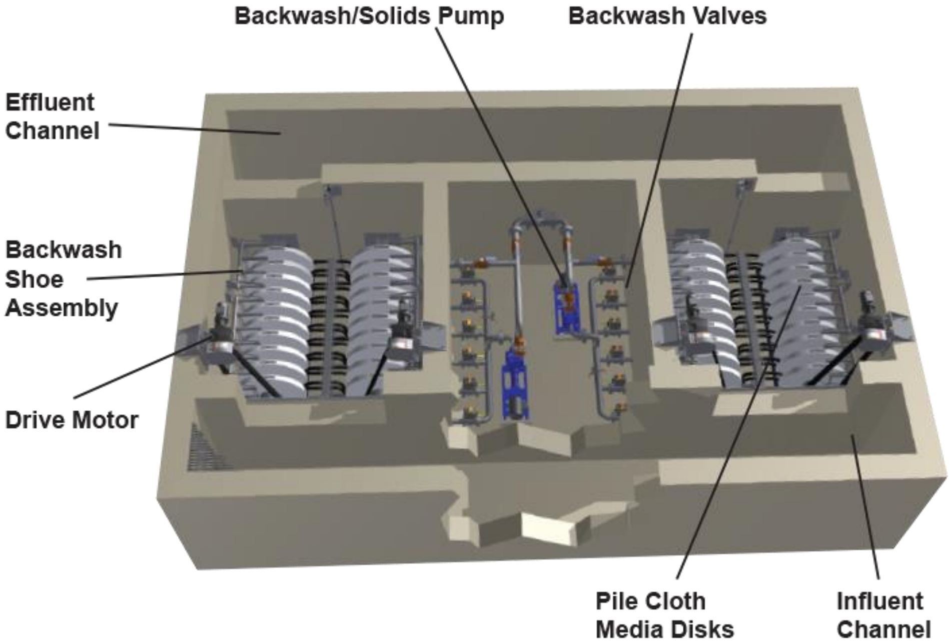
| Item | Annual Cost |
|---|---|
| Maintenance | $8,000 |
| Electricity | $2,000 |
| **TOTAL** | **$10,000** |

### 4.2.2 Alternative 2 – New Cloth Media Depth Filtration Disc Filter Train

This alternative consists of the construction of a new cloth media depth filtration disc filter train to operate in parallel with tertiary filters No. 1 and No. 2. One example of this filtration technology is the Aqua-Aerobic Systems, Inc. AquaDisk filter. The AquaDisk filter utilizes the same pile cloth media as the existing AquaDiamond filter, but it is arranged on a series of rotating discs inside of a steel or concrete containment vessel.

The AquaDisk, shown in **Figure 4.4** below, operates by having effluent from the secondary clarifiers enter the filter basin over an influent weir and pass through the fully submerged, normally stationary discs from the outside-in. Solids are captured on the cloth media on the exterior of the disc. A backwash shoe assembly is mounted in a stationary position on each disc which engages with the cloth media on both sides of each disc when a backwash sequence is initiated. The AquaDisk drive motor rotates the discs at a slow rate during backwash. The cloth media on each disc is backwashed as it rotates past the backwash shoe assembly.

*Figure 4.4 – AquaDisk Filter*




Advantages of this alternative include:

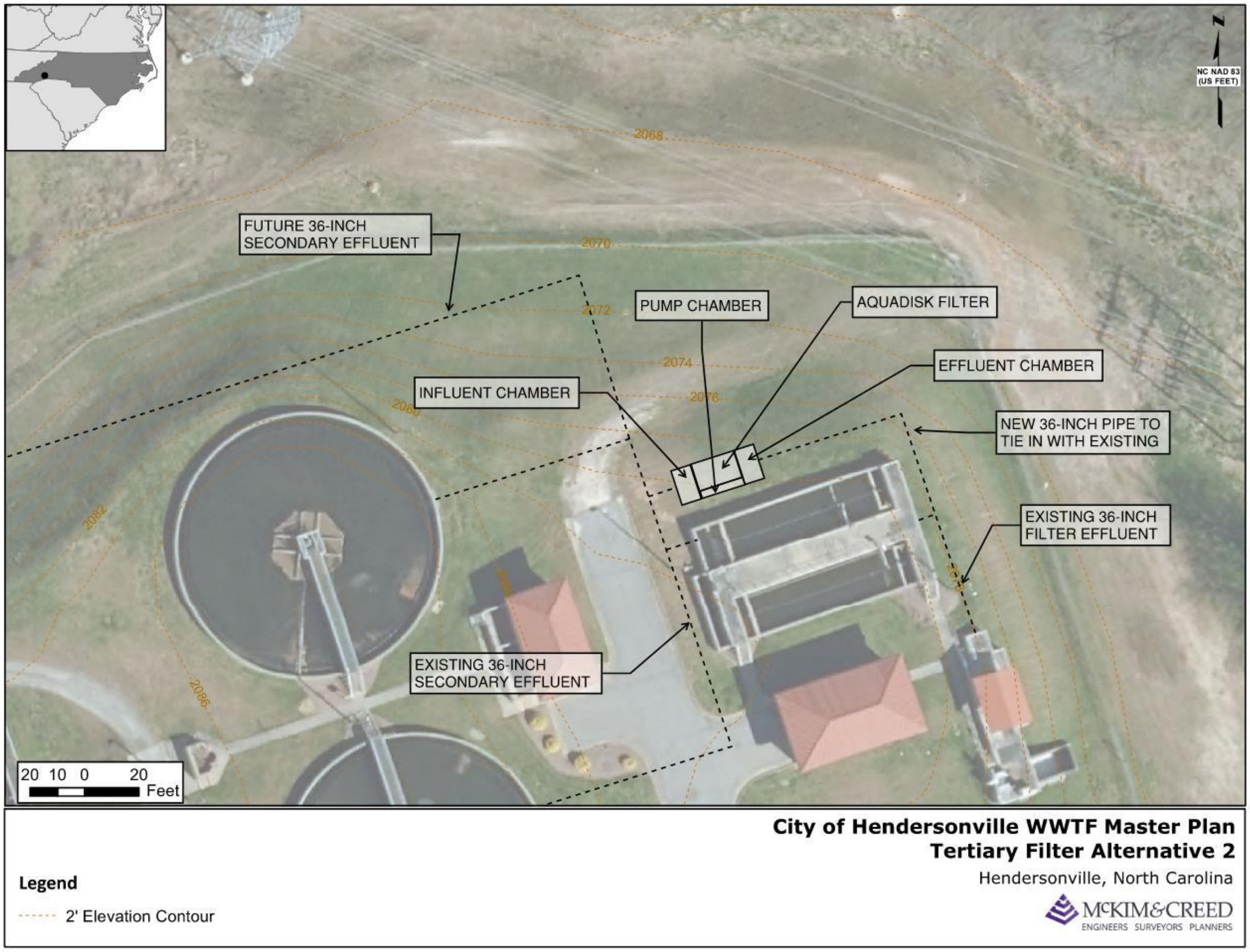
- Smaller footprint than AquaDiamond
- Less expensive than AquaDiamond
- Same manufacturer as AquaDiamond filters, good operator familiarity with cloth media maintenance requirements
- Equipment can be provided for installation in concrete basins, or can be provided in a "plug and play" steel containment unit

Disadvantages of this alternative include:

- Larger footprint than Hydrotech Discfilter
- More expensive than Hydrotech Discfilter
- Solids will deposit in the filter basin, so a solids-capture system is required
- A cover system is not provided by the manufacturer; will require a shelter structure to protect the equipment from weather and prevent algae growth
- Different filter types between new filter train and AquaDiamond filters No. 1 and No. 2

A preliminary layout map for this alternative is shown in **Figure 4.5** below.

*Figure 4.5 - Tertiary Filter Alternative 2 Proposed Layout*




Estimated conceptual cost opinion for Alternative 2 is included below in **Table 4.7.** This cost opinion assumes the AquaDisk filter would be installed in a cast-in-place concrete basin constructed on-site.

*Table 4.7 – Estimated Tertiary Filters Alternative 2 Cost Opinion*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $337,000 |
| 2 | Mechanical | $68,000 |
| 3 | Electrical | $68,000 |
| 4 | Instrumentation | $34,000 |
| 5 | Structural | $84,000 |
| 6 | Civil | $501,000 |
| 7 | Mobilization & Demobilization | $44,000 |
| 8 | Indirect Costs | $53,000 |
| 9 | General Conditions & Contractor Markup | $251,000 |
| 10 | 30% Contingency | $432,000 |
| 11 | Engineering, Legal, & Administration | $347,000 |
| | **Total Cost Opinion** | **$2,219,000** |

**Table 4.8** below shows the annual O&M costs for the AquaDisk filter at the 2040 design conditions.

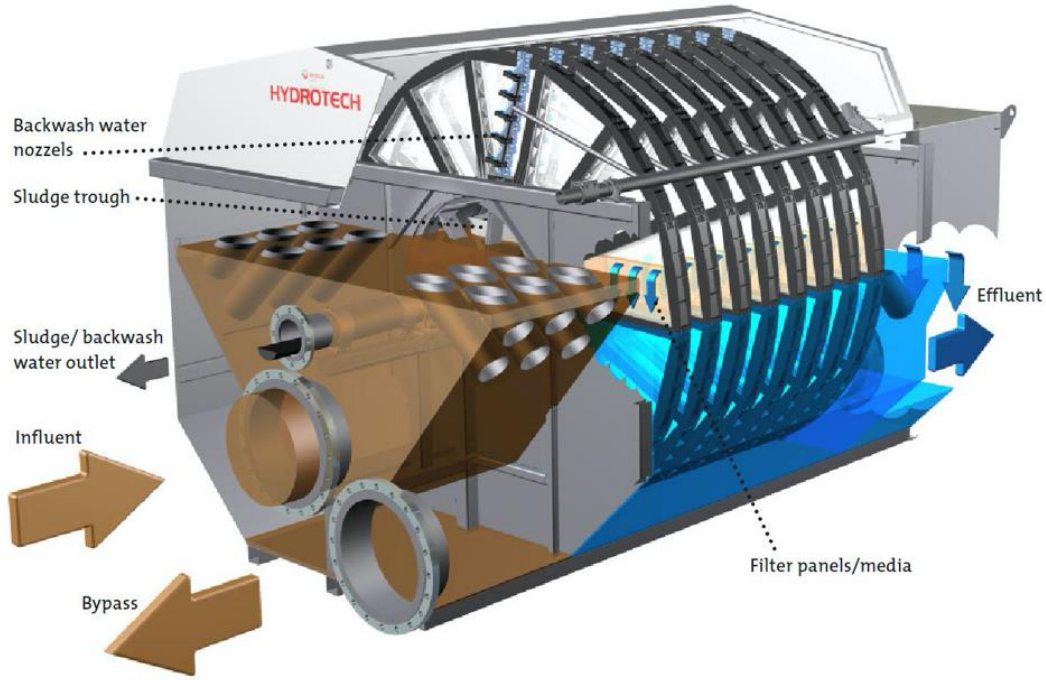
*Table 4.8 – Tertiary Filters Alternative 2 - Annual O&M Costs - 2040*

| Item | Annual Cost |
|---|---|
| Maintenance | $7,000 |
| Electricity | $2,000 |
| **TOTAL** | **$9,000** |

### 4.2.3 Alternative 3 – New Cloth Surface Filtration Disc Filter Train

Alternative 3 is similar to alternative 2, however it utilizes cloth surface filtration disc filters instead of cloth depth filtration disc filters. Several manufacturers produce cloth surface filtration disc filters including but not limited to Veolia's Hydrotech Discfilter, Evoqua's Forty-X Disc Filter, and WesTech's SuperDisc. For simplicity, this alternative is based on Veolia's Hydrotech Discfilter, however final equipment selection should be evaluated further during detailed design. One additional manufacturer to consider for this alternative, if selected, is the Nova Quantum disc filter which utilizes a woven stainless steel mesh filter media instead of woven cloth media. The woven stainless steel mesh used by the Nova Quantum disc filter is more durable than woven cloth media and provides extended service life that may offset other costs when compared in a present worth analysis for the design life of the equipment.

*Figure 4.6 – Hydrotech Discfilter*




The primary differences between this technology and the AquaDisk filter are related to the flow direction and the cloth media. Under this alternative, influent from the clarifiers flows by gravity into a center drum where it then flows through each disc from the inside-out, as shown in **Figure 4.6** above. Since the solids are captured on the inside of each disc, very little to no solids deposit in the disc filter basin, so a solids capture system is not necessary. Backwashing is accomplished by pumping the filtered effluent through backwash spray nozzles on the outside of each disc. During backwash, the filter discs are rotated to allow the spray nozzles to backwash the previously submerged sections, while the clean portion of the disc is rotated into the waste stream. The backwash water and solids washed from the cloth media flows into a center sludge trough and then flows by gravity to the in-plant drain system. The cloth media used by these disc filters is a woven polyester cloth instead of the synthetic pile fabric cloth used by the AquaDisc filter. The woven cloth media has a pore size of 10 microns, which is equivalent to the pile fabric cloth, but it does not provide any depth filtration.

Advantages of Alternative 3 include:

- Disc filter includes equipment cover (regardless of concrete or steel basin) to protect filter media from weather and algae growth, eliminating the need for a shelter structure
- No solids accumulation outside of filter media
- Smallest footprint, allows for ease of future expansion

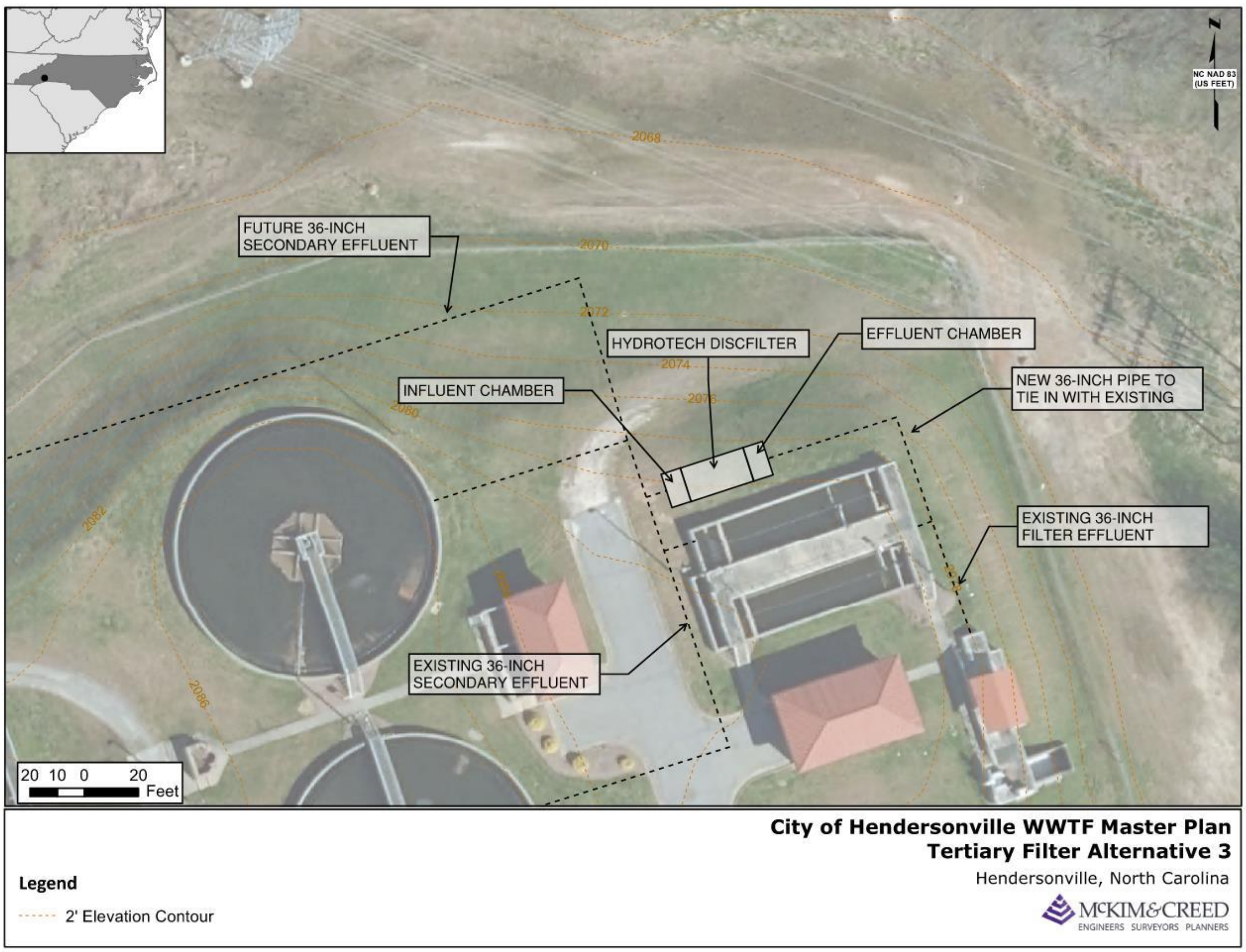
- Least expensive option
- Equipment can be provided for installation in concrete basins, or can be provided in a "plug and play" steel containment unit
- Optional mobile automated supplemental cleaning systems available for Hydrotech Discfilter to eliminate manual chemical cleaning

Disadvantages of Alternative 3 include:

- Different manufacturers between new filter train and AquaDiamond filters No. 1 and No. 2

A preliminary layout map for this alternative is shown in **Figure 4.7** below.

*Figure 4.7 - Tertiary Filter Alternative 3 Proposed Layout*



The estimated conceptual cost opinion for Alternative 3 is included below in **Table 4.9.** This cost opinion assumes the Hydrotech Discfilter would be installed in a cast-in-place concrete basin constructed on-site.

*Table 4.9 – Estimated Tertiary Filters Alternative 3 Cost Opinion*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $229,000 |
| 2 | Mechanical | $46,000 |
| 3 | Electrical | $46,000 |
| 4 | Instrumentation | $23,000 |
| 5 | Structural | $72,000 |
| 6 | Civil | $451,000 |
| 7 | Mobilization & Demobilization | $35,000 |
| 8 | Indirect Costs | $43,000 |
| 9 | General Conditions & Contractor Markup | $199,000 |
| 10 | 30% Contingency | $344,000 |
| 11 | Engineering, Legal, & Administration | $276,000 |
| | **Total Capital Cost** | **$1,764,000** |

**Table 4.10** below shows the annual O&M costs for the Hydrotech Discfilter at the 2040 design conditions.

*Table 4.10 Tertiary Filters Alternative 3 - Annual O&M Costs - 2040*

| Item | Annual Cost |
|---|---|
| Maintenance | $5,000 |
| Electricity | $1,000 |
| **TOTAL** | **$6,000** |

## 4.3 Recommendations

A summary of the characteristics of each alternative for tertiary filter No. 3 expansion is provided in **Table 4.11** below. As seen below, each of the three alternatives evaluated have very similar operational characteristics and all are expected to produce similar effluent quality. Based on this, equipment cost, total project cost, maintenance requirements, footprint requirements, and ease of future expansion are the most important aspects to consider when comparing these technologies.

*Table 4.11 – Comparison of Characteristics for Cloth Media Filters[1]*

| Parameter | Unit | AquaDiamond | AquaDisk | Discfilter |
|---|---|---|---|---|
| Typical hydraulic loading rate (HLR) | $gpm/ft^2$ | 2-5 | 2-5 | 2-5 |
| Peak HLR | $gpm/ft^2$ | 6.5 | 6.5 | 6.0 |
| Filter material | Type | Nylon and/or polyester | Nylon and/or polyester | Polyester or stainless steel |
| Nominal pore size of screen | microns (μm) | 5-10 | 5-10 | 10-40 |
| Direction of Flow | - | Out-in | Out-in | In-out |
| Submergence | % | 100 | 100 | 60-70 |
| Head Loss | ft | 0.2 – 1.0 | 0.2 – 1.0 | 0.25 – 1.0 |
| Backwash requirement | % of throughput | 2-5 | 2-5 | 2-4 |
| Containment Vessel Materials | - | Concrete Basin | Steel or Concrete | Steel or Concrete |
| Chemical Cleaning System | - | Manual | Manual | Manual or Automated |
| Footprint | - | Large | Small/Moderate | Small |

[1]*Source: Tchobanoglous, Stensel, Tsuchihashi, & Burton, 2014, p. 1178*

A summary of the capital costs and annual O&M costs for each alternative for the expansion of tertiary filter no. 3 is provided in **Table 4.12** below.

*Table 4.12 – Cost Comparison of Tertiary Filter No. 3 Alternatives*

| Alternative | Capital Costs | 2040 Annual O&M Cost |
|---|---|---|
| **Alternative 1: AquaDiamond** | $2,839,000 | $10,000 |
| **Alternative 2: AquaDisk** | $2,219,000 | $9,000 |
| **Alternative 3: Hydrotech Discfilter** | $1,764,000 | $6,000 |

Based on the information presented above, Alternative 3 based on the Hydrotech Discfilter is the recommended alternative for future expansion of tertiary filter No. 3 to meet the 2040 peak hydraulic capacity requirements of 19.5 MGD. Alternative 3 represents the lowest equipment and total project cost, has the smallest footprint of all alternatives evaluated, is available in a “plug and play” steel containment, is easily expandable with additional discs and additional filter units, does not require a separate shelter structure, and has options available for automated supplemental chemical cleaning. These benefits of Alternative 3 outweigh the disadvantage of having two separate filter equipment manufacturers on-site, since maintenance requirements of the third filter are minimal.

## 4.4 References


- Aqua-Aerobic Systems, Inc. (2021). *AquaDiamond Cloth Media Filter*. https://www.aqua-aerobic.com/filtration/cloth-media/aquadiamond/.

- Aqua-Aerobic Systems, Inc. (2021). *AquaDisk Cloth Media Filter*. https://www.aqua-aerobic.com/filtration/cloth-media/aquadisk/.
- Veolia Water Technologies, Inc. (2021). *Hydrotech Discfilter*. https://www.veoliawatertechnologies.com/en/technologies/hydrotech-discfilter.
- Tchobanoglous, G., Stensel, H. D., Tsuchihashi, R., & Burton, F. L. (2014). *Wastewater Engineering: Treatment and Resource Recovery* (5th ed.). New York, NY: McGraw-Hill Education.
- Water Environment Federation. (2017). *Liquid Stream Fundamentals: Tertiary Filtration*. https://www.wef.org/globalassets/assets-wef/direct-download-library/public/03---resources/wsec-2017-fs-027-mrrdc-lsf-filtration_final.pdf.

# 5. DISINFECTION AND POST-AERATION EVALUATION

## 5.1 Current Capacity Analysis

### 5.1.1 UV Disinfection

Disinfection of filtered effluent is currently accomplished using a Trojan UV4000 ultraviolet light disinfection system that was originally installed in 2001. The design information for the existing Trojan UV4000 system is summarized in **Table 5.1** below.

*Table 5.1 – UV Disinfection Current Capacity Summary*

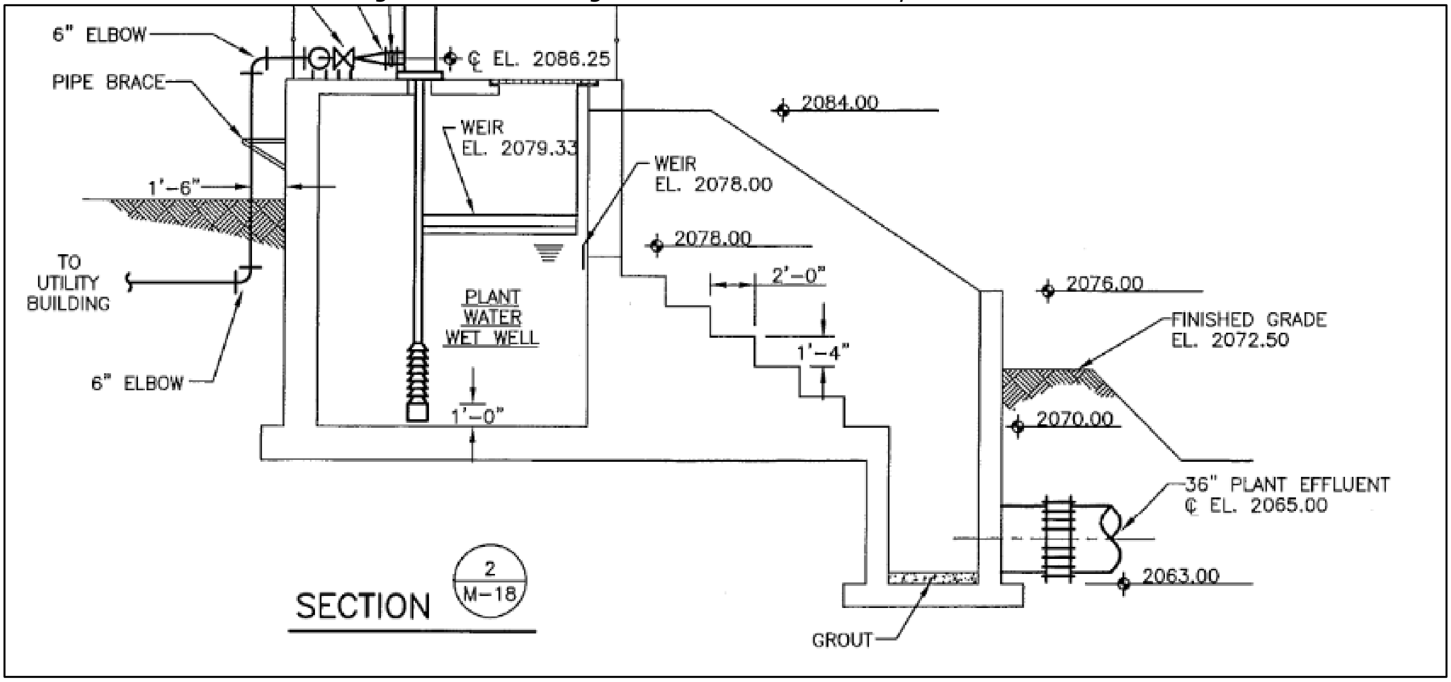
| Parameter | Units | Value |
|---|---|---|
| Peak Hour Design Flow | MGD | 12.0 |
| Design UV Dose | $mJ/cm^2$ | 25 |
| UV Transmission at 253.7 nm | % | 65 |
| Design Suspended Solids Concentration | mg/L | 30 |
| Design Disinfection Standard | FC/100mL (monthly geometric mean) | 200 |
| Design Head Loss at PHF | ft | 2.54 |
| Number of UV Banks | - | 2 |
| Number of Redundant UV Banks at PHF | - | 0 |

The existing UV disinfection system is limited by its peak hour capacity of 12 MGD, which coincides with the design hydraulic capacity of the existing WWTF. The existing UV4000 system was designed to operate with both banks in service at the peak hour design flow. This condition does not meet current NCDEQ Minimum Design Criteria for NPDES Wastewater Treatment Facilities, which requires at least one redundant bank at the peak hour design flow. Replacement of the existing UV disinfection equipment and expansion of the UV disinfection process capacity is required for all future design conditions. Per **Table 2.1**, the UV disinfection process will be required to provide a firm treatment capacity of 15.0 MGD for the 2025 design conditions and 19.5 MGD for the 2040 design conditions. It is recommended that the replacement of the existing UV disinfection equipment be sized based on providing a firm capacity of 15.0 MGD with a minimum of one bank out of service. The UV disinfection replacement should also provide provisions for future expansion to meet the 19.5 MGD firm capacity required for the 2040 design conditions.

### 5.1.2 Cascade Reaeration Steps

Post-aeration of treated effluent from the WWTF is accomplished by cascade reaeration steps, which are attached to the effluent end of the existing disinfection channel. Cascade reaeration steps are the simplest, and usually most cost effective means of post-aeration of treated effluent if sufficient free-fall and land area is available. The existing cascade reaeration steps at the WWTF are 8-feet wide, have a free-fall of 8-feet from the effluent weir to the bottom step, and a total free-fall of 13-feet from the weir to the centerline of the outfall pipe as shown in **Figure 5.1** below.

*Figure 5.1 – Existing Cascade Reaeration Steps Dimensions*




Cascade reaeration steps are typically designed and sized using empirical formulas to determine the overall height of the steps, and general design guidelines for hydraulic loading per foot of step width. The empirical formula used to calculate the total height required for cascade reaeration is shown in **Figure 5.2** below.

*Figure 5.2 – Cascade Reaeration Height Formula*

$$H = \frac{R - 1}{0.11ab(1 + 0.046 * T)}$$

Where:

*H* = Cascade height, feet

*R* = dissolved oxygen deficit ratio

*a* = water-quality parameter equal to 0.8 for a wastewater treatment plant effluent

*b* = weir geometry parameter (weir = 1.0, steps = 1.1, step-weir = 1.3)

*T* = wastewater temperature, °C

The formula for the dissolved oxygen deficit ratio, R, is shown below in **Figure 5.3**.

*Figure 5.3 – Dissolved Oxygen Deficit Ratio Formula*

$$R = \frac{C_S - C_0}{C_S - C}$$

Where:
$R$ = dissolved oxygen deficit ratio
$C_S$ = dissolved oxygen saturation concentration of the wastewater at temperature T, mg/L
$C_0$ = dissolved oxygen concentration of the postaeration influent, mg/L
$C$ = required final dissolved oxygen level after postaeration, mg/L

The design formulas above are most highly dependent upon the temperature of the wastewater, the resulting DO saturation concentration of the wastewater, and the required effluent DO concentration. The DO saturation concentration in water is lowest at warmer water temperatures, meaning it is the most difficult to dissolve oxygen into the treated effluent in the summer. Therefore, cascade reaeration steps are designed for summer conditions, when the free-fall required to produce a specified DO concentration is highest. Historical effluent wastewater temperatures were collected from 2014 through 2019 for the period of July through August to determine the average summer wastewater temperature during the warmest period of the year. These values are summarized in **Table 5.2** below. A summer wastewater temperature of 24 °C was used for this current capacity analysis.

*Table 5.2 – Historical July-August Effluent Wastewater Temperatures*

| Year | Jul-Aug Average Temperature |
|---|---|
| 2014 | 22.89 |
| 2015 | 24.13 |
| 2016 | 24.21 |
| 2017 | 22.32 |
| 2018 | 23.32 |
| 2019 | 23.61 |
| **2014 – 2019 Average** | **23.41** |

The WWTF's current NPDES effluent permit limit for dissolved oxygen is 5.0 mg/L. This limit also applies to the future facility permitted capacity of 6.0 MGD per the current permit provisions. It is expected that this permit limit will also apply to the future facility permitted capacity of 7.8 MGD for 2040 conditions. The weir geometry parameter, *b*, applicable to the existing cascade reaeration steps is 1.1. Typical hydraulic loading rates for cascade reaeration steps range from 100,000 to 500,000 gal/day per foot of step width under AADF conditions. A summary of the capacity calculations for cascade reaeration steps is shown below in **Table 5.3**.

*Table 5.3 – Cascade Reaeration Steps Current Capacity Summary*

| Parameter | Units | Design Year | | |
|---|---|---|---|---|
| | | 2021 | 2025 | 2040 |
| Effluent DO Permit Limit | mg/L | 5.0 | | |
| Summer Wastewater Temperature | °C | 24 | | |
| Winter Wastewater Temperature | °C | 10 | | |
| Initial DO, $C_0$, Assumed[1] | mg/L | 0.5 | | |
| WWTF Elevation Above MSL | ft | 2,070 | | |
| DO Saturation, $C_s$, summer | mg/L | 7.794 | | |
| DO Saturation, $C_s$, winter | mg/L | 10.467 | | |
| R, summer | - | 2.611 | | |
| R, winter | - | 1.823 | | |
| *a* | - | 0.8 | | |
| *b* | - | 1.1 | | |
| **Required Cascade Height, H, summer** | **ft** | **7.91** | | |
| **Required Cascade Height, H, winter** | **ft** | **5.82** | | |
| **Ex. Cascade Height, H, (Weir to Bottom Step)** | **ft** | **8** | | |
| **Ex. Cascade Height, H, (Weir to Outfall Centerline El.)** | **ft** | **13** | | |
| Typical Hydraulic Loading Rate at AADF[2] | gpd/ft | 100,000 – 500,000 | | |
| WWTF AADF | MGD | 3.0 | 4.23 | 5.9 |
| Width of Existing Cascade Steps | ft | 8 | 8 | 8 |
| **Hydraulic Loading Rate at Design Capacity** | **gpd/ft** | **375,000** | **528,750** | **737,500** |
| **Min. Required Cascade Step Width[2]** | **ft** | **6.0** | **8.46** | **11.8** |

*[1]Initial DO concentration after disinfection, prior to post-aeration.*
*[2]Source: Tchobanoglous, Stensel, Tsuchihashi, & Burton, 2014, p. 446*

The existing cascade reaeration steps provide sufficient free-fall to meet effluent DO concentration permit limits of 5.0 mg/L at any design condition. The height of the existing cascade aeration steps is significantly greater than the height required, which will ensure effluent DO limits are significantly exceeded during normal conditions. Historical effluent data, summarized in **Table 5.4** below, confirms average performance is well above effluent DO limits. This post-aeration performance will continue if adequate hydraulic loading conditions are provided for cascade reaeration.

*Table 5.4 – Historical Average Summer Effluent DO Concentrations*

| Year[1] | Average Summer Effluent DO Conc. (mg/L)[2, 3] |
|---|---|
| 2017 | 6.60 |
| 2018 | 6.75 |
| 2019 | 6.86 |
| **2017 – 2019 Average** | **6.74** |

*[1]Effluent DO concentration data unavailable from 2014 - 2016*
*[2]Summer conditions per NPDES permit include April 1 through October 31.*
*[3]Summer effluent DO concentrations indicate worst expected performance due to higher wastewater temperatures.*

In addition, the width of the existing steps is adequately sized for the current WWTF design. The existing steps width is just under the minimum recommended width for the projected 2025 AADF, however, the height of the existing steps is expected to overcome this minor exceedance. Effluent DO concentrations are still expected to exceed the permit limit of 5.0 mg/L at the 2025 AADF. Beyond 2025 design conditions, the width of the existing steps is undersized to provide an adequate hydraulic loading rate. Expansion of the cascade reaeration steps will be required to provide an adequate hydraulic loading rate at the 2040 design conditions. A minimum step width of 12 feet is expected to be required to meet 2040 design conditions. It is recommended that effluent DO concentrations be carefully monitored as the AADF approaches and exceeds the projected 2025 AADF of 4.23 MGD to pinpoint when expansion of the existing cascade reaeration steps will be required.

It is also important to note that the existing cascade reaeration steps are subject to being submerged when flooding of Mud Creek occurs. The current 100-year floodplain elevation of Mud Creek at the WWTF site is 2,076.40 feet, per the most recent FEMA Flood Insurance Rate Map (FIRM) panel. The existing effluent weir to the cascade reaeration steps is located at an elevation of 2,077.20 feet per the most recent survey, completed by McKim & Creed in 2020 as part of this master plan. Flooding that results in partial submergence of the cascade reaeration steps will reduce the effluent DO concentration. However, the effects of flooding of this magnitude are short-lived and are not expected to significantly impede the City's ability to meet monthly average effluent DO permit limits.

### 5.1.3 Hydraulic Limitations

The existing disinfection channel and 36-inch WWTF outfall were evaluated for hydraulic limitations at peak hourly flows above the current facility design to determine if hydraulic improvements will be necessary. The hydraulic profile boundary conditions surrounding the existing disinfection channel are summarized in **Table 5.5** below based on the most recent survey performed by McKim & Creed.

*Table 5.5 – Disinfection Channel Hydraulic Boundary Conditions*

| Parameter | Units | Value |
|---|---|---|
| Upstream Hydraulic Limitation – Tertiary Filters Effluent Weir | ft above MSL | 2,082.3 |
| Downstream Hydraulic Limitation – Cascade Reaeration Weir | ft above MSL | 2,077.2 |
| Maximum Head Available[1] | ft | 5.0 |

*[1]Maximum upstream water level assumed to be 0.1-ft below tertiary filter effluent weir.*

Based on the hydraulic boundary conditions shown above, there appears to be sufficient head available to avoid any hydraulic limitations in the disinfection channel. However, the existing Trojan UV4000 disinfection equipment causes excessive head loss at peak hourly flows due to the design of its reaction chamber. The Trojan UV4000 system design concentrates all flow through a small reaction chamber to ensure efficient UV transmission. Newer UV disinfection equipment designs allow a much large surface area for flow past the UV lamps, which results in significantly less head loss. Most of the hydraulic limitations occurring in the disinfection channel may be remedied by replacing the existing UV disinfection equipment.

In addition to the UV disinfection equipment, the existing disinfection channel weir and the effluent weir to cascade reaeration are both undersized and result in excessive head losses above peak hourly flows of 15 MGD. Above 15 MGD, the upstream water level is greater than one foot above the weir crest elevation for both weirs. Head loss may be allowed to exceed one foot for either weir, however the excessive head losses developed may impact upstream equipment including disinfection equipment and the tertiary filters. The WWTF's outfall to Mud Creek is also hydraulically limited at 15 MGD during flood conditions. During flood conditions, the existing cascade reaeration structure is expected to overflow, which may be remedied by extending the lower portion of the structure wall to match the height of the upper portion. However, even if the cascade reaeration structure wall is extended to prevent overflow, the existing effluent weir will be submerged at plant flows exceeding 15 MGD without upsizing the existing outfall or constructing a redundant outfall.

Hydraulic limitations of the disinfection channel weir to the plant water pump wet well may be alleviated by the design of a new disinfection channel to house new UV disinfection equipment. Hydraulic limitations from the new disinfection channel to the plant water pump wet well may prevented by constructing appropriately sized conduits or connections between the new disinfection channel and the existing plant water wet well. Other considerations may include the use of finger weirs following the new disinfection equipment to provide water level control with additional weir length and reduced head loss in a smaller footprint. As noted above, the existing cascade reaeration steps are undersized for future conditions and will require expansion or replacement regardless of the weir's hydraulic limitations. Hydraulic limitations of the effluent weir to cascade reaeration may be alleviated by expanding the cascade reaeration steps and providing additional weir length to prevent excessive head loss. The existing outfall pipe is recommended to be upsized or a redundant outfall be installed during construction of the cascade reaeration expansion.

## 5.2 Alternatives Evaluation

Based on the findings of the current capacity analyses described above, the following improvements will be required:

- Expansion of the UV disinfection equipment to meet 2025 design conditions of 15 MGD firm capacity at peak hour flow with a minimum of one bank out of service
- Expansion of the UV disinfection equipment to meet 2040 design conditions of 19.5 MGD firm capacity at peak hour flow with a minimum of one bank out of service
- Expansion of the cascade reaeration steps to meet effluent DO permit limits beyond 2025 design conditions
- Upsizing of the 36-inch diameter WWTF outfall, or installation of a parallel outfall to convey peak hour flows beyond 15 MGD at FEMA 100-year flood conditions

Replacement of the existing UV disinfection equipment will require the construction of a new disinfection channel to provide continuous disinfection during construction using the existing equipment. McKim & Creed previously identified two alternatives for construction of a new disinfection channel, which differ based on their location and reuse of the existing NPW wet well and cascade reaeration steps. The two alternatives for improvements to the disinfection and post-aeration processes are described in more detail below.

### 5.2.1 Alternative 1 – New UV Disinfection Channel Between Existing Disinfection Channel and Utility Building

This alternative consists of constructing a new UV disinfection channel No. 2 between the existing disinfection channel and the existing Utility Building as shown in **Figure 5.4** below. The new disinfection channel No. 2 is proposed to be designed to provide a firm treatment capacity of 15 MGD at the peak hour flow conditions with at least one UV bank out of service. Low-pressure high-intensity UV disinfection equipment is proposed to be used, which is proven to be much more energy efficient and result in less head loss than the existing Trojan UV4000 equipment.

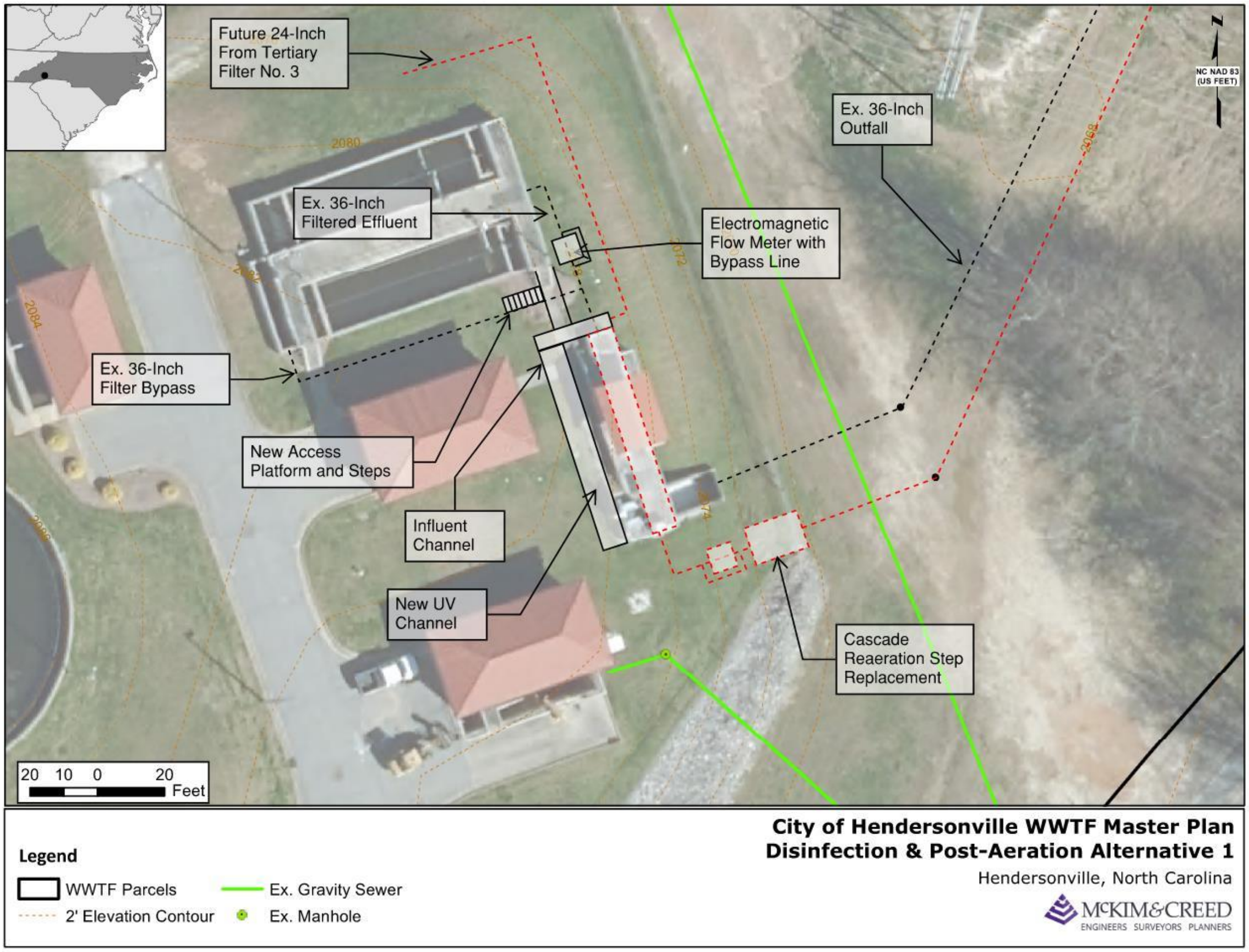
TrojanUVSigna equipment is proposed as the basis of design for the purposes of this master plan due to the City's familiarity with Trojan Technologies' equipment and service availability. Final equipment selection is recommended to be reviewed during detailed design. A new electromagnetic flow meter, meter vault, and bypass line are recommended to be provided on the tertiary filter effluent line to improve the accuracy of effluent flow metering and online UV dose calculation. The existing UV disinfection equipment may continue to be maintained and used for redundancy under the 2025 conditions or may be demolished and the existing channel readied for future retrofit with new UV disinfection equipment. For the purposes of this alternatives evaluation, it was assumed that the existing Trojan UV4000 system would remain and continue to be maintained until additional capacity is required beyond the 2025 design conditions. When operating multiple disinfection channels in parallel, it is recommended that inlet conditions are considered to ensure relatively even flow splitting between channels. Control valves may be installed on inlet piping to each channel, or a common inlet channel may be constructed with isolation gates. It was assumed that a common inlet channel would be constructed to limit head losses that would be associated with throttling control valves.

The effluent chamber of the new UV disinfection channel is proposed to be connected to the existing NPW pump wet well by cutting in a new gate or piping connection. This will allow continued use of the NPW pump wet well and cascade reaeration steps to meet the requirements of the 2025 design conditions.

Additional treatment capacity may be provided in the future by retrofitting the existing UV disinfection channel with new TrojanUVSigna disinfection equipment matching the 15 MGD firm capacity of disinfection channel No. 2. Both channels may be operated in parallel in the future at approximately 65% capacity each to meet the 2040 design conditions at a peak hour flow of 19.5 MGD. One channel would be completely redundant under normal operating conditions to allow a channel to be removed from service for cleaning and maintenance. The retrofit of the existing UV disinfection channel is expected to require major modifications to the existing structure due to the equipment submergence requirements and the hydraulic profile of the existing WWTF. It is expected that the bottom elevation of the disinfection channel must be lowered 1 to 2 feet to provide the submergence required for the equipment. Future improvements to meet the 2040 design conditions also include replacement of the existing cascade reaeration steps with new cascade steps that are at least 12 feet wide, and replacement of the existing 36-inch diameter outfall with at least a 42-inch diameter outfall.

Per the recommendations of Technical Memorandum No. 1, the construction of the new disinfection channel No. 2 is assumed to include new solid removable grating on all new and existing channels, and construction of a clear span shelter structure over both disinfection channels.

*Figure 5.4 - Disinfection and Post-Aeration Alternative 1 Proposed Layout*




Advantages of this alternative include:

- Maximizes reuse of existing infrastructure by maintaining the existing disinfection channel, NPW pump wet well, and cascade reaeration steps up to a permitted capacity of 6.0 MGD
- Alleviates existing hydraulic limitations

- Immediate improvements avoid new development within the FEMA 100-year floodplain and permitting requirements associated
- Minimizes immediate term capital costs

Disadvantages of this alternative include:

- Multiple WWTF shutdowns are expected to be required during construction for electrical, piping, and structural modifications
- May require major modifications to the existing disinfection channel to allow future equipment retrofit

The conceptual cost opinion for Alternative 1 is summarized below in **Table 5.6**. The conceptual cost opinion includes costs of the immediate installation of new disinfection channel No. 2, and future construction to retrofit the original disinfection channel with new UV equipment and construct new cascade reaeration steps and a new outfall. Estimated annual O&M costs for Alternative 1 at current and future conditions are summarized in **Table 5.7** and **Table 5.8** below.

*Table 5.6 – Disinfection and Post-Aeration Alternative 1 Cost Opinion*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $1,170,000 |
| 2 | Mechanical | $234,000 |
| 3 | Electrical | $234,000 |
| 4 | Instrumentation | $149,000 |
| 5 | Structural | $746,000 |
| 6 | Civil | $421,000 |
| 7 | Mobilization & Demobilization | $119,000 |
| 8 | Indirect Costs | $143,000 |
| 9 | General Conditions & Contractor Markup | $688,000 |
| 10 | 30% Contingency | $1,187,000 |
| 11 | Engineering, Legal, & Administration | $786,000 |
| | **Total Cost Opinion** | **$5,927,000** |

*Table 5.7 – Disinfection and Post-Aeration Alternative 1 – Annual O&M Costs – 2021*

| Item | Annual Cost |
|---|---|
| Maintenance | $12,000 |
| Electricity | $8,000 |
| **TOTAL** | **$20,000** |

*Table 5.8 – Disinfection and Post-Aeration Alternative 1 – Annual O&M Costs – 2040*

| Item | Annual Cost |
|---|---|
| Maintenance | $24,000 |
| Electricity | $11,000 |
| **TOTAL** | **$35,000** |

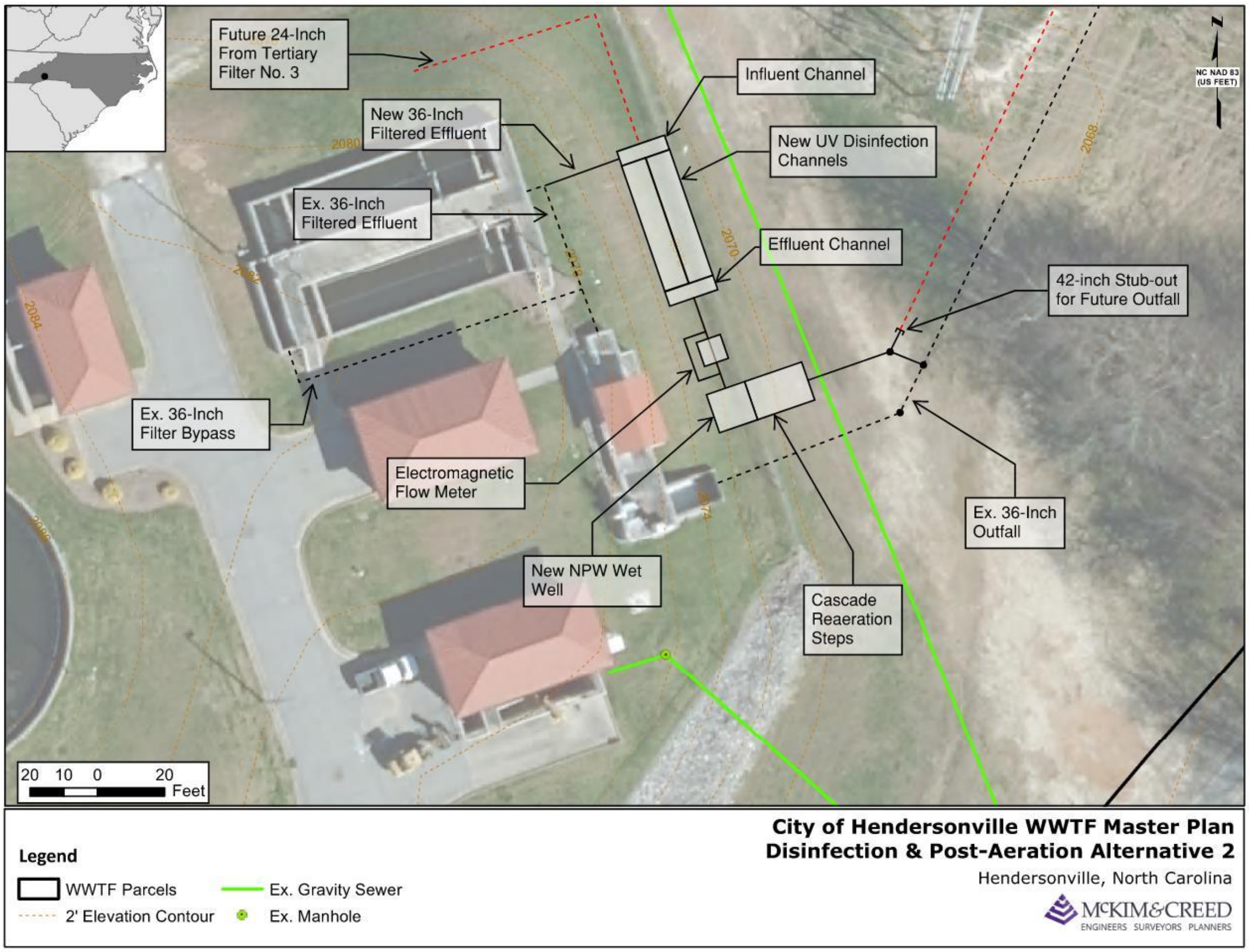
### 5.2.2 Alternative 2 – New Disinfection Channel and Post-Aeration East of the Existing Channel

Disinfection and Post-Aeration Alternative 2 consists of the complete replacement of the existing disinfection channel, NPW pump wet well, cascade reaeration steps, and WWTF outfall as shown in **Figure 5.5** below. This alternative is considered to provide new facilities tailored to the exact needs of the WWTF and to provide more flexibility for future expansion beyond the 2040 design conditions. Under this alternative it is assumed that a new disinfection structure, effluent flow meter vault, NPW pump wet well, and cascade reaeration steps would be constructed immediately to the east of the existing tertiary filters and disinfection channel. The new disinfection structure is proposed to consist of low-pressure high-intensity UV disinfection equipment similar to the TrojanUVSigna system installed in two parallel open concrete channels. Each channel is proposed to be rated for a firm capacity of 15 MGD at peak hour flow with a minimum of one UV bank out of service, for a total firm capacity of 30 MGD with both channels in service. Two channels are proposed to provide complete system redundancy at the 15 MGD peak hour flow associated with the WWTF permitted capacity of 6.0 MGD. The 2040 design conditions at a peak hourly flow rate of 19.5 MGD may be met with both channels operating at approximately 65% of firm capacity. Space for a third parallel open concrete disinfection channel may be provided for future expansion beyond 2040 design conditions.

A new electromagnetic flow meter, meter vault, and bypass line are proposed to be constructed following the disinfection structure to provide accurate measurement of effluent flow and online UV dose calculation. A new NPW pump wet well and cascade reaeration structure is proposed to be constructed following the new effluent meter vault. New NPW pumps sized for current and future demands are proposed to be installed, with electromagnetic flow meters on the pump discharge lines to subtract recycled NPW flows from the WWTF effluent flow measurement. The new cascade reaeration steps are proposed to be a minimum of 12-feet wide to meet 2040 design conditions described in the current capacity evaluation section above. The length and height of the cascade reaeration steps is proposed to be designed to match the existing structure. Space may be provided for future expansion of the cascade reaeration steps beyond the 2040 design conditions. The outfall from the new cascade reaeration structure is proposed to be connected to the existing 36-inch diameter outfall, with a stub-out for future replacement of the existing outfall. The future outfall replacement is assumed to be at least 42-inches in diameter.

The new structures proposed in alternative 2 are located within the FEMA 100-year floodplain and partially within the floodway of Mud Creek. This alternative will require site fill to raise the ground elevation above the FEMA 100-year flood elevation of 2,076.4 feet. Floodplain development permitting, flood modeling, and a LOMR or CLOMR will be required for this alternative.

*Figure 5.5 - Disinfection and Post-Aeration Alternative 2 Proposed Layout*




Advantages of this alternative include:

- New disinfection and post-aeration facilities catered to the exact needs of current and future treatment capacities
- Greater flexibility in disinfection channel design and equipment selection
- Simpler effluent flow metering

- Lower head loss expected

Disadvantages of this alternative include:

- Greater immediate capital costs to construct completely new disinfection and post-aeration facilities
- Expected to require a completely new electrical feed to the disinfection equipment
- Significant grading/fill required within the FEMA 100-year floodplain and floodway of Mud Creek
- Construction within FEMA 100-year floodplain and floodway of Mud Creek will require additional permitting to acquire a floodplain development permit and a LOMR/CLOMR
- Construction within FEMA 100-year floodplain and floodway of Mud Creek may result in significant construction delays and potential for additional cost due to frequent flooding of project site

The conceptual cost opinion for Alternative 2 is summarized below in **Table 5.9**. The conceptual cost opinion includes costs of the immediate installation of the new disinfection channels, effluent flow metering, NPW pump wet well, and cascade reaeration steps, and future construction to construct a new outfall. Estimated annual O&M costs for Alternative 2 at current and future conditions are summarized in **Table 5.10** and **Table 5.11** below.

*Table 5.9 – Disinfection and Post-Aeration Alternative 2 Cost Opinion*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $1,209,000 |
| 2 | Mechanical | $242,000 |
| 3 | Electrical | $242,000 |
| 4 | Instrumentation | $121,000 |
| 5 | Structural | $541,000 |
| 6 | Civil | $609,000 |
| 7 | Demo | $50,000 |
| 8 | Mobilization & Demobilization | $132,000 |
| 9 | Indirect Costs | $145,000 |
| 10 | General Conditions & Contractor Markup | $693,000 |
| 11 | 30% Contingency | $1,196,000 |
| 12 | Engineering, Legal, & Administration | $467,000 |
| | **Total Cost Opinion** | **$5,647,000** |

*Table 5.10 – Disinfection and Post-Aeration Alternative 2 – Annual O&M Costs – 2021*

| Item | Annual Cost |
|---|---|
| Maintenance | $25,000 |
| Electricity | $8,000 |
| **TOTAL** | **$33,000** |

*Table 5.11 – Disinfection and Post-Aeration Alternative 2 – Annual O&M Costs – 2040*

| Item | Annual Cost |
|---|---|
| Maintenance | $25,000 |
| Electricity | $11,000 |
| **TOTAL** | **$36,000** |

## 5.3 Recommendations

Improvements to the UV disinfection equipment are needed immediately to improve equipment reliability and expand treatment capacity per the recommendations of Technical Memorandum No. 1, based on the current capacity of the existing equipment, and based on recent equipment failures reported by City staff. Improvements to the cascade reaeration steps are required beyond an AADF of 4.23 MGD to ensure effluent dissolved oxygen concentrations meet permit limits. Improvements to the existing outfall pipe are required beyond peak flows of 15 MGD under FEMA 100-year flood conditions to alleviate hydraulic limitations. Based on these needs, two alternatives were evaluated to provide immediate improvements to UV disinfection and to meet future needs for post-aeration and effluent disposal.

Alternative 1 consists of the construction of a new UV disinfection channel and disinfection equipment between the existing disinfection channel and utility building in the immediate term. A new flow meter is also proposed to be provided on the filtered effluent pipe to improve effluent flow metering in the near term. Future construction under alternative 1 consists of retrofitting the existing UV disinfection channel with new UV disinfection equipment to match the new equipment installed in the immediate term. New cascade reaeration steps, additional effluent flow metering, and replacement of the existing outfall pipeline would also be constructed in the future term. Future construction is assumed to occur in the year 2040.

Alternative 2 consists of the complete replacement of the existing UV disinfection channel, NPW pump wet well, and cascade reaeration steps to the east of the existing structures within the FEMA 100-year floodplain and floodway of Mud Creek in the immediate term. Two new UV disinfection channels are provided under this alternative to provide complete redundancy of UV disinfection equipment up to a peak hour flow of 15 MGD. Immediate term construction would also include a new effluent flow meter and vault between the disinfection channels and NPW pump wet well. Future construction under alternative 2 consists of the replacement of the existing outfall pipeline and is assumed to occur in the year 2040.

Table 5.12 below summarizes the total capital costs, annual O&M costs, and the total net present value of each alternative. As seen below, Alternative 1 has a slightly higher total capital cost, however, the total net present value is significantly lower than Alternative 2. Alternative 1 has a lower total NPV despite a higher total capital cost due to the lower immediate term capital costs. Immediate term capital costs for alternative 1 were significantly lower due to maximizing the use of the existing UV disinfection equipment, NPW pump wet well, and cascade reaeration steps.

*Table 5.12 – Cost Comparison of Disinfection and Post-Aeration Alternatives*

| Alternative | Capital Costs | 2021 Annual O&M Cost | 2040 Annual O&M Cost | Total NPV |
|---|---|---|---|---|
| **Alternative 1: New Channel Between Existing Channel and Utility Building** | $5,927,000 | $20,000 | $35,000 | $4,351,000 |
| **Alternative 2: New Disinfection Channels and Post-Aeration** | $5,647,000 | $33,000 | $36,000 | $5,434,000 |

Based on these results, alternative 1 is the recommended alternative for implementation. Separate conceptual cost opinions for the immediate term and future term improvements recommended under alternative 1 are presented in **Table 5.13** and **Table 5.14** below.

*Table 5.13 – Disinfection and Post Aeration Alternative 1 Immediate Term Cost Opinion*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $585,000 |
| 2 | Mechanical | $117,000 |
| 3 | Electrical | $117,000 |
| 4 | Instrumentation | $90,000 |
| 5 | Structural | $457,000 |
| 6 | Civil | $39,000 |
| 7 | Mobilization & Demobilization | $57,000 |
| 8 | Indirect Costs | $67,000 |
| 9 | General Conditions & Contractor Markup | $322,000 |
| 10 | 30% Contingency | $556,000 |
| 11 | Engineering, Legal, & Administration | $393,000 |
| | **Total Cost Opinion** | **$2,800,000** |

*Table 5.14 – Disinfection and Post Aeration Alternative 1 Future Term (2040) Cost Opinion*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $585,000 |
| 2 | Mechanical | $117,000 |
| 3 | Electrical | $117,000 |
| 4 | Instrumentation | $59,000 |
| 5 | Structural | $289,000 |
| 6 | Civil | $382,000 |
| 7 | Demo | $50,000 |
| 8 | Mobilization & Demobilization | $62,000 |
| 9 | Indirect Costs | $76,000 |
| 10 | General Conditions & Contractor Markup | $366,000 |
| 11 | 30% Contingency | $631,000 |
| 12 | Engineering, Legal, & Administration | $393,000 |
| | **Total Cost Opinion** | **$3,127,000** |

## 5.4 References


- Tchobanoglous, G., Stensel, H. D., Tsuchihashi, R., & Burton, F. L. (2014). *Wastewater Engineering: Treatment and Resource Recovery* (5th ed.). New York, NY: McGraw-Hill Education.


# 6. BIOSOLIDS SYSTEM EVALUATION

## 6.1 Purpose and Background

Solids handling and disposal practices at the WWTF have been previously evaluated as part of the *Solids Management Plan Evaluation Report* prepared by McKim & Creed. The previous report provided recommendations for the implementation of a new biosolids thermal dryer at the WWTF, along with guidance for potential disposal outlets. The purpose of this section is to support the previous recommendations by evaluating the capacity of existing solids handling processes and equipment that are to remain, and to evaluate alternatives for improvements to process capacity, operations, and flexibility. The goal of these evaluations is to continue to provide sludge thickening and dewatering processes to support the operation of a new thermal drying facility. Technical Memorandum No. 1 identified repair and replacement needs for the existing thickening and dewatering processes. Improvements were also recommended to provide aerated thickened sludge storage prior to dewatering operations to provide additional flexibility in dewatering schedules, and to reduce the potential for foul odors in the thickeners. Evaluations of alternatives to accomplish these recommended improvements are documented in this section.

## 6.2 Sludge Thickening

### 6.2.1 Current Capacity Analysis

The existing gravity thickeners at the WWTF are each 50-foot in diameter and have a side water depth (SWD) of 13-feet. Waste activated sludge (WAS) is pumped from the secondary clarifiers to the two existing gravity thickeners via two parallel 8-inch diameter force mains. WAS is currently pumped to the gravity thickeners periodically, for approximately 8 to 10 hours every other day. As noted previously in Technical Memorandum No. 1, the existing gravity thickeners are currently utilized in a non-traditional sense to thicken and provide storage of thickened WAS prior to dewatering. It was also previously recommended that aerated sludge storage be provided prior to dewatering to provide increased operational flexibility in dewatering schedules due to the lack of storage in the gravity thickeners. The current capacity of the existing gravity thickeners was evaluated based on the assumption that aerated thickened sludge storage will be provided, allowing the gravity thickeners to operate in a traditional manner. Gravity thickeners are typically operated very similarly to secondary clarifiers, with relatively consistent flows to the thickeners, and frequent or continuous thickened sludge withdrawal.

The current capacity of the existing gravity thickeners, and all other biosolids handling processes, is evaluated based on the maximum month sludge production estimated to occur at the facility design flow. Maximum month conditions are selected as the basis of design for these systems to ensure they are sized to handle maximum sustained solids loading. In addition, current capacity is evaluated assuming one unit is out of service to ensure adequate redundancy is provided for all unit processes. Maximum month conditions were evaluated for the facility's current rated capacity as well as the projected capacity required for the 2040 planning horizon. Maximum month WAS projections for both design conditions were estimated using the BioWin process model. **Table 6.1** below summarizes the maximum month sludge production estimates used to evaluate the capacity of the existing gravity thickeners.

*Table 6.1 – Maximum Month Sludge Production Estimates*

| Parameter | Units | Design Year | |
|---|---|---|---|
| | | 2021 | 2040 |
| WAS Flowrate | gpd | 129,000 | 188,000 |
| WAS Concentration | % dry solids | 0.8% | 0.95% |
| WAS Mass Rate | dry lbs/day | 8,622 | 14,836 |

The design of gravity thickeners is governed by the solids loading rate (SLR) to the thickener, and the surface overflow rate (SOR). Typical design guidelines for separate thickening of WAS are reported in **Table 6.2**, below.

*Table 6.2 – Typical Design Criteria for Gravity Thickeners for Waste Activated Sludge*

| Parameter | Units | Recommended Design Range |
|---|---|---|
| Maximum Surface Overflow Rate[1] | gal/day/ft$^2$ | 100 - 200 |
| Solids Loading Rate[1] | lbs/day/ft$^2$ | 5 – 8 |

[1]*Source: Tchobanoglous, Stensel, Tsuchihashi, & Burton, 2014, pp. 1489-1490*

**Table 6.3** below summarizes the physical characteristics and loading rates for the existing gravity thickeners under current and 2040 conditions, at both AADF and MMF. The loading rates shown below are based on one gravity thickener in operation. Under current conditions, the gravity thickeners are appropriately sized to maintain an acceptable SLR for maximum month conditions with one thickener out of service. However, the maximum month SOR is well below the recommended design range for current operating conditions. Additionally, the SOR and SLR for one gravity thickener at current AADF conditions are both far below the recommended design ranges. The current average and max month loading conditions will result in longer than typical sludge and hydraulic retention times that may result in septic conditions and foul odors. These issues can be mitigated by providing dilution water to be added to the gravity thickeners to maintain adequate hydraulic loading and limit odors. The facility's NPW supply is generally preferred to be used for dilution water, if needed, which would be recycled to the head of the facility with the thickener overflow. It is also recommended to maintain a sludge blanket depth of 2 to 4 feet to reduce sludge residence time in the thickener and prevent septic conditions. Sludge blanket depths should be maintained on the lower end of this range during summer conditions to limit odor concerns.

The existing gravity thickeners are adequately sized for average loading at future 2040 conditions with one thickener out of service. The predicted 2040 maximum month SLR is expected to approach the capacity limits of a single gravity thickener. It is recommended that operations staff monitor thickening performance during sustained loading events at future conditions to determine when it may be necessary to operate both gravity thickeners in parallel. In addition, operation staff may be able to anticipate seasonal variations in sludge production and act preemptively to ensure adequate thickening capacity. Operating staff may preemptively increase the frequency of dewatering operations to reduce the sludge blanket level in the thickeners to provide additional capacity for periods of high sludge production. Future maximum day loading conditions are expected to require operation of both gravity thickeners in parallel to provide adequate thickening capacity. It is expected that both gravity thickeners will be operated in parallel during normal operation at the 2040 design conditions. Therefore, 2040 maximum day loading conditions are not expected to require significant operational modifications. Based on the information presented in **Table 6.3** below, no additional gravity thickeners are expected to be required for the 2040 design conditions.

*Table 6.3 – Gravity Thickeners Current Capacity Summary*

| Parameter | Units | Design Year | |
|---|---|---|---|
| | | 2021 | 2040 |
| Number of Gravity Thickeners | - | 2 | |
| Diameter, each | ft | 50 | |
| Side Water Depth | ft | 13 | |
| Surface Area per Thickener | $ft^2$/thickener | 1,963.5 | |
| Total Surface Area | $ft^2$ | 3,927.0 | |
| Surface Overflow Rate, AADF[1] | gal/day/$ft^2$ | 38.4 | 77.4 |
| Solids Loading Rate, AADF[1] | lbs/day/$ft^2$ | 2.6 | 5.5 |
| Surface Overflow Rate, Max Month[1] | gal/day/$ft^2$ | 65.7 | 95.7 |
| Solids Loading Rate, Max Month[1] | lbs/day/$ft^2$ | 4.4 | 7.6 |

*[1]Loading rates based on one gravity thickener in operation.*

## 6.2.2 Alternatives Evaluation

As noted previously, improvements are recommended to provide aerated thickened sludge storage and improve dewatering operational flexibility. The City currently operates the dewatering belt filter presses approximately 5 to 6 days per month on average. Based on current operations, a thickened sludge storage capacity of at least 5 days is required to accommodate this schedule. There are two primary alternatives to provide TWAS storage, which include the construction of new TWAS storage tanks, or implementation of gravity belt thickening and conversion of the existing gravity thickeners to TWAS storage tanks. These two alternatives are described in more detail in the following sections.

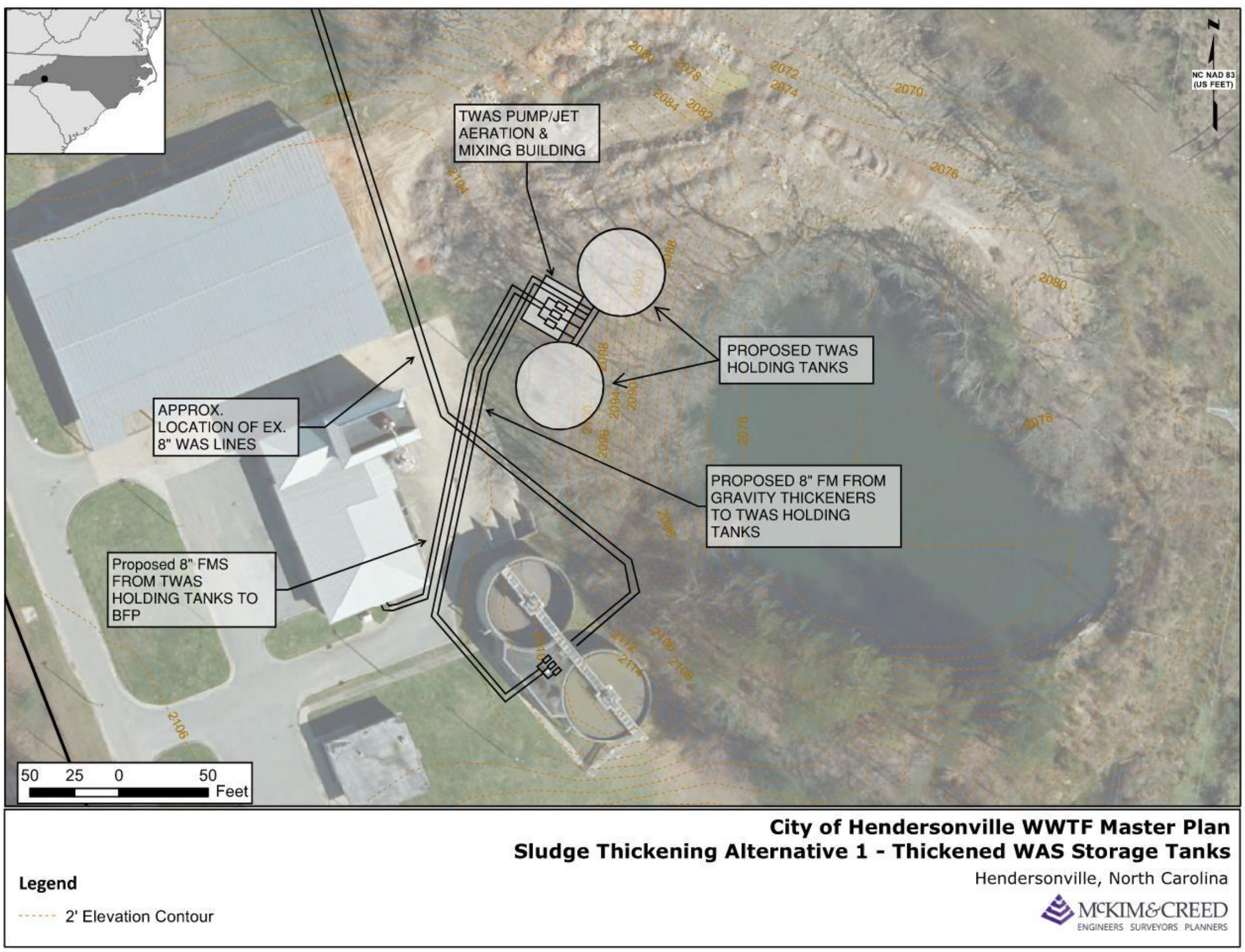
### 6.2.2.1 *Alternative 1 – Construct New Thickened WAS Storage Tanks*

This alternative maintains the use of the existing gravity thickeners and requires the construction of new aerated thickened WAS storage tanks prior to dewatering, as shown in **Figure 6.1** below. WAS will continue to be pumped to the existing gravity thickeners where it will be thickened to approximately 3% solids. The implementation of thickened WAS storage tanks will allow the existing gravity thickeners to operate as a flow through process with continuous or frequent thickened WAS (TWAS) withdrawal, and continuous overflow of supernatant which will be returned to the in-plant drain system for further treatment. The existing belt filter press feed pumps will be repurposed to pump TWAS to the new aerated TWAS storage tanks. TWAS suction piping to the existing pumps will be modified as noted in Technical Memorandum No. 1 to allow any of the three existing pumps to withdraw TWAS from either gravity thickener. The existing gravity thickeners will also require concrete rehabilitation as noted in Technical Memorandum No. 1. At least two new aerated TWAS storage tanks are recommended to be constructed to provide redundancy in the event one tank needs to be taken out of service for cleaning or maintenance. **Table 6.4** below summarizes required tank sizing assuming a 15-foot working depth for both 5 days and 10 days of storage. It is recommended that two 50-foot diameter TWAS storage tanks be constructed to provide nearly 5 days of TWAS storage at the 2040 maximum month design conditions with one tank in service, and nearly 10 days of storage with both tanks in service.

*Table 6.4 – Comparison of TWAS Storage Tank Sizing*

| Parameter | Units | Design Year | |
|---|---|---|---|
| | | 2021 | 2040 |
| 5 day tank diameter (1 tank) | ft | 40 | 52.5 |
| 5 day tank diameter (2 tanks) | ft | 28.3 | 37.1 |
| 5 day tank diameter (3 tanks) | ft | 23.1 | 30.3 |
| 10 day tank diameter (1 tank) | ft | 56.6 | 74.2 |
| 10 day tank diameter (2 tanks) | ft | 40 | 52.5 |
| 10 day tank diameter (3 tanks) | ft | 32.7 | 42.8 |

*Figure 6.1 - Sludge Thickening Alternative 1 Proposed Layout*



Mixing and aeration equipment will be required within the aerated TWAS storage tanks to prevent septic conditions and odor generation, and to keep the thickened sludge in suspension. Mixing equipment that provides a strong motive force, supplied by either pumping or mechanical mixers, is recommended for this application due to the high solids content of the sludge. Liquid level in the TWAS storage tanks is expected to vary significantly based on dewatering operations. As a result, it is recommended to decouple mixing and aeration systems so that they may be adjusted separately with varying tank levels to reduce energy demands. The preferred means to accomplish this is to use either mechanical mixers and separate coarse bubble aeration equipment, or by using a jet aeration and mixing system.

A jet aeration and mixing system consists a of a pump (either submersible or externally mounted), a blower unit mounted externally, and a jet nozzle header installed within the tank. An example of a jet aeration and mixing system installed in a basin is shown in **Figure 6.2** below.

*Figure 6.2 - Example Jet Aeration System*

*Source: Fluidyne Corporation, http://www.fluidynecorp.com/aeration-mixing/Jet-Aeration.aspx*

Each nozzle has a primary (inner) and secondary (outer) nozzle. A chopper pump recirculates TWAS from the storage tank into the jet aeration and mixing header and ejects it back into the tank through the primary nozzles at a high velocity to induce a high rate of mixing. Air is fed into the secondary nozzles by a blower, through an air header that runs along the liquid header. The air is mixed with the liquid within the outer nozzle, where the high velocity of the liquid jet shears the air flow into fine bubbles that are entrained with the jet plume. The jets can be operated without air being fed through the outer nozzle, and air flow and liquid flow can be adjusted separately to accomplish varying requirements for mixing and aeration. The fine air bubbles and strong turbulence produced by the jets results in a high oxygen transfer efficiency, which reduces the mass rate of air required to be supplied, which in turn reduces the size of the blowers needed to provide air to the process.

A jet aeration system was assumed to be used for the purposes of this evaluation since it requires no moving equipment within the tanks, has a high oxygen transfer efficiency, and uses less energy than other aeration systems. New pumping facilities will also be required to pump TWAS from the aerated TWAS storage tanks to the belt filter presses for dewatering. The pump facility will be located between the TWAS storage tanks and piped such that either tank can feed any pump.

Advantages of this alternative include:

- Maintains the use of the existing gravity thickening equipment, which is simple to operate and maintain
- Gravity thickening has lower operating costs compared to other thickening technologies
- Simple construction with many options for TWAS storage tank materials and configuration
- TWAS holding tanks requires minimal operator attention, consisting primarily of occasional pump maintenance and occasional tank cleanings

Disadvantages of this alternative include:

- Lower thickened WAS solids concentration compared to other thickening technologies, results in larger thickened WAS storage tanks to accomplish the same storage duration
- Thickening performance using gravity thickeners can be highly variable
- May require addition of polymer prior to gravity thickeners to ensure adequate thickening at higher sludge production rates
- May require modifications to the tanks to include dilution water to improve stability of thickening performance and limit odor generation
- Continuous or frequent intermittent thickened sludge withdrawal from the gravity thickeners may result in "rat-holing" if thickened solids are drawn off too quickly. This results in a cone of depression in the sludge blanket where unthickened sludge short-circuits to sludge withdrawal

The estimated conceptual cost opinion for Alternative 1 is included below in **Table 6.5**.

*Table 6.5 Estimated Sludge Thickening Alternative 1 Cost Opinion*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $302,000 |
| 2 | Mechanical | $61,000 |
| 3 | Electrical | $61,000 |
| 4 | Instrumentation | $31,000 |
| 5 | Structural | $1,370,000 |
| 6 | Civil | $590,000 |
| 7 | Mobilization & Demobilization | $97,000 |
| 8 | Indirect Costs | $115,000 |
| 9 | General Conditions & Contractor Markup | $553,000 |
| 10 | 30% Contingency | $954,000 |
| 11 | Engineering, Legal, & Administration | $767,000 |
| | **Total Cost Opinion** | **$4,901,000** |

**Table 6.6** and **Table 6.7** show the annual O&M costs for current and future production, respectively.

*Table 6.6 Sludge Thickening Alternative 1 - Annual O&M Costs - 2021*

| Item | Annual Cost |
|---|---|
| Maintenance | $7,000 |
| Electricity | $17,000 |
| **TOTAL** | **$24,000** |

*Table 6.7 Sludge Thickening Alternative 1 - Annual O&M Costs - 2040*

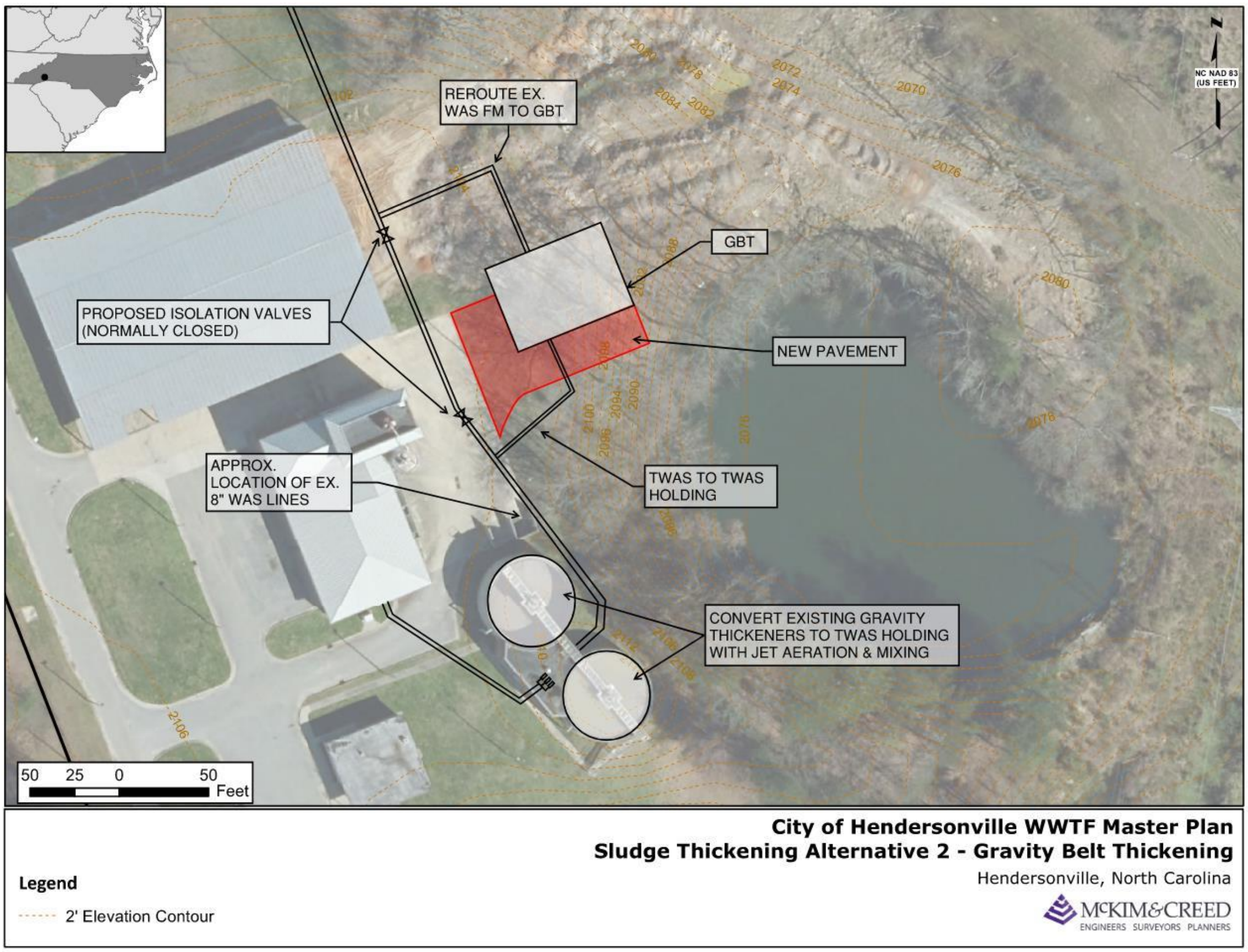
| Item | Annual Cost |
|---|---|
| Maintenance | $7,000 |
| Electricity | $34,000 |
| **TOTAL** | **$41,000** |

#### 6.2.2.2 *Alternative 2 – Construct New Thickening Facility and Retrofit Existing Thickeners for Aerated Thickened WAS Storage*

Despite the capacity of the existing gravity thickeners, gravity thickening is well known to be best suited for thickening of primary sludge, or a combination of primary sludge and WAS. Gravity thickening of WAS is generally less efficient and produces a thickened sludge concentration of 2% to 3% solids. This typical concentration range is consistent with operating data obtained for the City's gravity thickeners. Conversely, other thickening technologies such as gravity belt thickeners and rotary drum thickeners can produce thickened WAS concentrations of 4% to 7% solids, in a smaller footprint. Improved thickening to achieve a higher thickened sludge concentration will reduce the volume required for storage of thickened WAS.

This alternative consists of the construction of a new thickening facility using either gravity belt thickeners or rotary drum thickeners to achieve a thickened WAS solids concentration of approximately 5%. Thickened WAS from the new thickening facility will be pumped to the existing gravity thickener tanks, which will be retrofitted as aerated thickened sludge holding tanks prior to dewatering. The existing belt filter press feed pumps will continue to be used to feed thickened WAS from the existing gravity thickener tanks after retrofitting to aerated thickened sludge storage tanks. A preliminary layout map for this alternative is shown in **Figure 6.3** below.

*Figure 6.3 - Sludge Thickening Alternative 2 Proposed Layout*




For the purposes of this evaluation, it is assumed that a new thickening facility would be constructed using gravity belt thickeners (GBTs). An example GBT is shown in **Figure 6.4**. If Alternative 2 is selected, detailed equipment evaluation and final selection should be evaluated further during preliminary engineering evaluations. GBTs were assumed to be used for this evaluation because:

- Capital and operating costs for GBTs and RDTs are expected to be very similar.
- GBTs are widely used for separate thickening of WAS to solids concentrations of approximately 5%, with solids capture ranging from 90 to 98% (Tchobanoglous, Stensel, Tsuchihashi, & Burton, 2014, p. 1496).
- It is expected that operational staff will quickly adjust to operation of GBTs due to familiarity with the existing BFPs.
- GBTs may be semi-automated to allow the operation of GBTs to be coupled with sludge wasting operations.

*Figure 6.4 Gravity Belt Thickener*

Periodic operator attention is required for GBTs to ensure stable operations. Sludge loading rates on the upper end of manufacturer recommendations will require increased operator attention. New thickened sludge pumping equipment will be required for this alternative to transport thickened WAS to aerated thickened WAS storage in the retrofitted existing gravity thickener tanks. Positive displacement pumps are recommended for this application. Applicable pump types include double disc pumps (similar to the existing BFP feed pumps), plunger pumps, progressive cavity, diaphragm, or rotary lobe pumps. Jet aeration equipment is also assumed to be used in this alternative for mixing and aeration in the aerated TWAS storage tanks. Concrete rehabilitation will be required for the existing gravity thickener tanks as noted in Technical Memorandum No. 1.

Because the GBTs would be directly fed by the WAS/RAS pump station, they would operate in conjunction with the solids wasting schedule, which has historically been approximately 8 hours a day, 7 days a week based of WWTF operations data. Assuming 200 GPM/meter width of belt for each GBT, **Table 6.8** shows the number of GBTs of varying size for the current and 2040 AADF design conditions. Max month conditions can be addressed by increasing the thickening shift length by a few hours as needed.

*Table 6.8 - Number of GBTs Needed for 8 hr/day, 7 day/week Sludge Wasting Schedule*

| Parameter | Units | Design Year | |
|---|---|---|---|
| | | 2021 | 2040 |
| WAS Flow to GBT, AADF | gpd | 75,400 | 152,000 |
| WAS Flow to GBT, AADF for 8 hour thickening shift | gpm | 157 | 317 |
| Number of 1-meter wide GBTs needed[1] | - | 1 | 2 |
| Number of 2-meter wide GBTs needed[1] | - | 1 | 1 |

*[1]Assumes 200 gpm/meter width of thickening belt (Water Environment Federation Manual of Practice 8, 6th Edition, 2018). Number of GBTs indicates duty units, add 1 unit for redundancy.*

Advantages of this alternative include:

- Reduced aerated TWAS storage volume required due to increased solids concentration
- Reduced footprint for sludge thickening process
- Easily expandable for future thickening capacity by installing additional GBTs or RDTs
- Potential for improved dewatering capabilities due to greater TWAS concentration; increased dewatered cake solids concentrations will result in reduced natural gas demand for thermal drying

Disadvantages of this alternative include:

- Increased operation and maintenance complexity associated with TWAS pumping, mixing, and aeration due to increased solids concentration
    - Reduction of TWAS concentration may be accomplished to alleviate these issues, but at the expense of increasing TWAS storage volume requirements
- Increased operating costs due to additional polymer and wash water requirements
- Increased capital costs

The estimated conceptual cost opinion for Alternative 2 is included below in **Table 6.9**Table 2.29**.**

*Table 6.9 Estimated Sludge Thickening Alternative 2 Cost Opinion*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $889,000 |
| 2 | Mechanical | $178,000 |
| 3 | Electrical | $178,000 |
| 4 | Instrumentation | $89,000 |
| 5 | Structural | $2,257,000 |
| 6 | Civil | $472,000 |
| 7 | Demo | $50,000 |
| 8 | Mobilization & Demobilization | $165,000 |
| 9 | Indirect Costs | $194,000 |
| 10 | General Conditions & Contractor Markup | $940,000 |
| 11 | 30% Contingency | $1,624,000 |
| 12 | Engineering, Legal, & Administration | $1,305,000 |
| | **Total Cost Opinion** | **$8,341,000** |

**Table 2.30** and **Table 2.31** show the annual O&M costs for current and future production, respectively.

*Table 6.10 Sludge Thickening Alternative 2 - Annual O&M Costs - 2021*

| Item | Annual Cost |
|---|---|
| Maintenance | $18,000 |
| Electricity | $18,000 |
| Polymer | $10,000 |
| **TOTAL** | **$46,000** |

*Table 6.11 Sludge Thickening Alternative 2 - Annual O&M Costs - 2040*

| Item | Annual Cost |
|---|---|
| Maintenance | $18,000 |
| Electricity | $35,000 |
| Polymer | $20,000 |
| **TOTAL** | **$73,000** |

# 6.3 Sludge Dewatering

## 6.3.1 Current Capacity Analysis

### 6.3.1.1 *Belt Filter Presses*

Dewatering of the sludge produced at the WWTF is accomplished by two SernaTech Sernagiotto Model BPF 2000 WR 15 belt filter presses. Design and O&M information for the existing BFPs was collected from the operation and maintenance manuals provided by the City, summarized in **Table 6.12** below.

*Table 6.12 – Existing Belt Filter Press O&M Information*

| Parameter | Units | Value |
|---|---|---|
| Number of BFP Units | - | 2 |
| Belt low speed | ft/min | 5 |
| Belt high speed | ft/min | 30 |
| Tank Mixer Drive | kW | 1.5 |
| Mixer low speed | rpm | 7 |
| Mixer high speed | rpm | 70 |
| Number of Belts per BFP | - | 2 |
| Belt width (each) | in | 82.7 |
| Belt width (each), metric units | m | 2.10 |
| Belt Length (each) | in | 1,059 |
| Air Consumption | cfm | 4 |
| Air supply pressure (min.) | psig | 85 |
| Wash Water Consumption | gpm | 80 |
| Wash water supply pressure (min.) | psig | 85 |
| Belt motor power (each) | kW | 2.2 |
| Belt motor voltage | V | 460 |
| Belt motor phases | - | 3 |
| Belt motors per BFP | - | 2 |

Design solids and hydraulic loading rates were not referenced in the operation and maintenance manual for the existing equipment. However, the original basis of design for the existing WWTF (*201 Facilities Planning Study, Hendersonville Planning Area*) indicated that each 2 meter belt filter press was designed for a feed capacity of 100 gpm of thickened WAS. Historical operating records were reviewed to confirm the basis of design operating capacity for the BFP. The facility's operating staff records solids wasting, thickened sludge flows, and dewatering rates each dewatering operating day in "sludge wasting reports". Average operating parameters are summarized in **Table 6.13** below. The operating parameters collected from the sludge wasting reports appeared to verify the original basis of design. In review of the sludge wasting reports, it was noted that facility staff typically operate the BFPs with thickened WAS (TWAS) feed rates slightly lower than the basis of design capacity of 100 gpm. Several periods of operation were also noted with TWAS feed rates higher than the basis of design.

*Table 6.13 – Average Dewatering Operational Parameters (2018 – 2019)*

| Parameter | Units | Value |
|---|---|---|
| Average Belt Filter Press Operating Hours per Dewatering Day | hrs/day | 11 |
| Average No. of Dewatering Days per Month | - | 5.6 |
| Average No. of BFPs in Operation | - | 2 |
| Average TWAS Feed Rate per BFP | gpm | 80 |
| Estimated Max. TWAS Feed Rate per BFP | gpm | 123 |

Generally, BFPs are sized based on typical solids and hydraulic loading rates per meter of belt width. Typical values and the estimated range of allowable solids and hydraulic loading rates for the existing BFPs are summarized in **Table 6.14** below.

*Table 6.14 – Typical Belt Filter Press Solids and Hydraulic Loading Rates*

| Parameter | Units | Value |
|---|---|---|
| Typical Solids Loading Rate per Meter Belt Width[1] | lbs/hr/m | 400 - 750 |
| Typical Hydraulic Loading Rate per Meter Belt Width[1] | gpm/m | 50 - 100 |
| Existing BFP Belt Width | m | 2 |
| Estimated Solids Loading Rate per BFP | lbs/hr | 800 – 1,500 |
| Estimated Hydraulic Loading Rate per BFP | gpm | 100 - 200 |

[1]*Source: Tchobanoglous, Stensel, Tsuchihashi, & Burton, 2014, p. 1575*

The range of typical design values for each BFP also corroborates the original basis of design and current BFP operations. It should be noted that the existing gravity thickeners currently achieve a TWAS solids concentration of approximately 3.3% on average. Assuming this TWAS solids concentration and the basis of design hydraulic loading rate of 100 gpm per BFP would result in a solids loading rate per BFP of approximately 1,700 lbs/hr. This estimated solids loading rate based on the original basis of design information exceeds the maximum value of 1,500 lbs/hr based on typical design recommendations. Therefore, the current capacity of the existing BFPs was evaluated based on typical design recommendations with a maximum solids loading rate of 1,500 lbs/hr per BFP.

Current capacity of the existing BFPs is evaluated based on the maximum month sludge loading, similar to the sludge thickening facilities described above. Current capacity was evaluated assuming that gravity thickening remains in operation, which results in the highest hydraulic loading rates due to lowest anticipated thickened WAS solids concentration. Current capacity was evaluated with only one BFP in operation to ensure adequate redundancy if one BFP fails or is out of service for maintenance. Current capacity was also evaluated assuming both BFPs were in operation to resemble normal operating conditions. **Table 6.15** below summarizes the results of these analyses.

*Table 6.15 – Belt Filter Press Current Capacity Summary*

| Parameter | Units | Design Year | |
|---|---|---|---|
| | | 2021 | 2040 |
| WAS Flow, Max Month | gal/day | 129,000 | 188,000 |
| WAS Concentration | mg/L | 8,014 | 9,462 |
| WAS Mass Rate, max month | dry lbs/day | 8,622 | 14,836 |
| Gravity Thickener Capture Rate[1] | % | 90% | 90% |
| TWAS Flow, Max Month, at 3.3% solids | gal/day | 28,195 | 48,514 |
| TWAS Concentration (existing) | mg/L | 33,000 | 33,000 |
| TWAS Mass Rate, max month | dry lbs/day | 7,760 | 13,352 |
| Recommended Maximum SLR per BFP | lbs/hr | 1,500 | 1,500 |
| No. of BFPs in Service | - | 1 | 1 |
| Operating Hours per Week at Max. SLR | hrs/wk | 36.2 | 62.3 |
| Max. Dewatering Days per Week | days/wk | 5 | 5 |
| Required Operating Hours per Day | hrs/day | 7.24 | 12.46 |
| Estimated Hydraulic Loading Rate per BFP | gpm | 91 | 91 |
| No. of BFPs in Service | - | 2 | 2 |
| Operating Hours per Week at Max. SLR | hrs/wk | 18.1 | 31.2 |
| Max. Dewatering Days per Week | days/wk | 5 | 5 |
| Required Operating Hours per Day | hrs/day | 3.62 | 6.23 |
| Estimated Hydraulic Loading Rate per BFP | gpm | 91 | 91 |

[1]*Assumed solids capture rate per Solids Management Plan Evaluation Report*

Based on the information presented above, the existing BFPs provide sufficient capacity for both current and future 2040 maximum month sludge production. However, dewatering with a single BFP at the 2040 conditions requires extended operating hours each operating day. For this analysis, it was assumed that five days per week was the maximum allowable operating conditions based on qualified staff availability. Future 2040 maximum month dewatering requirements can be processed using a single shift for four days a week with both BFPs operational. Under normal operating conditions with both BFPs operational, current maximum month dewatering requirements can be processed over a single shift three days a week. The current maximum month dewatering requirements can be processed over a single shift five days a week with just one BFP in operation. The City may also choose to extend dewatering operations to weekend days at future conditions to reduce operating hours per day if necessary when a single BFP is in use.

Despite the available capacity of the existing BFPs, it is recognized that they are approximately 20 years old, and they will likely reach the end of their useful life prior to 2040. Based on current condition and maintenance practices, it is assumed that the existing equipment will require replacement by 2030. At which time, it is recommended to replace the existing BFPs with equipment that has a slightly higher design capacity to allow more flexibility in dewatering schedules. Newer BFP technology is capable of delivering higher capacities within the same existing footprint. It is assumed the existing equipment will be replaced with new BFPs in place of the existing BFPs. Replacement should be staged to allow continuous dewatering operations. This may be accomplished by replacing one BFP at a time such that at least one BFP is always available for operation, or by temporarily providing contracted dewatering services such that complete replacement may be accomplished over a shorter duration.

Based on the onsite equipment condition assessment, the associated dewatering polymer feed system is also nearing the end of its reliable useful life. It is recommended that the polymer feed system be replaced at the same time as the BFPs to allow SCADA integration with the controls for the new BFPs and improved operational automation.

#### 6.3.1.2 *Dewatered Cake Conveyor*

The existing dewatered cake conveyor at the WWTF transports dewatered sludge cake from the BFPs to the product storage bay, where it is then moved to the covered storage shelter by facility staff using a front end loader. The existing conveyor is a Serpentix Model H conveyor. Original design information for the dewatered cake conveyor was collected from submittal drawing information collected from the City's records. The original design information is summarized in **Table 6.16** below.

*Table 6.16 – Dewatered Cake Conveyor Original Design Data*

| Parameter | Units | Value |
|---|---|---|
| Conveyor Manufacturer | - | Serpentix |
| Conveyor Model | - | Model H |
| Conveyor Belt Width | inches | 26 |
| Conveyor Belt Speed | ft/min | 22 |
| Centerline Length of conveyor | ft | 127 |
| Total Belt Length | ft | 260 |
| Belt Drive Motor Power | hp | 5 |
| Belt Drive Motor Speed | rpm | 1750 |
| Belt Drive Motor Voltage | V | 460 |
| Belt Drive Motor Phase | - | 3 Phase |
| Conveyor Capacity | tons/hr | 5 |
| Mfr. Assumed Material Density | $lbs/ft^3$ | 65 |
| Volumetric Capacity per Belt Pan | $in^3$ | 135 |

BFP loading rates will be approximately the same at current and 2040 conditions, based on current and expected continuing operations near the maximum recommended solids loading rate of 1,500 dry lbs/hour per BFP. The BFPs will only operate more or less frequently to maintain pace with sludge production rates. As a result, the dewatered cake production rate per hour (and the required dewatered cake conveyor capacity) will also remain the same under current and 2040 conditions. **Table 6.17** below summarizes the required dewatered cake conveyor capacities with one and two BFPs running.

*Table 6.17 – Dewatered Cake Conveyor Current Capacity Summary*

| Parameter | Units | Value |
|---|---|---|
| Recommended Maximum SLR per BFP | lbs/hr | 1,500 |
| BFP Solids Capture Rate | % | 95% |
| Dewatered Cake Solids Concentration | % solids | 16% |
| No. of BFPs in Service | - | 1 |
| Dewatered Cake Production, wet weight | tons/hr | 4.45 |
| Remaining Conveyor Capacity | tons/hr | 0.55 |
| No. of BFPs in Service | - | 2 |
| Dewatered Cake Production, wet weight | tons/hr | 8.90 |
| Remaining Conveyor Capacity | tons/hr | **-3.90** |

Based on the original design documentation summarized above, the existing dewatered cake conveyor capacity is undersized when both BFPs are in operation. However, the cake conveyor capacity is primarily a function of belt speed. It is known that the City has continually operated both BFPs at nearly the same loading rates over most of the cake conveyor's operating life since installation in 2000. This indicates that the existing cake conveyor has sufficient capacity with proper belt speed adjustment. Serpentix product data for current Model H conveyors indicate capacities up to 100 tons/hr.

Regardless, the existing conveyor has experienced significant wear along its drive chain, rollers, and other components. Per correspondence with a Serpentix representative, if the conveyor support structure and frames are in acceptable condition, the moving components (chains, carriages, roller, beltings, drive station, etc.) can be replaced to increase the conveyor's capacity in lieu of a complete system replacement. Prior to replacement, it is recommended to consult with Serpentix to determine the feasibility of retrofitting the existing conveyor, or if complete replacement will be required. It is recommended that the replacement of the dewatered cake conveyor, regardless of whether it is refurbished or completely replaced, be rated for a design capacity of at least 10 wet tons/hour.

#### 6.3.1.3 *Belt Filter Press Feed Pumps*

Belt filter press feed pumping capacity requirements are dictated by the capacity of the belt filter presses. The BFP feed pumps at the WWTF are in the thickening building and consist of three Model 4-inch double disc pumps manufactured by Penn Valley Pump. Each pump is driven by a drive belt connected to a 7.5 hp inverter duty motor and variable frequency drive. The existing piping arrangement allows for any of the existing BFP feed pumps to deliver TWAS to either of the BFPs. Design characteristics and capacity limitations of the BFP feed pumps are summarized in **Table 6.18** below. Based on the design information below, the existing belt filter press feed pumps have sufficient firm capacity for current and 2040 conditions. As mentioned in Technical Memorandum No. 1, the existing BFP feed pumps are nearing the end of their useful life and will need to be replaced. The existing pumps may be replaced with like-kind pumps of similar capacity. It should be noted that newer models of the same pumps have increased capacity within the same footprint due to improvements in the manufacturer's design. Increased BFP feed pump capacity will provide additional flexibility for increased loading rates to new BFPs with higher capacities.

*Table 6.18 – Belt Filter Press Feed Pumps Current Capacity Summary*

| Parameter | Units | Value |
|---|---|---|
| Number of BFP Feed Pumps | - | 3 |
| Pump Suction Diameter | in | 4 |
| Pump Discharge Diameter | in | 4 |
| Common Suction Header Diameter | in | 8 |
| Common Discharge Diameter | in | 6 |
| Motor Power | hp | 7.5 |
| Max. Motor Speed | rpm | 1750 |
| Max. Drive Shaft Speed | rpm | 519 |
| Max. Capacity (one pump) | gpm | 95 |
| TDH at Max. Capacity (one pump) | ft | 80 |
| Max. Working Pressure | psig | 43 |
| Stall Pressure | psig | 110 |
| Max. Sludge Dry Solids Content | % dry solids | 9% |

### 6.3.2 Alternatives Evaluation

Based on the current capacity analysis of the sludge dewatering equipment, replacement of the existing BFPs, dewatering polymer feed systems, dewatered cake conveyor, and BFP feed pumps in like-kind will meet the 2040 design conditions. It is therefore recommended to replace these pieces of equipment within the timeframe established in Technical Memorandum No.1 or earlier, as needed.

#### 6.3.2.1 *Replacement of Existing Belt Filter Press*

The opinion of probable project cost for installation for the belt filter press replacement is summarized below in **Table 6.19**. The opinion includes the cost for complete, like-kind replacement of the presses as well as the polymer feed system required for proper operation.

*Table 6.19 Belt Filter Press Replacement Cost Opinion*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $567,000 |
| 2 | Mechanical | $114,000 |
| 3 | Electrical | $114,000 |
| 4 | Instrumentation | $57,000 |
| 5 | Demo | $30,000 |
| 6 | Mobilization & Demobilization | $36,000 |
| 7 | Indirect Costs | $43,000 |
| 8 | General Conditions & Contractor Markup | $203,000 |
| 9 | 30% Contingency | $350,000 |
| 10 | Engineering, Legal, & Administration | $281,000 |
| | **Total Cost Opinion** | **$1,795,000** |

#### 6.3.2.2 *Replacement of Dewatered Cake Conveyor*

The opinion of probable project cost for installation of a Serpentix dewatered cake conveyor is summarized below in **Table 6.20**. The opinion includes the cost for complete, in-kind replacement of the Serpentix Sludge Conveyor. Depending on the condition of the existing conveyor, replacing parts & rebuilding the conveyor can cost nearly 60 to 80% of the cost of a new complete conveyor. It was assumed that the conveyor was completely replaced to be economically conservative. Typical assumptions for electrical and instrumentation installation costs have been modified for this cost opinion due to limited scope of work associated with a like-kind replacement. In addition, the project contingency has been reduced from 30% to 10%, and the engineering, legal, and administration cost opinion has been reduced due to the limited scope of this project.

*Table 6.20 Dewatered Cake Conveyor Cost Opinion*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $395,000 |
| 2 | Mechanical | $79,000 |
| 3 | Electrical | $20,000 |
| 4 | Instrumentation | $20,000 |
| 5 | Demo | $20,000 |
| 6 | Mobilization & Demobilization | $22,000 |
| 7 | Indirect Costs | $27,000 |
| 8 | General Conditions & Contractor Markup | $123,000 |
| 9 | 10% Contingency | $71,000 |
| 10 | Engineering, Legal, & Administration | $68,000 |
| | **Total Cost Opinion** | **$845,000** |

#### 6.3.2.3 *Replacement of Belt Filter Press Feed Pumps*

The cost of replacement of the BFP feed pumps is captured in the estimated cost opinion for sludge thickening Alternative 1, found in **Section 6.2.2.1**. It is assumed that the existing BFP feed pumps will be replaced in like-kind, matching the existing Penn Valley Pump Co. double disc style positive displacement pumps.

## 6.4 Sludge Drying

The *Solids Management Plan Evaluation Report* previously prepared by McKim & Creed recommended that the City construct a new thermal drying facility at the WWTF to produce a Class A-EQ biosolids product. The recommended thermal drying process includes maintaining current sludge thickening and dewatering processes prior to sludge drying to reduce the water content of the sludge and limit natural gas usage. The new thermal drying facility was recommended to be constructed under the existing sludge storage shelter with modifications to the structure made to enclose the equipment and provide a clean operating environment. Dewatered cake solids are recommended to be transferred to the new thermal drying facility via a conveyor where they will be discharged into a thermal dryer feed hopper to maintain a consistent feed rate to the dryer. A direct medium temperature gas fired belt dryer was recommended for sludge drying due to its low cost, ease of implementation, and expected dried product characteristics. Storage and haul-off of dried biosolids were proposed to be accomplished by constructing a product storage silo or hopper and truck load-out station adjacent to the thermal drying facility. The conceptual cost opinion for these improvements are summarized below in **Table 6.21**.

*Table 6.21 – Thermal Drying Facility Cost Opinion*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $3,084,000 |
| 2 | Mechanical | $617,000 |
| 3 | Electrical | $617,000 |
| 4 | Instrumentation | $309,000 |
| 5 | Structural | $640,000 |
| 6 | Civil | $31,000 |
| 7 | Demo | $240,000 |
| 8 | Mobilization & Demobilization | $222,000 |
| 9 | Indirect Costs | $261,000 |
| 10 | General Conditions & Contractor Markup | $1,266,000 |
| 11 | 30% Contingency | $2,187,000 |
| 12 | Engineering, Legal, & Administration | $1,757,000 |
| | **Total Cost Opinion** | **$11,231,000** |

## 6.5 Recommendations

Improvements to the solids handling processes at the WWTF are recommended to improve operational flexibility, rehabilitate existing equipment, replace existing equipment that has reached the end of its useful life, and produce a Class A-EQ biosolids product with a wide variety of disposal options. It is recommended that aerated thickened WAS storage be provided prior to dewatering to improve thickening operations, reduce odor generation from the existing gravity thickeners, and provide additional flexibility in the operation of the dewatering equipment. Two alternatives were evaluated to provide TWAS storage:

- Alternative 1:Construction of new aerated TWAS storage tanks following gravity thickening, and
- Alternative 2: Construction of a new gravity belt thickening facility and retrofits to the existing gravity thickeners to convert them to aerated TWAS storage tanks.

A cost comparison of the TWAS storage alternatives is provided in **Table 6.22** below.

*Table 6.22 – Cost Comparison of TWAS Storage Alternatives*

| Alternative | Capital Costs | 2021 Annual O&M Cost | 2040 Annual O&M Cost | O&M NPV | Total NPV |
|---|---|---|---|---|---|
| **Alternative 1: New TWAS Storage Tanks** | $4,901,000 | $24,000 | $41,000 | $439,000 | $5,340,000 |
| **Alternative 2: New GBT Facility and Conversion of Existing Gravity Thickeners to TWAS Storage** | $8,341,000 | $46,000 | $73,000 | $808,000 | $9,149,000 |

As seen in the cost comparison, it is significantly more cost effective to maintain the existing gravity thickeners and construct new aerated TWAS holding tanks. The conceptual cost opinion and estimated O&M costs for Alternative 1 are significantly lower than Alternative 2. Therefore, it is recommended that the City proceed with Alternative 1 to maintain the existing gravity thickeners and construct new aerated TWAS storage tanks prior to dewatering. Structural rehabilitation of the existing gravity thickeners is recommended to occur in near future, especially for gravity thickener No. 1 due to its advanced age as noted in condition assessment described in Technical Memorandum No. 1. Two 50-ft diameter aerated TWAS storage tanks are recommended to be constructed, complete with jet aeration and mixing equipment, new BFP feed pumps, and modifications to the existing BFP feed piping. The recommended TWAS storage tank sizing will provide nearly 8 days of TWAS storage at current maximum month conditions with on tank out of service, and nearly 5 days of storage at 2040 design maximum month conditions with one tank out of service. Construction of the aerated TWAS storage tanks can be phased such that a single tank is constructed in the near term, and a second tank is constructed in the future as influent loading and sludge production increases.

In the dewatering building, it is recommended that the City replace the existing BFPs, dewatered cake conveyor, and BFP polymer feed systems when operation and maintenance of the existing equipment is no longer feasible or reliable. It is expected that replacement of the existing dewatered cake conveyor will be needed before replacement of the BPFs and polymer feed systems, based on the condition assessments noted in Technical Memorandum No. 1. Therefore, two separate projects are expected to be required. However, replacement of the existing BFPs may require modifications to the dewatered cake conveyor location and dimensions. Therefore, it is recommended that the City consult with the dewatered cake conveyor's manufacturer to perform a site inspection and more accurately determine when rehabilitation or replacement of the conveyor will be necessary.

Following dewatering, implementation of a new thermal drying facility remains the recommended alternative as noted previously in the *Solids Management Plan Evaluation Report* prepared by McKim & Creed. Construction of the new thermal drying facility and ancillary equipment is recommended to occur once landfill disposal of unstabilized biosolids is infeasible due to disposal cost or reliability.

## 6.6 References


- Tchobanoglous, G., Stensel, H. D., Tsuchihashi, R., & Burton, F. L. (2014). *Wastewater Engineering: Treatment and Resource Recovery* (5th ed.). New York, NY: McGraw-Hill Education.

# 7. SUMMARY OF RECOMMENDATIONS

Evaluations were conducted for each area of the City of Hendersonville's WWTF to evaluate the current capacity of each treatment process and major pieces of equipment, and to identify, evaluate, and recommend improvements and expansions to address current and future wastewater treatment needs. The evaluations completed as part of this technical memorandum were also based on the existing condition assessments documented in Technical Memorandum No. 1 of this master plan. Recommended improvements throughout the facility based on the evaluations contained in this technical memorandum are summarized in this section. Additional replacement and rehabilitation projects identified in Technical Memorandum No. 1, recommended flow equalization improvements described in Technical Memorandum No. 3, and other projects previously identified by the City will be included in the overall master plan document and Capital Improvement Plan. The final master plan document will serve as a summary of all evaluations performed as documented in the three technical memoranda, and will provide the overall Capital Improvement Plan for the WWTF including expected project costs.

## 7.1 Preliminary Treatment

Expansion of the existing influent pumping station and upsizing of the existing influent pumps is recommended to provide firm pumping capacity to meet the current and future peak influent flow rates. Minor structural repairs and rehabilitation to the existing influent pump station structure per the condition assessments in Technical Memorandum No. 1 are recommended to prolong the service life of the existing influent pumping station. Heating and ventilation improvements are also recommended within the existing influent pumping station building and dry well per TM No. 1. Screening of influent wastewater is recommended to be relocated ahead of the expanded influent pumping station to protect the influent pumps from ragging and excessive wear. The screening facility is recommended to consist of chain-driven multi-rake bar screens with a bar spacing of ¼-inch (6 mm) per the Ten State Standards for WWTF's without primary treatment. Influent flow measurement at the influent pumping station is recommended to be replaced by new electromagnetic flow meters on the influent force mains leaving the expanded pump station, or alternatively, using multiple Parshall flumes located immediately downstream of the new screening equipment in each screening channel. A new mechanically induced vortex grit removal system is recommended to be constructed at the old plant site immediately upstream of the proposed inline flow equalization basin per the recommendations of Technical Memorandum No. 3.

## 7.2 Secondary Treatment

The results of the secondary process evaluation concluded that maintenance of the existing extended aeration process and future modifications to convert it to a Modified Ludzack-Ettinger (MLE) process is preferred over other process modifications evaluated. Other process modifications for process intensification to reduce spatial requirements were not able to eliminate future needs for additional aeration basins or clarifiers, therefore these alternatives were not economically advantageous. It is recommended that the existing extended aeration process be modified to an MLE process to continue to avoid chemical alkalinity addition through natural alkalinity recovery, and to reduce process aeration demands due to anoxic denitrification. These modifications are recommended to occur when increased influent TKN loading necessitates additional alkalinity recovery. Recommended improvements to convert the extended aeration process to an MLE process include:

- Implementation of new anoxic zone mixing consisting of a new compressed gas mixing system
- Addition of nitrified internal mixed liquor recycle pumps and piping to each aeration basin to recycle nitrified mixed liquor from the end of the aeration basin back to the head of the basin for denitrification

Intermediate improvements to the existing extended aeration process prior to the conversion to an MLE process are also recommended to include:

- Replacement of the existing constant speed multi-stage centrifugal blowers with new turbo blowers with VFDs to provide improved energy efficiency and replace equipment that has reached the end of its useful life
- Rehabilitation of the existing blower building per the recommendations of Technical Memorandum No. 1 and retrofits to the structure to provide an enclosed blower room for protection of turbo blower intakes
- Replacement of the existing RAS pump No. 2 and the existing WAS pumps in like-kind, minor structural repairs to the existing Recycle Pumping Station, and improvements to the existing Recycle Pumping Station heating and ventilation systems per the recommendations of Technical Memorandum No. 1

Future expansion of the MLE secondary treatment process is recommended to meet 2040 loading conditions and provide sufficient system redundancy. The future expansion of the secondary process is recommended to include:

- Primary effluent splitter box
- A new 2.4 MG aeration basin No. 3, to match existing aeration basins No. 1 and No. 2, including a dedicated anoxic zone with compressed gas mixing and a NRCY pump and pipeline

- A new blower building No. 2 to house new turbo blowers, a compressed gas mixing system for the anoxic zone, NRCY pump VFDs, and all associated electrical and control equipment
- A new MLSS splitter box to direct aeration basin No. 3 effluent to a new secondary clarifier No. 3, and provide long-term future expansion capability to include a fourth aeration basin and secondary clarifier
- A new 90-ft diameter secondary clarifier No. 3, to match existing secondary clarifiers No. 1 and No. 2
- A new recycle pumping station No. 2 to include RAS and WAS pumping serving aeration basin No. 3 and secondary clarifier No. 3

## 7.3 Tertiary Filtration

The existing traveling hood sand filter No. 2 is recommended to be replaced with a new AquaDiamond filter No. 2 to match AquaDiamond filter No. 1 in the near future. This is recommended to provide sufficient redundancy up to the future 15 MGD peak hour hydraulic capacity of the WWTF. A new canopy structure is recommended to be constructed over the existing filter basins per the recommendations of Technical Memorandum No. 1 to prevent excessive algae growth and protect the filter equipment from weathering. Future expansion of the tertiary filters to meet the 2040 peak hour hydraulic capacity of 19.5 MGD is recommended to include the construction of a new filter No. 3 consisting of a new cloth media surface filtration disc filter unit similar or equal to the Hydrotech Discfilter. Future filter No. 3 is recommended to include manufacturer provided covers over the filter discs to prevent the need for an additional canopy structure.

## 7.4 Disinfection and Post-Aeration

A new UV disinfection channel No. 2 is recommended to be constructed between the existing disinfection channel and the existing utility building. The existing UV disinfection equipment is recommended to be maintained to provide additional disinfection redundancy, if needed. Construction of the new UV disinfection channel is recommended to include a common influent channel to promote equal flow splitting between disinfection channels, connection to the existing NPW wet well and cascade reaeration steps, replacement of the existing fiberglass grating on the existing disinfection channel with solid covers, and construction of a new canopy structure over both disinfection channels to prevent algae growth, protect the equipment from weathering, and provide additional protection from lightning damage. Future improvements to meet 2040 design conditions are recommended to include:

- Retrofit the existing UV disinfection channel with new UV disinfection equipment to match UV disinfection channel No. 2
- Replace the existing cascade reaeration steps to ensure effluent DO permit limits are met
- Replace the existing 36-inch outfall pipeline to alleviate hydraulic bottlenecks at the FEMA 100-year flood conditions

## 7.5 Biosolids

Improvements to sludge thickening includes the rehabilitation of the existing gravity thickeners and construction of new aerated TWAS storage tanks to provide additional thickening and dewatering operational flexibility. The existing belt filter press feed pumps are recommended to be replaced and repurposed to transfer TWAS to new TWAS storage tanks prior to dewatering. New belt filter press feed pumps and jet aeration and mixing equipment are recommended to be installed along with the TWAS storage tanks. The existing dewatering belt filter presses and belt filter press polymer feed systems are recommended to be replaced in-place when maintenance and upkeep of the existing equipment is no longer feasible. The existing dewatered cake conveyor is recommended to be rehabilitated or replaced per recommendations of a manufacturer inspection to prevent failure of the existing cake conveyor. Future improvements to biosolids management are recommended to include the construction of a new biosolids thermal drying facility as previously recommended in the *Solids Management Plan Evaluation* report.

## 7.6 Schedule for Improvements

The improvements recommended above have been grouped into three primary phases based on the immediacy of their needs. These three phases are summarized below:

### 7.6.1 Phase 1 – Immediate and Near-Term Needs

The Phase 1 WWTF Improvements are recommended to consist of the following:

- Construction of a new UV disinfection channel No. 2
- Replacement of tertiary filter No. 2 to match AquaDiamond filter No. 2
- Expansion of the influent pumping station
- Construction of a new screening facility upstream of the expanded headworks
- Construction of a new grit removal facility upstream of the proposed inline EQ basin
- Construction of a new inline flow EQ basin
- Blower replacement and blower building improvements
- Dewatered cake conveyor replacement

- RAS/WAS pump replacements and recycle pumping station improvements

### 7.6.2 Phase 2 – Intermediate Term Needs

The Phase 2 WWTF Improvements are recommended to consist of the following:

- Rehabilitation of the existing gravity thickeners and construction of new TWAS storage
- Construction of a new biosolids thermal drying facility
- Conversion of the existing extended aeration process to a Modified Ludzack-Ettinger process including anoxic zone mixing and nitrified internal mixed liquor recycle pumps and piping
- Replacement of the existing dewatering belt filter presses and belt filter press polymer feed systems

### 7.6.3 Phase 3 – Long Term Future Needs

The Phase 3 WWTF Improvements are recommended to consist of the following:

- Expansion to a third MLE secondary treatment train to meet 2040 loading conditions
- Construction of tertiary filter No. 3
- Expansion of UV disinfection through retrofits to the existing UV disinfection channel to match UV disinfection channel No. 2
- Replacement of cascade reaeration steps
- Replacement of the effluent outfall

# APPENDIX A:

## PRELIMINARY SCREENING OF BIOLOGICAL PROCESS ALTERNATIVES

# PAIRWISE COMPARISON WEIGHTING WORKSHEET
## CITY OF HENDERSONVILLE
## WASTEWATER TREATMENT FACILITY MASTER PLAN
**PROJECT NO.: 06496-0009**

| | |
|---|---|
| **Subject:** | Preliminary Screening of Biological Process Alternatives |

| | |
|---|---|
| **Evaluator Name:** | |

**Part 1 of 3: Cost vs. Non-Cost Criteria**

| Criteria | Criteria Weight |
|---|---|
| Cost | 35% |
| Non-Cost | 65% |
| Total | 100% |

*Note: Total weight must equal 100%.*

**Part 2 of 3: Cost Criteria Evaluation**

| Criteria | Criteria Weight |
|---|---|
| Capital Cost | 20% |
| Lifecycle O&M Cost | 15% |
| Cost Subtotal | 35% |

*Note: Cost subtotal must match weight subtotal from Part 1 above.*

**Part 3 of 3: Non-Cost Criteria Evaluation**
*Note: See explanation of scoring and example scoring matrix below.*

| Criteria \ Criteria | Adaptability to Future Effluent Limits | Land Use | Energy Efficiency | Maintenance Requirements | Chemical Requirements | Sludge Quantity | Sludge Quality | Personnel Requirements | Operator Familiarity | Constructability / Maintenance of Plant Operations During Construction | Other Environmental Impacts (stream/wetland) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Adaptability to Future Effluent Limits | 1 | | | | | | | | | | |
| Land Use | | 1 | | | | | | | | | |
| Energy Efficiency | | | 1 | | | | | | | | |
| Maintenance Requirements | | | | 1 | | | | | | | |
| Chemical Requirements | | | | | 1 | | | | | | |
| Sludge Quantity | | | | | | 1 | | | | | |
| Sludge Quality | | | | | | | 1 | | | | |
| Personnel Requirements | | | | | | | | 1 | | | |
| Operator Familiarity | | | | | | | | | 1 | | |
| Constructability/Maintenance of Plant Operations During Construction | | | | | | | | | | 1 | |
| Other Environmental Impacts (stream/wetland) | | | | | | | | | | | 1 |

| HOW TO COMPLETE THE MATRIX ABOVE | |
|---|---|
| **Step 1.** | Compare the criteria in the vertical column to the criteria in the top row. Criteria cannot be compared to themselves, so a 1 is always placed in the cell that represents the comparison of one criteria to itself. If the criteria in the vertical column is MORE important than the criteria in the top row, place a WHOLE number in the cell where the row and column intersect. If the criteria in the vertical column is LESS important than the criteria in the top row, place a FRACTION in the cell where the row and column intersect. See the scoring values in the table below for reference. |
| **Step 2.** | Fill in all of the cells ABOVE the center diagonal line first. |
| **Step 3.** | Next, fill in all of the cells BELOW the center diagonal line. The scores in the cells below the diagonal line are the inverse of the scores above the diagonal line. |

| Definition | Value | Definition | Value |
|---|---|---|---|
| Equally Important | 1 | Equally Important | 1/1 |
| Slightly More Important | 2 | Slightly Less Important | 1/2 |
| Moderately More Important | 3 | Moderately Less Important | 1/3 |
| Much More Important | 4 | Much Less Important | 1/4 |
| Extremely More Important | 5 | Extremely Less Important | 1/5 |

***Example Matrix***

| Criteria \ Criteria | A | B | C | D | E |
|---|---|---|---|---|---|
| A | 1 | 5 | 2 | 1/3 | 1 |
| B | 1/5 | 1 | 1/5 | 2 | 4 |
| C | 1/2 | 5 | 1 | 4 | 1/3 |
| D | 3 | 1/2 | 1/4 | 1 | 2 |
| E | 1 | 1/4 | 3 | 1/2 | 1 |

**Explanation:**

In the example above, criteria A is extremely more important than criteria B, so a value of 5 is placed in the cell where the A on the left intersects with the B on the top.
Inversely, criteria B is extremely less important than criteria A, so a value of 1/5 is placed in the cell where the B on the left intersects with the A on the top.
The rest of the matrix is filled out in the same fashion, comparing the criteria on the left to the criteria on the top.

# PAIRWISE COMPARISON WEIGHTING SUMMARY
# CITY OF HENDERSONVILLE
# WASTEWATER TREATMENT FACILITY MASTER PLAN
PROJECT NO.: 06496-0009

| Subject: | Preliminary Screening of Biological Process Alternatives |
|---|---|

**Part 1 of 3: Cost vs. Non-Cost Criteria**

| Criteria | Average Criteria Weights |
|---|---|
| Cost | 35% |
| Non-Cost | 65% |
| **Total** | **100%** |

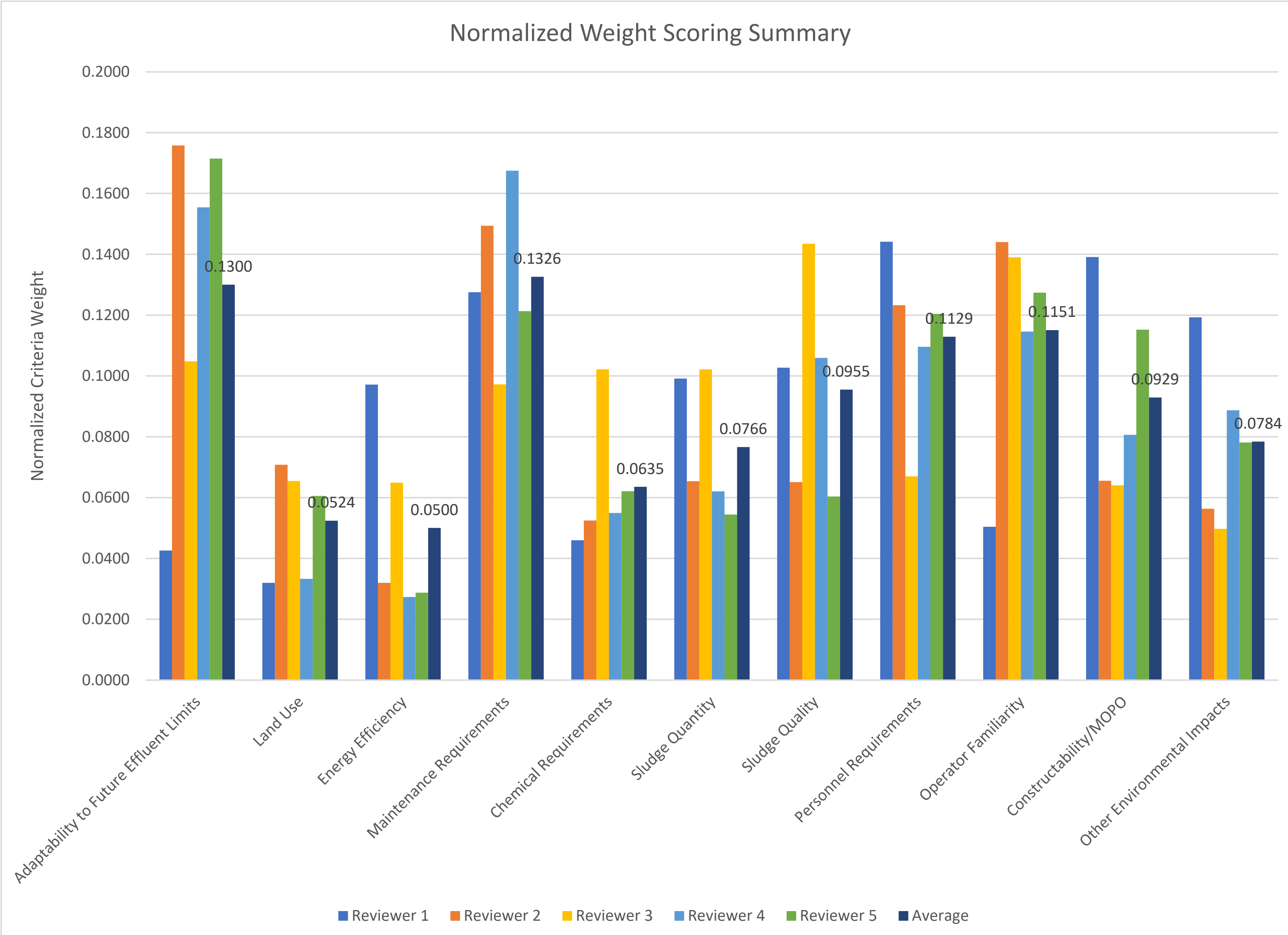
**Part 2 of 3: Cost Criteria Evaluation**

| Criteria | Average Criteria Weights |
|---|---|
| Capital Cost | 20% |
| Lifecycle O&M Cost | 15% |
| **Cost Subtotal** | **35%** |

**Part 3 of 3: Non-Cost Criteria Evaluation**

| Average Normalized Weights | | | | | | |
|---|---|---|---|---|---|---|
| Reviewer | Reviewer 1 | Reviewer 2 | Reviewer 3 | Reviewer 4 | Reviewer 5 | Average |
| Criteria | | | | | | |
| Adaptability to Future Effluent Limits | 0.0426 | 0.1758 | 0.1048 | 0.1554 | 0.1715 | 0.1300 |
| Land Use | 0.0320 | 0.0708 | 0.0655 | 0.0333 | 0.0605 | 0.0524 |
| Energy Efficiency | 0.0971 | 0.0320 | 0.0649 | 0.0273 | 0.0287 | 0.0500 |
| Maintenance Requirements | 0.1275 | 0.1494 | 0.0972 | 0.1675 | 0.1213 | 0.1326 |
| Chemical Requirements | 0.0460 | 0.0525 | 0.1022 | 0.0550 | 0.0621 | 0.0635 |
| Sludge Quantity | 0.0991 | 0.0654 | 0.1022 | 0.0620 | 0.0544 | 0.0766 |
| Sludge Quality | 0.1027 | 0.0651 | 0.1434 | 0.1059 | 0.0604 | 0.0955 |
| Personnel Requirements | 0.1441 | 0.1233 | 0.0670 | 0.1096 | 0.1204 | 0.1129 |
| Operator Familiarity | 0.0504 | 0.1440 | 0.1390 | 0.1146 | 0.1274 | 0.1151 |
| Constructability/MOPO | 0.1391 | 0.0655 | 0.0640 | 0.0806 | 0.1152 | 0.0929 |
| Other Environmental Impacts | 0.1193 | 0.0563 | 0.0497 | 0.0887 | 0.0781 | 0.0784 |
| *sum* | *1.00* | *1.00* | *1.00* | *1.00* | *1.00* | *1.00* |

| Criteria | Average Percent Weight |
|---|---|
| **Maintenance Requirements** | 8.6% |
| **Adaptability to Future Effluent Limits** | 8.5% |
| **Operator Familiarity** | 7.5% |
| **Personnel Requirements** | 7.3% |
| **Sludge Quality** | 6.2% |
| **Constructability/Maintenance of Plant Operations During Construction** | 6.0% |
| **Other Environmental Impacts (stream/wetland)** | 5.1% |
| **Sludge Quantity** | 5.0% |
| **Chemical Requirements** | 4.1% |
| **Land Use** | 3.4% |
| **Energy Efficiency** | 3.3% |



Normalized Weight Scoring Summary
Normalized Criteria Weight
0.2000
0.1800
0.1600
0.1400
0.1200
0.1000
0.0800
0.0600
0.0400
0.0200
0.0000
0.1300
0.0524
0.0500
0.1326
0.0635
0.0766
0.0955
0.1129
0.1151
0.0929
0.0784
Adaptability to Future Effluent Limits
Land Use
Energy Efficiency
Maintenance Requirements
Chemical Requirements
Sludge Quantity
Sludge Quality
Personnel Requirements
Operator Familiarity
Constructability/MOPO
Other Environmental Impacts
Reviewer 1
Reviewer 2
Reviewer 3
Reviewer 4
Reviewer 5
Average

# BIOLOGICAL PROCESS ALTERNATIVES PRELIMINARY SCREENING MATRIX
## CITY OF HENDERSONVILLE
## WASTEWATER TREATMENT FACILITY MASTER PLAN
PROJECT NO.: 06496-0009

| | | Alternative 1 | | Alternative 2 | | Alternative 3 | | Alternative 4 | | Alternative 5 | | Alternative 1(a) | | Alternative 2(a) | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Baseline Process - Expand Existing | | BioMag Ballasted Activated Sludge | | Membrane Bioreactors (MBR) | | IFAS/MBBR | | New WWTF at New Site | | Expand Baseline Process with Modified Ludzack Ettinger (MLE) | | BioMag with MLE | |
| **Criteria** | Weight | Score | Weighted Score | Score | Weighted Score | Score | Weighted Score | Score | Weighted Score | Score | Weighted Score | Score | Weighted Score | Score | Weighted Score |
| **Cost Criteria** | | | | | | | | | | | | | | | |
| **Capital Cost** | 20% | 8 | 1.6 | 6 | 1.2 | 2 | 0.4 | 4 | 0.8 | 1 | 0.2 | 7 | 1.4 | 5 | 1 |
| **Life Cycle O&M Cost** | 15% | 7 | 1.05 | 4 | 0.6 | 3 | 0.45 | 6 | 0.9 | 2 | 0.3 | 8 | 1.2 | 5 | 0.75 |
| **Cost Subtotal** | 35% | 15 | 2.65 | 10 | 1.8 | 5 | 0.85 | 10 | 1.7 | 3 | 0.5 | 15 | 2.6 | 10 | 1.75 |
| **Non-cost Criteria** | | | | | | | | | | | | | | | |
| *Adaptability to future effluent limit restrictions* | 8.5% | 5 | 0.425 | 3 | 0.255 | 7 | 0.595 | 5 | 0.425 | 10 | 0.85 | 8 | 0.68 | 8 | 0.68 |
| *Land Use* | 3.4% | 3 | 0.102 | 8 | 0.272 | 10 | 0.34 | 5 | 0.17 | 10 | 0.34 | 1 | 0.034 | 7 | 0.238 |
| *Energy Efficiency* | 3.3% | 8 | 0.264 | 5 | 0.165 | 1 | 0.033 | 6 | 0.198 | 5 | 0.165 | 9 | 0.297 | 5 | 0.165 |
| *Maintenance Requirements* | 8.6% | 8 | 0.688 | 4 | 0.344 | 1 | 0.086 | 6 | 0.516 | 3 | 0.258 | 7 | 0.602 | 4 | 0.344 |
| *Chemical Requirements* | 4.1% | 2 | 0.082 | 2 | 0.082 | 1 | 0.041 | 6 | 0.246 | 10 | 0.41 | 10 | 0.41 | 5 | 0.205 |
| *Sludge Quantity* | 5.0% | 5 | 0.25 | 4 | 0.2 | 10 | 0.5 | 8 | 0.4 | 5 | 0.25 | 6 | 0.3 | 5 | 0.25 |
| *Sludge Quality* | 6.2% | 4 | 0.248 | 6 | 0.372 | 3 | 0.186 | 5 | 0.31 | 5 | 0.31 | 6 | 0.372 | 7 | 0.434 |
| *Personnel Requirements* | 7.3% | 10 | 0.73 | 7 | 0.511 | 3 | 0.219 | 7 | 0.511 | 1 | 0.073 | 10 | 0.73 | 7 | 0.511 |
| *Operator Familiarity* | 7.5% | 10 | 0.75 | 7 | 0.525 | 1 | 0.075 | 6 | 0.45 | 9 | 0.675 | 8 | 0.6 | 5 | 0.375 |
| *Constructability/Maintenance of Plant Operations during Construction* | 6.0% | 7 | 0.42 | 8 | 0.48 | 1 | 0.06 | 3 | 0.18 | 10 | 0.6 | 7 | 0.42 | 5 | 0.3 |
| *Other Environmental Impacts (i.e. stream/wetland impacts)* | 5.1% | 2 | 0.10 | 7 | 0.36 | 10 | 0.51 | 4 | 0.20 | 5 | 0.26 | 1 | 0.05 | 5 | 0.26 |
| **Non-Cost Subtotal** | 65% | 64 | 4.06 | 61 | 3.56 | 48 | 2.65 | 61 | 3.61 | 73 | 4.19 | 73 | 4.50 | 63 | 3.76 |
| **Total Scores** | **100%** | **79** | **6.71** | **71** | **5.36** | **53** | **3.50** | **71** | **5.31** | **76** | **4.69** | **88** | **7.10** | **73** | **5.51** |

| PRELIMINARY SCREENING FINAL SCORING SUMMARY | | |
|---|---|---|
| Alternative # | Alternative | Total Weighted Score |
| 1(a) | Baseline + MLE | 7.10 |
| 1 | Baseline | 6.71 |
| 2(a) | BioMag + MLE | 5.51 |
| 2 | BioMag | 5.36 |
| 4 | IFAS | 5.31 |
| 5 | New WWTF at New Site | 4.69 |
| 3 | MBR | 3.50 |

| SUGGESTED ALTERNATIVES FOR DETAILED ANALYSIS | | |
|---|---|---|
| Alternative # | Alternative | Total Weighted Score |
| 1(a) | Baseline + MLE | 7.10 |
| 2(a) | BioMag + MLE | 5.51 |
| 4 | IFAS | 5.31 |

# APPENDIX D:

## TECHNICAL MEMORANDUM No. 3 – FLOW EQUALIZATION PRELIMINARY ENGINEERING EVALUATION

# CITY OF HENDERSONVILLE
## WASTEWATER TREATMENT FACILITY MASTER PLAN

## Technical Memorandum No.3 –
## Flow Equalization Preliminary Engineering Evaluation

Date: June 2022

**Prepared for:**


City of Hendersonville
305 Williams Street
Hendersonville, NC 28792


**Prepared by:**


McKim & Creed, Inc.
8020 Tower Point Dr.
Charlotte, NC 28227
Firm License No. F-1222


**McKim & Creed Project 06496-0009**

# CONTENTS

# TABLES

# FIGURES



# APPENDICES

# LIST OF ACRONYMS

| Abbreviation | Definition |
|---|---|
| AADF | Annual Average Daily Flow |
| ANSI | American National Standards Institute |
| ATC | Authorization to Construct |
| CIP | Cast-In-Place |
| CLOMR | Conditional Letter of Map Revision |
| CY | Cubic Yards |
| DO | Dissolved Oxygen |
| EPS | Extended Period Simulation |
| EQ | Equalization |
| FRP | Fiberglass Reinforced Plastic |
| GFS | Glass-Fused-to-Steel |
| HMI | Human Machine Interface |
| hp | Horsepower |
| HWL | High Water Level |
| L | Liters |
| LOMR | Letter of Map Revision |
| MG | Million Gallons |
| mg | milligram |
| MGD | Million Gallons per Day |
| NCAC | North Carolina Administrative Code |
| NCDEQ | North Carolina Department of Environmental Quality |
| NCDOT | North Carolina Department of Transportation |
| NOAA | National Oceanic and Atmospheric Administration |
| NPDES | National Pollutant Discharge Elimination System |
| NPV | Net Present Value |
| NRCS | Natural Resources Conservation Service |
| PF | Peaking Factor |
| PLC | Programmable Logic Controller |
| RAS | Return Activated Sludge |
| RDII | Rainfall Derived Infiltration and Inflow |
| SCADA | Supervisory Control and Data Acquisition |
| SCFM | Standard Cubic Feet per Minute |
| SOR | Surface Overflow Rate |
| SR | Secondary Road |
| SS | Stainless Steel |
| SSAIA | Sanitary Sewer Asset Inventory and Assessment |
| SSO | Sanitary Sewer Overflow |
| SWD | Side Water Depth |
| UV | Ultraviolet |
| VFD | Variable Frequency Drive |
| WAS | Waste Activated Sludge |
| WW | Wastewater |
| WWTF | Wastewater Treatment Facility |
| WWTP | Wastewater Treatment Plant |

# 1. PURPOSE AND BACKGROUND

The City of Hendersonville's Wastewater Treatment Facility (WWTF) serves the City of Hendersonville, portions of Laurel Park, the Village of Flat Rock, and a portion of the central region of Henderson County. The existing WWTF has a permitted capacity of 4.8 MGD and was designed for a peak hydraulic capacity of 12 MGD. The City recently completed an asset inventory and assessment master plan for the City's wastewater collection system. The Sanitary Sewer Asset Inventory Assessment (SSAIA) Master Plan report (completed in March of 2019) included evaluation of the collection system's hydraulic capacity based on multiple design storm events and provided recommendations for improvements to address capacity limitations. A critical finding of the SSAIA report identified hydraulic capacity limitations at the WWTF under predicted current and future 2-year design storm events. The SSAIA report recommended the construction of flow equalization facilities at the WWTF to avoid potential sanitary sewer overflows (SSOs) resulting from the limited hydraulic capacity of the WWTF.

Per the SSAIA Master Plan report, the City of Hendersonville's wastewater collection system currently experiences peak wet weather flows that are greater than the hydraulic capacity of the WWTF, which may result in SSOs. Wet weather flows to the WWTF consist of the normal dry weather sanitary sewer flow from the service area plus rainfall-derived inflow and infiltration (RDII) to the collection system. RDII is a result of flooding or pooling near manholes, unidentified or illegal cross-connections to roof drains or storm sewer systems, ground water infiltration through cracks or breaks in sanitary sewer pipes or manholes below grade, and other mechanical faults in aging sewer systems.

Wastewater flow equalization facilities are commonly employed at wastewater treatment facilities to protect against high volumes of influent flow over a short period of time, also referred to as surges. Wastewater flow equalization facilities include a flow equalization basin (or basins) and methods to control the rate of flow to and from the flow equalization basin(s). Flow equalization facilities are used to store large volumes of influent flow during surge events until the surge in influent flow has passed. Once the influent flow rate to the WWTF declines, the wastewater stored in the equalization basin(s) is then discharged back to the WWTF at a controlled rate. The use of wastewater flow equalization facilities may be further broken down into two purposes; (1) equalization of the normal diurnal pattern of influent flow to the WWTF to ensure uniform operation of treatment processes, or (2) equalization of peak influent flows from wet weather events to prevent washout of solids from the treatment process and/or prevent SSOs in the collection system.

The purpose of this technical memorandum is to evaluate alternatives to provide flow equalization facilities for the Hendersonville WWTF to address the hydraulic limitations identified by the SSAIA Master Plan report. The flow equalization facilities evaluated herein are intended to store wet weather flows that exceed the current and future expected peak hour hydraulic capacity of the WWTF to prevent or reduce the likelihood of sanitary sewer overflows (SSOs) in the collection system. The following sections detail the analyses performed and the alternatives evaluated for the selection of appropriate flow equalization facilities. This technical memorandum includes the evaluation of and recommendations for the following design considerations:

- Required EQ basin volume/sizing
- Type of EQ basin construction (i.e., earthen basin, CIP concrete, prestressed concrete, bolted steel)
- Location of flow equalization facilities
- Type of flow equalization to be implemented (in-line vs. off-line)
- Pumping/control methods
- Appurtenances for aeration, mixing, odor control, cover, flushing, etc.
- Capital, operation, and maintenance costs

# 2. FLOW PROJECTIONS

Wet weather flow projections were previously developed for the base year (2017), 2025, and 2040 conditions as part of the SSAIA Master Plan report. The previous wet weather flow projections were used as the basis for the hydraulic capacity assessment of the City's wastewater collection system to determine when capacity improvements were necessary and what design storm conditions the capacity improvements should be sized to accommodate. The wet weather flow projections were developed based on historical and projected dry weather flows and RDII from the 2-year, 5-year, and 10-year design storms. Projected dry weather flows were based on expected population growth, employment growth, redevelopment, industrial development, septic conversions, and private WWTP connections, among other factors. Rainfall intensities for the design storm events were based off NOAA's National Weather Service Hydrometeorological Design Studies Center Precipitation Frequency Data Server for the City. The design storm events were distributed to the collection system areas using the NRCS Type II distribution. The SSAIA Master Plan used the 2-year design storm as the basis for the collection system design capacity, and the future capacity improvements identified were recommended to be designed for the 10-year design storm.

Impacts of design storm events on the wastewater collection system were previously modeled as part of the SSAIA Master Plan using Innovyze's InfoSewer modeling software. Each design storm event was modeled using an extended period dynamic simulation scenario to determine the peak flow rates and flow depths and to identify surcharge conditions throughout the collection system. McKim & Creed coordinated with the City to collect simulated hydrographs for the influent flow into the WWTF Influent Pumping Station for each design storm scenario to determine the peak flows from each scenario. The peak instantaneous flows from base year (2017), 2025, and 2040 wet weather flow projections for the 2-year and 10-year design storms are summarized in **Table 2.1** below.

*Table 2.1 – Summary of Peak Wet Weather Flows*

| Year | 2-Year Storm Peak Flow (MGD) | 10-Year Storm Peak Flow (MGD) |
|---|---|---|
| Base (2017)* | 17.4 | 22.8 |
| 2025 | 22.5 | 29.4 |
| 2040 | 28.3 | 36.5 |

**Base year established by SSAIA Master Plan report*

The previously developed wet weather flow projections for future 2025 and 2040 conditions included several assumptions that were reviewed to determine the validity of the projections. First, the previous projections assumed that RDII rates that were observed in the base year (2017) would not increase throughout the analysis period. This assumption was supported by the facts that the City has an ongoing condition assessment program, a rehabilitation and replacement program, a system inspection program, and because future capacity improvements should experience reduced I/I rates with newer materials and better construction practices. The previous projections also assumed that the recommended capacity improvements identified in the SSAIA Master Plan would be implemented on schedule.

The 2025 wet weather flow projections assumed that the Mud Creek sanitary sewer outfall would be replaced where identified (per SSAIA project ID G-06) and that the downstream flow restriction at the WWTF would be eliminated by constructing flow equalization facilities (SSAIA project ID T-01). The 2040 wet weather flow projections assumed that all remaining capacity improvement projects identified would be completed on-schedule. Based on conversations with the City and knowledge of currently on-going capacity improvement projects, these assumptions appear to remain valid, as the City is currently meeting or exceeding the project schedule recommended in the SSAIA report.

# 3. EQUALIZATION BASIN SIZING

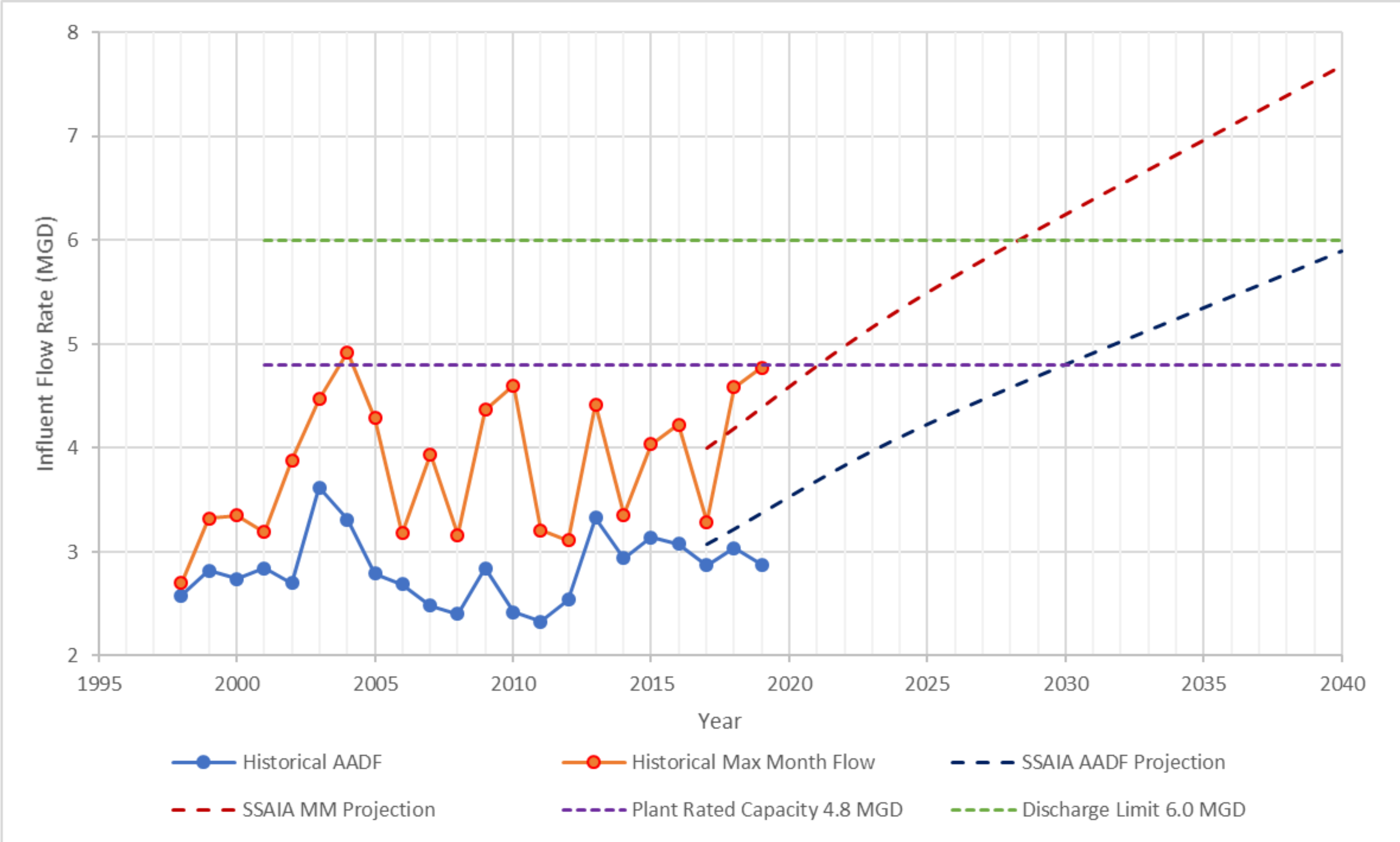
## 3.1 Evaluation of Sizing Scenarios

To compute the required EQ basin volume, a flow balance was performed based on the simulated hydrographs extracted from the City's Innovyze InfoSewer collection system model. The required inputs to perform the flow balance are (1) the flow entering the system as a function of time, and (2) the equalized flow rate to the WWTF as a function of time. Hydrographs were collected from the City for total flows tributary to the existing WWTF under 2-year and 10-year storm events at the 2025 and 2040 planning levels. The hydrographs exported from the collection system model each provided projected flow rates at the WWTF in 5-minute intervals over a 4-day extended period simulation (EPS).

As noted above, the goal of new flow equalization facilities at the WWTF is to provide storage for wet weather flows that exceed the facility's hydraulic capacity. Based on this, the equalized flow rate to the WWTF for the flow balance analyses is the expected hydraulic capacity of the WWTF under each condition. The existing WWTF was designed for a peak hour hydraulic capacity of 12 MGD based on a peaking factor of 2.5 applied to the facility's permitted capacity of 4.8 MGD. It was assumed that the future hydraulic capacity of the WWTF would continue to be based on a peaking factor of 2.5. The current permit for the WWTF also includes provisions for a future permitted capacity of 6.0 MGD upon issuance of an Authorization to Construct for facility expansion. The 6.0 MGD permitted capacity corresponds to a hydraulic capacity of 15 MGD, based on the 2.5 peaking factor.

The future hydraulic capacities of the WWTF for each condition were assigned based on the expected timing of expansions to the WWTF and the permitted capacity associated with each expansion. The expected timing of expansions to the existing WWTF is based on the 80/90% rule per 15A NCAC 02T .0118, and the future WWTF influent flow projections. The 80/90% rule states, in short, that permits for expansion must be acquired, and plans and specifications for expansion must be submitted before the annual average daily flow (AADF) to the WWTF reaches 90% of the permitted capacity. The future influent flow projections were previously presented and discussed in Technical Memorandum No. 1 of this Master Plan. The future influent flow projections are also shown in **Figure 3.1** below for reference.

*Figure 3.1 – Historical and Projected WWTF Influent Flow Rates*




Using the 80/90% rule and the future influent flow projections, the expected timing of expansions and the associated permitted and peak hour hydraulic capacities of the WWTF are listed in **Table 3.1** below. The first expansion to a permitted capacity of 6.0 MGD is based on current permit provisions for this future discharge limit. The next expansion to a permitted capacity of 7.8 MGD is based on the 2040 maximum month flow projection of 7.68 MGD to ensure maximum month flows do not result in permit violations. The actual required timing of expansions to the existing WWTF will be based on actual flows and loading to the facility. Actual future flows and loading to the WWTF may necessitate completion of expansions sooner or later than shown below.

*Table 3.1 – Summary of Expected Timing of Future WWTF Expansions*

| Permitted WWTF Capacity (MGD) | WWTF Hydraulic Capacity (MGD) (PF = 2.5) | AADF at 90% of Permitted Capacity | Year Expansion Expected to be Completed |
|---|---|---|---|
| 4.8 (current capacity) | 12.0 | 4.32 | 2025 |
| 6.0 | 15.0 | 5.40 | 2035 |
| 7.8 | 19.5 | 7.02 | 2050* |

**Based on linear extrapolation of future influent flow projections beyond 2040*

Based on the table above, flow equalization basin sizing for future conditions is based on a WWTF hydraulic capacity of 15.0 MGD in 2025, and 19.5 MGD in 2040. In addition, two other conditions are considered for flow equalization basin sizing to provide additional data points for reference. For 2025, flow equalization basin sizing is also evaluated based on the current WWTF hydraulic capacity of 12.0 MGD to provide a comparison of the EQ basin volume required if the hydraulic capacity remains unchanged at the 6.0 MGD permitted capacity. For 2040, flow equalization basin sizing is also evaluated based on a peaking factor of 3.0 times the future hydraulic capacity, which corresponds to a hydraulic capacity of 23.4 MGD.

**Figure 3.2**, **Figure 3.3**, **Figure 3.4**, and **Figure 3.5** show the hydrographs for 2-year and 10-year storm events in 2025 and 2040. In each of the hydrographs, the influent flow is compared against two potential hydraulic capacities for the WWTF. The required flow equalization volume for each scenario was found by determining the area between the influent flow curve and the hydraulic capacity line during the time intervals when influent flow exceeded the hydraulic capacity. This was computed by determining the volume of influent flow exceeding the hydraulic capacity at each 5 minute interval and summing each incremental volume over the period when influent flow exceeded the hydraulic capacity.

*Figure 3.2 – 2025 2-Year Storm WWTF Influent Flow Hydrograph*

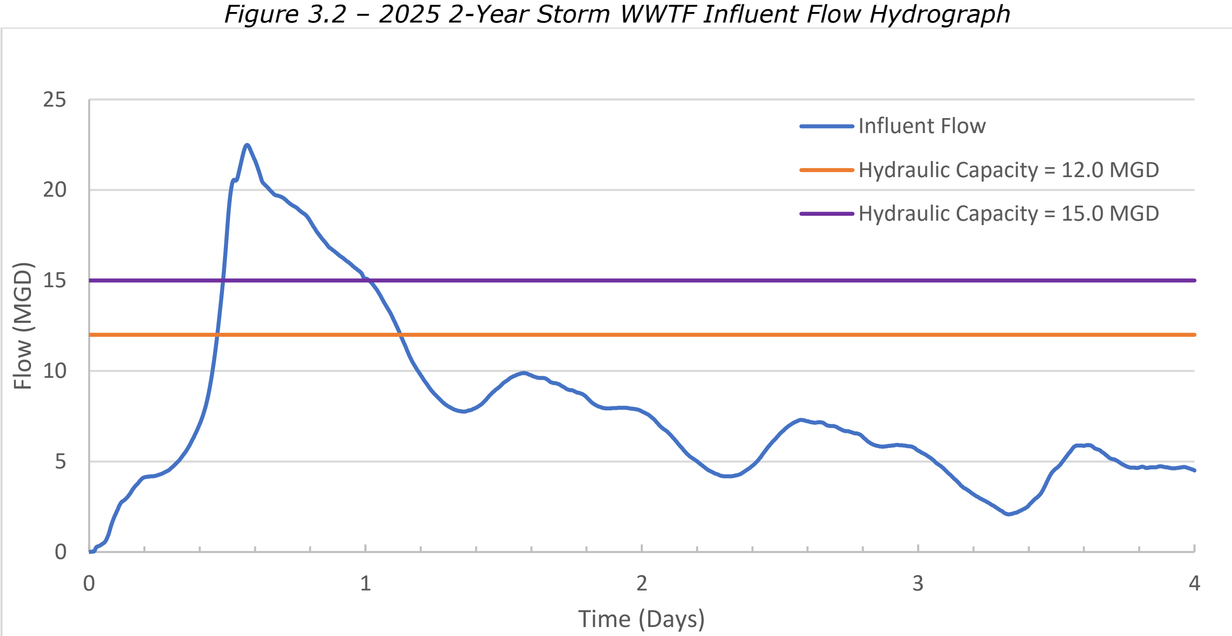


*Figure 3.3 – 2025 10-Year Storm WWTF Influent Flow Hydrograph*

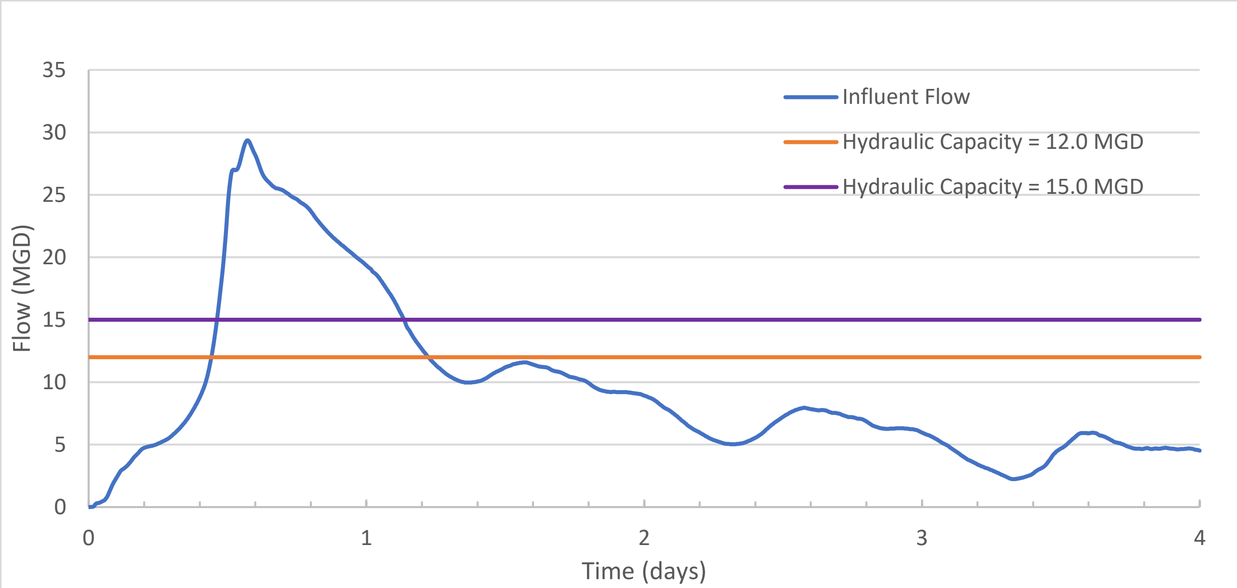



*Figure 3.4 – 2040 2-Year Storm WWTF Influent Flow Hydrograph*

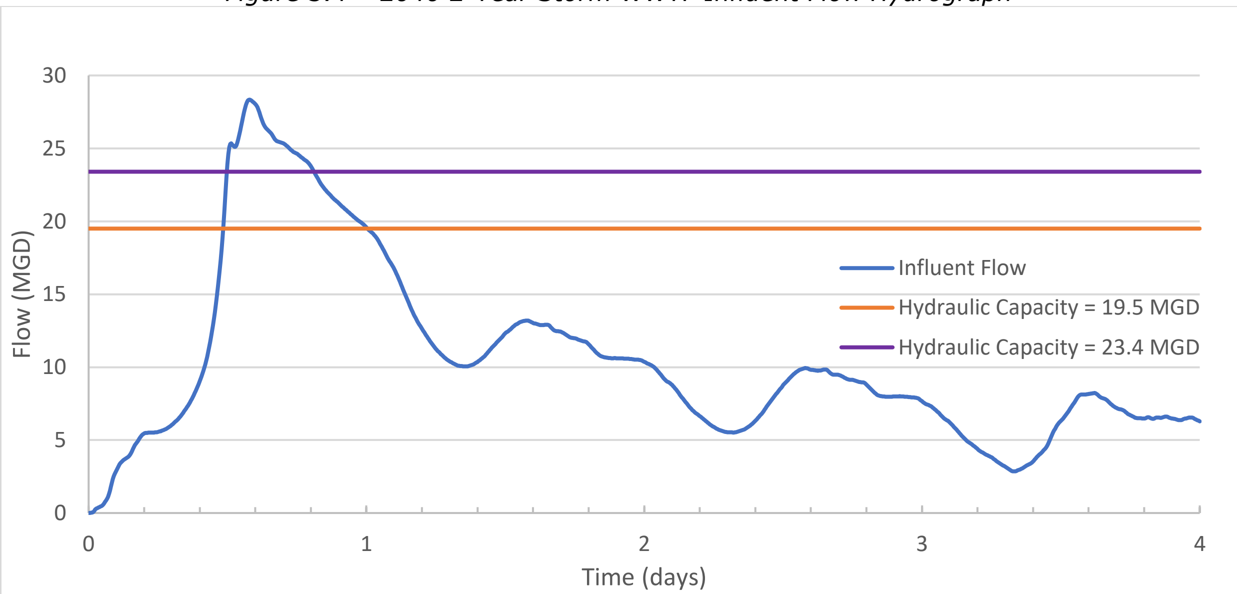
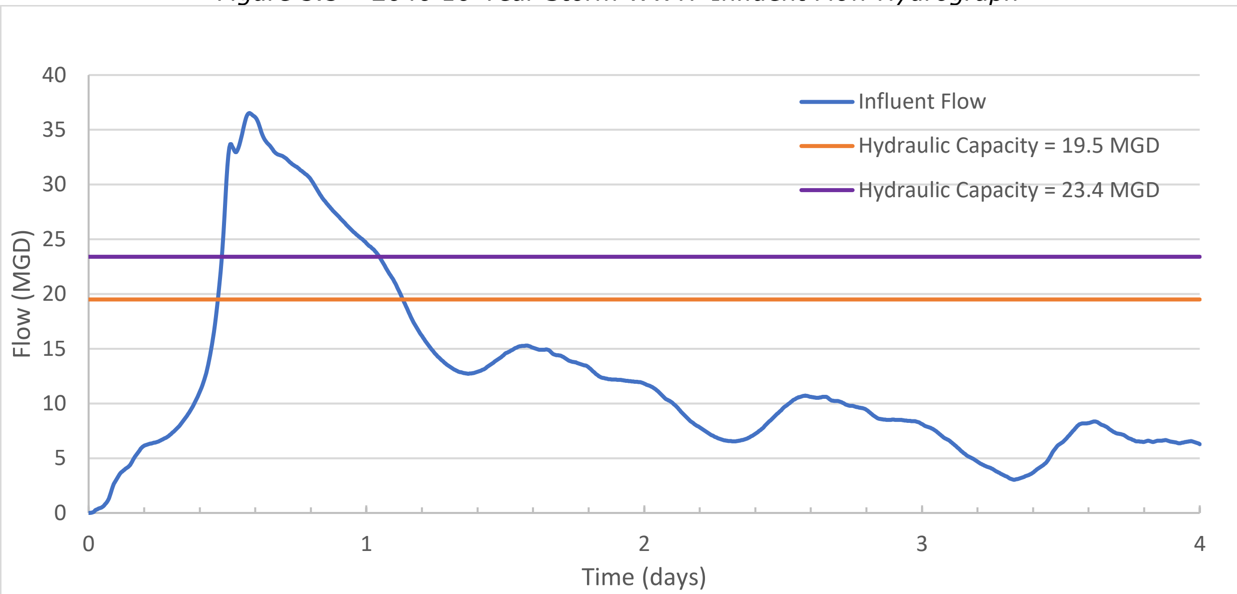


Figure 3.5 – 2040 10-Year Storm WWTF Influent Flow Hydrograph




A summary of the equalization basin sizing for the 2-year storm events is shown in **Table 3.2**. A summary of the equalization basin sizing for the 10-year storm events is shown in **Table 3.3**.

*Table 3.2 – Equalization Basin Sizing Summary for 2-Year Storm Event*

| Year | 2-Year Storm Peak Flow Rate (MGD) | Permitted Treatment Facility Flow Rate (MGD) | Plant Hydraulic Capacity (MGD) | EQ Volume Required (MG) |
|---|---|---|---|---|
| 2025 | 22.5 | 6.0 | 12.0 | 3.68 |
| 2025 | 22.5 | 6.0 | 15.0 | 1.87 |
| 2040 | 28.3 | 7.8 | 19.5 | 2.32 |
| 2040 | 28.3 | 7.8 | 23.4 | 0.74 |

*Table 3.3 – Equalization Basin Sizing Summary for 10-Year Storm Event*

| Year | 10-Year Storm Peak Flow Rate (MGD) | Permitted Treatment Facility Flow Rate (MGD) | Plant Hydraulic Capacity (MGD) | EQ Volume Required (MG) |
|---|---|---|---|---|
| 2025 | 29.4 | 6.0 | 12.0 | 7.27 |
| 2025 | 29.4 | 6.0 | 15.0 | 5.10 |
| 2040 | 36.5 | 7.8 | 19.5 | 6.27 |
| 2040 | 36.5 | 7.8 | 23.4 | 3.86 |

Per the SSAIA Master Plan report, future improvements to the City's wastewater collection system were triggered based on capacity limitations associated with peak flows from a 2-year storm event at each planning horizon. In addition, all future improvements to the collection system identified by the SSAIA Master Plan are intended to be sized to accommodate the peak flow from a 10-year storm event to provide protection against SSOs.

The 10-year storm events were evaluated in this EQ basin sizing exercise to provide a reference for the EQ storage volume required if the entire collection system could accommodate peak flows from a 10-year storm event to the WWTF. As seen in the sizing summary tables above, the 10-year storm events required EQ basin volumes approximately two to five times larger than a corresponding 2-year storm event. The EQ basin volumes required to accommodate a 10-year storm event are impractical based on the land area available at the City's WWTF. In addition, it is not practical to size future flow EQ facilities based on a 10-year storm event because the collection system is not designed to accommodate the peak flows from a 10-year storm event.

The largest EQ basin volume required for the 2-year storm events occurred in the year 2025 assuming a WWTF hydraulic capacity of 12 MGD. This condition is shown to indicate the required EQ basin volume if expansion of the WWTF's hydraulic capacity is not completed prior to this planning horizon. Expansion of the WWTF's hydraulic capacity to 15 MGD would require expansion of the influent pumping station, screening and grit removal equipment, recycle pumping station, replacement of tertiary filter no. 2 to match the AquaDiamond in tertiary filter no. 1, and replacement of the UV disinfection equipment. The EQ basin volume required for the 12 MGD hydraulic capacity scenario is 58.6% larger than the next highest EQ basin volume required. The next highest EQ basin volume required for the 2-year storm events is 2.32 MG which will be required in 2040 with a WWTF hydraulic capacity of 19.5 MGD.

The proposed EQ basin is not recommended to be sized for the 12 MGD hydraulic capacity in 2025 because it would oversize the EQ basin for all future scenarios once the WWTF's hydraulic capacity is expanded. The cost to expand or replace the existing influent pumping station, screening equipment, and grit removal equipment does not impact this recommendation because these process areas will require expansion regardless of the EQ basin sizing. The influent pumping station, screening, and grit removal are all recommended to be placed upstream of the EQ basin, therefore they will all need to be sized to handle the non-equalized peak hour flow rates regardless. In addition, the cost to expand the existing UV disinfection equipment is not considered in the comparison of the EQ basin sizing alternatives because this equipment has reached the end of its useful life and is expected to be replaced with new UV disinfection equipment, regardless of which EQ basin sizing scenario is chosen. The same logic is applied to the replacement of tertiary filter no. 2, since this replacement is also considered imminent and is not impacted by the selection of EQ basin sizing.

The 2025 (15 MGD) hydraulic capacity and the 2040 (19.5 MGD) hydraulic capacity 2-year storm scenarios that were evaluated require very similar EQ basin volumes of 1.87 MG and 2.32 MG, respectively. The 2040 – 23.4 MGD hydraulic capacity scenario evaluated results in the smallest EQ basin volume required since the allowable peaking factor for WWTF hydraulic capacity was increased from 2.5 to 3.0 times the permitted capacity.

The selection of the WWTF hydraulic capacity significantly impacts the EQ basin volume required. Increasing the allowable peaking factor for the WWTF hydraulic capacity shifts capacity requirements and capital expenditures towards additional expansion of the treatment processes instead of construction of flow EQ facilities. Inversely, increasing the size of the flow EQ basin increases the capital cost of EQ facilities and reduces the required sizing of downstream equipment and costs associated with treatment process expansion. Based on the previous analyses of WWTF hydraulics described in Technical Memorandum #2 of this Master Plan, the selection of a larger hydraulic capacity peaking factor will require widespread modifications to the existing treatment processes and piping between processes. For instance, increasing the peak hydraulic capacity of the existing WWTF beyond 15 MGD would require the following improvements:

- Expansion of the existing influent pump station to increase firm pumping capacity to 22.5 MGD for 2025 design conditions, and 28.3 MGD for 2040 design conditions
    - Required regardless of flow EQ implementation
- Expansion of the existing screening equipment with the construction of additional screening channels, or replacement of the existing screening facility upstream of the influent pump station, to increase firm screening capacity to 22.5 MGD for 2025 design conditions, and 28.3 MGD for 2040 design conditions
    - Required regardless of flow EQ implementation
- Expansion of the existing grit removal equipment with the construction of additional grit chambers, or replacement of the existing grit removal equipment with new vortex-style grit removal equipment, to increase grit removal capacity to 22.5 MGD for 2025, and 28.3 MGD for 2040 with all units in service
    - Required regardless of flow EQ implementation
- Construction of an additional (third) secondary clarifier to maintain a maximum surface overflow rate (SOR) of 1,200 gpd/ft$^2$ at peak hourly flows with all clarifiers in service to prevent solids carryover
- Construction of a new mixed liquor distribution box and associated yard piping to accommodate a third secondary clarifier
- Construction of a new RAS/WAS pump station or expansion of the existing Recycle Pumping Station to accommodate a third secondary clarifier

- Construction of a third tertiary filter and associated yard piping to provide firm capacity for peak hour flows exceeding 15 MGD, and to alleviate the hydraulic bottleneck in the existing 36-inch diameter filtered effluent piping
- Expansion of UV disinfection to provide at least two channels with new disinfection equipment to provide firm capacity for peak hour flows exceeding 15 MGD
    - Includes construction of a new UV disinfection channel as currently planned, and retrofit of the existing disinfection channel with matching UV disinfection equipment, or construction of two new disinfection channels
- Expansion or replacement of the existing cascade reaeration steps to eliminate the hydraulic bottleneck at the effluent weir to cascade reaeration
- Expansion or replacement of the existing 36-inch diameter outfall to alleviate the hydraulic bottleneck at peak hour flows exceeding 15 MGD at 100-year flood conditions

As seen above, widespread modifications to the WWTF would be required to increase the allowable peak hydraulic capacity of the WWTF in lieu of providing flow EQ or to reduce the size of the EQ basin. It is expected that the cost of these widespread modifications to the WWTF will be greater than the cost to provide flow EQ based on a WWTF hydraulic capacity peaking factor of 2.5.

## 3.2 Recommended Sizing

The EQ basin sizing is recommended to be based off the volumes determined from the 2025 (15 MGD) hydraulic capacity and 2040 (19.5 MGD) hydraulic capacity 2-year storm scenarios. The EQ basin volumes required by these two scenarios were very similar and could be feasibly constructed on the existing WWTF site. The similar EQ volume requirements for these two scenarios (1.87 MG versus 2.32 MG) also allows the initial EQ basin volume to be sized to accommodate both scenarios without oversizing the EQ basin for the immediate near-term needs of the WWTF. Therefore, an EQ basin working volume of 2.32 MG is recommended based on the volume requirement determined from the 2040 (19.5 MGD) hydraulic capacity 2-year storm scenario.

A safety factor of approximately 10% was applied to the EQ basin working volume determined from the hydrographs to provide adequate safety factor per typical design practice. The resulting basis of design EQ basin working volume is 2.5 MG.

With the EQ basin working volume established, the actual basin volume may be determined by accounting for the minimum operating water level to protect mixing and aeration equipment, and by accounting for freeboard requirements. The NCDEQ Minimum Design Criteria for NPDES Wastewater Treatment Facilities requires a minimum freeboard of 12 inches for all concrete and steel treatment units, plus additional freeboard as needed to prevent splashing outside of the unit. It is assumed that the minimum freeboard of 12 inches will provide adequate protection against splashing outside of the unit. However, the actual freeboard required will be based on manufacturer's recommendations for the mixing and aeration systems selected for design and construction. The minimum operating water level in the EQ basin was assumed to be 3 feet, based on the assumption that jet aeration and mixing equipment would be used. However, the minimum operating water level is subject to change during final design, based on the recommendations of the mixing and aeration equipment manufacturer(s).

A total EQ basin volume of 3.0 MG is recommended as the basis of design. This total volume includes the 2.5 MG working volume determined from the hydrographs, a freeboard of 12 inches, and a minimum operating water level of 3 feet.

# 4. BASIN CONFIGURATION AND CONSTRUCTION

New flow equalization basins may be constructed out of earthen, concrete, or steel materials. Lined earthen basins are typically the cheapest alternative where sufficient land area is available and geotechnical conditions are favorable. Concrete basins may be either rectangular or circular and may be constructed as cast-in-place structures, modular precast/prestressed basins, or prestressed concrete tanks. Steel tanks of this size are circular and are typically constructed of epoxy coated or glass/porcelain enameled bolted steel panels with a concrete floor slab and foundation. Despite their cost advantages, lined earthen basins were not considered in this evaluation for the City of Hendersonville's WWTF due to the limited land area available at the site. This section details the evaluation of the following three primary alternatives for EQ basin configuration and construction:

1. Rectangular cast-in-place (CIP) concrete basin
2. Circular prestressed concrete tank
3. Circular bolted steel tank

Available land area is limited at the existing WWTF, so a side water depth (SWD) of 20 feet was selected for all alternatives to balance space requirements with EQ pumping requirements. The resulting dimensions for a rectangular basin are 200' x 100' x 21' (length x width x height), with a SWD of 20 feet, resulting in a total water volume of approximately 2.99 MG (400,000 $ft^3$). The circular basin alternatives require a diameter of 160 feet and a height of 21 feet, with a SWD of 20 feet, for a total water volume of approximately 3.00 MG (402,124 $ft^3$).

## 4.1 Alternative 1: Rectangular Cast-In-Place Concrete Basin

A rectangular CIP concrete basin may be constructed by forming, reinforcing, and pouring the concrete structure on-site using typical concrete design and construction practices. CIP concrete structures are extremely versatile because they can be designed and constructed to meet virtually any dimensions or configuration, and are easily compartmentalized. Concrete construction is generally well suited to wastewater environments, and CIP concrete basins are typically expected to have a design life of 50 years or more. However, CIP concrete construction is both labor and time intensive, which leads to higher construction costs. CIP concrete construction requires a high level of skilled labor to ensure proper and timely pouring, molding, and curing. CIP concrete basins also require the longest construction time compared to prestressed concrete tanks and bolted steel tanks, and adverse weather conditions can significantly impact the construction schedule.

A rectangular geometry was selected for this alternative because it is the most space efficient shape. A rectangular CIP concrete basin allows for common-wall construction with adjacent processes, such as screening and/or grit removal, and requires less space for any future expansions to flow EQ facilities. However, a rectangular geometry will require thicker walls compared to a circular concrete basin due to the structural inefficiency of a rectangle. A list of the advantages and disadvantages of this alternative are listed in **Table 4.1** below. The estimated cost for this alternative is found in **Table 4.4** at the end of this section.

*Table 4.1 – Advantages and Disadvantages of a Rectangular CIP Concrete Basin*

| Rectangular CIP Concrete Basin | |
|---|---|
| **Advantages** | **Disadvantages** |
| Most space efficient basin geometry | Higher capital costs |
| Allows for versatile design and easily compartmentalized | Longest construction time |
| Long design life (≥ 50 years) | |
| May be bid competitively | |

## 4.2 Alternative 2: Circular Prestressed Concrete Tank

There are four types of circular prestressed concrete basins; I) cast-in-place concrete with vertical prestressed reinforcement, II) shotcrete with a steel diaphragm, III) precast concrete panels with a steel diaphragm, and IV) cast-in-place concrete with a steel diaphragm. All four tank types require specialized construction methods, such as circumferentially prestressing the wall with steel wire or strand. As a result, tank construction is typically performed by the tank manufacturer. The primary difference between the four types of prestressed tanks is the means and methods of construction, so final prestressed tank type selection may be largely influenced by the proximity or availability of qualified tank manufacturers at the time of project implementation. The most common type of prestressed concrete tank in the water and wastewater industry in the southeast United States is the Type II tank due to the proximity of prestressed concrete tank contractors.

Like traditional CIP concrete basins, prestressed concrete tanks are typically expected to have a design life of 50 years or more. Adverse weather conditions can also significantly impact the construction schedule, but they are typically constructed faster than CIP concrete basins. If partitioning of the tank is desired, circular prestressed concrete tanks require special designs to include internal partition walls. The increased cost of a specially designed prestressed concrete tank may be more expensive than constructing two smaller single compartment prestressed concrete tanks. Both CIP concrete basins and prestressed concrete tanks have lower maintenance costs than epoxy coated steel tanks, which require sand blasting and repainting of the tank interior and exterior. Type II prestressed concrete tanks typically do not require interior coatings to protect the shotcrete surface from corrosion, which results in significant cost savings in construction and maintenance. The high cement content of shotcrete provides excellent protection against corrosion under most conditions, with the exception of industrial strength wastewaters. The steel diaphragm in prestressed tank types II, III, and IV provides an additional defense against leaks, but may be susceptible to corrosion if construction quality control is inadequate. A list of the advantages and disadvantages of this alternative are listed in **Table 4.2** below. The estimated cost for this alternative is found in **Table 4.4** at the end of this section.

*Table 4.2 - Advantages and Disadvantages of a Circular Prestressed Concrete Basin*

| Circular Prestressed Concrete Basin | |
|---|---|
| **Advantages** | **Disadvantages** |
| Lowest capital cost | Longer construction time than bolted steel tanks |
| Low maintenance costs and requirements | Multiple tanks required to provide compartmentalization |
| Faster construction time than CIP concrete basins | Cannot be expanded using common wall construction |
| Long design life (≥ 50 years) | Less space efficient than rectangular CIP concrete basin |
| May be bid competitively | |
| Enhanced leak protection with use of steel diaphragm | |
| Interior coatings typically not required | |

## 4.3 Alternative 3: Circular Bolted Steel Tank

Bolted steel tanks have a fast construction time, as installation and assembly is straight forward. Construction requires limited amounts of skilled labor, can be completed without cranes or special equipment, and is not significantly impacted by weather. Of the two coating systems available (factory-applied epoxy and factory-applied glass/porcelain enamel) for bolted-steel tanks, the glass/porcelain enamel, which is also referred to as a glass-fused-to-steel (GFS) system, is recommended for this application. The City of Hendersonville has several GFS bolted steel tanks installed throughout their water system over the last 17 years and has been satisfied with their construction quality and low maintenance requirements. The GFS system is also effective at holding aggressive liquids such as domestic wastewater, has a higher bonding rating and hardness than epoxy systems, never requires recoating, and requires less maintenance than epoxy-coated bolted steel tanks to achieve the same lifespan. The bolted design allows for relatively simple maintenance (damaged panels are typically replaced or the affected area is covered with a sealant in the field), and future expansion can be achieved within the same footprint by adding vertical rings of panels to the tank.

Bolted steel tanks must be emptied for certain maintenance and repair tasks and cannot be compartmentalized to maintain partial storage capacity. Other disadvantages to GFS bolted steel tanks include the increased likelihood of leaks due to the large number of bolted and gasketed joints, low impact resistance of the GFS coating system, the requirement of a cathodic protection system, and higher capital costs than prestressed concrete tanks and epoxy-coated bolted steel tanks. A list of the advantages and disadvantages of this alternative are listed in **Table 4.3** below. The estimated cost for this alternative is found in **Table 4.4** at the end of this section.

*Table 4.3 - Advantages and Disadvantages of a Circular Bolted Steel Tank*

| Circular Bolted Steel Tank | |
|---|---|
| **Advantages** | **Disadvantages** |
| Low maintenance costs and requirements | 2nd highest capital cost |
| Fastest construction time | Shorter design life (≥ 20-25 years) |
| GFS coating system is extremely resistant to corrosion in WW environments | Multiple tanks required to provide compartmentalization |
| Tanks have performed well in City's water system | Cannot be expanded using common wall construction |
| May be bid competitively | Uses more land area that rectangular CIP concrete basin |

## 4.4 Capital Cost Comparison of Tank Alternatives

Preliminary opinions of construction costs for each tank geometry and construction method alternative were developed based on budgetary proposals from tank suppliers/contractors and recent bid prices from similar projects. The capital cost summary provided in **Table 4.4** below is based on the construction cost for the tank, including typical site preparation costs assuming the tank is constructed at the old plant site across the road from the existing WWTF. Additional discussion and comparison of siting alternatives is discussed in the later sections of this document.

*Table 4.4 - Construction and Configuration Basin Costs*

| Alternative | Geometry and Construction Method | Capital Cost* |
|---|---|---|
| 1 | Rectangular Cast in Place Concrete Basin | $2,900,000 |
| 2 | Circular Prestressed Concrete Tank | $1,500,000 |
| 3 | Circular Glass-Fused-To-Steel Bolted Steel Tank | $2,700,000 |

**Assumes tank/basin is built at the old plant site*

Based on the analysis above, and the listed advantages of circular prestressed concrete tanks, it is recommended that a circular prestressed concrete tank be implemented for flow equalization at the Hendersonville WWTF.

# 5. EQ BASIN APPURTENANCES

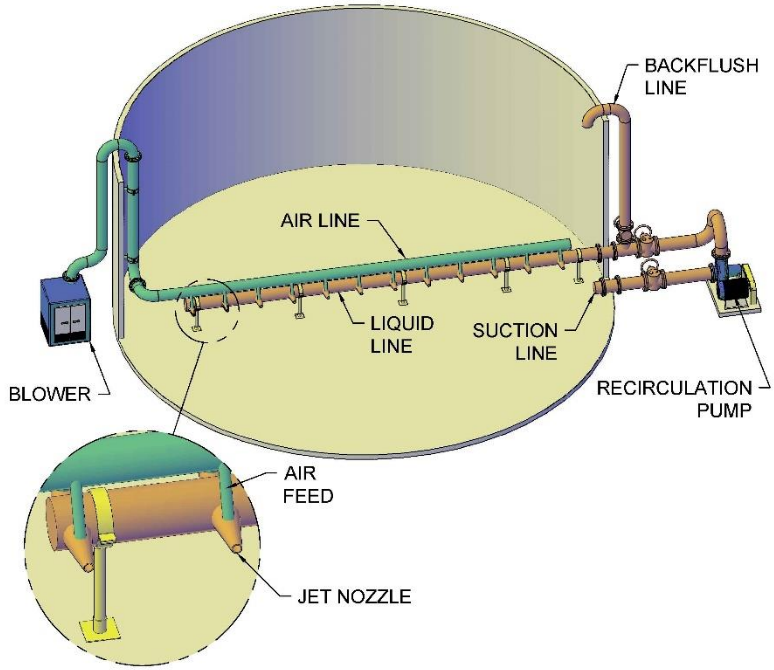
## 5.1 Mixing and Aeration

Mixing and aeration systems are recommended to be installed within the proposed flow equalization basin to maintain solids in suspension and to prevent septic conditions and odor issues in the EQ basin. Per the Ten State Standards, aeration systems are recommended for all EQ basins and should be designed to maintain a DO concentration of at least 1.0 mg/L. The following sections compare commonly utilized mixing and aeration technologies for flow EQ basins.

### 5.1.1 Jet Aeration and Mixing

Jet aeration and mixing equipment utilizes a fixed piping system within the EQ basin to both mix and aerate the wastewater. A jet aeration and mixing system includes pumps, blowers, and a jet aeration and mixing manifold. The jet aeration and mixing manifold can be constructed out of either stainless steel or FRP and includes both a liquid line and an air line, as shown in **Figure 5.1** below. The jet mixing pump(s) recirculate the EQ basin contents through the liquid line in the manifold and discharges the liquid through a series of nozzles along the manifold. The blower(s) discharge low pressure air through the air line of the manifold which is discharged through each nozzle along with the recirculated wastewater to entrain the air and produce fine bubble aeration.

*Figure 5.1 – Jet Aeration and Mixing System Example Schematic*




*Source: Mixing Systems, Inc. (https://www.mixing.com/jet-aeration)*

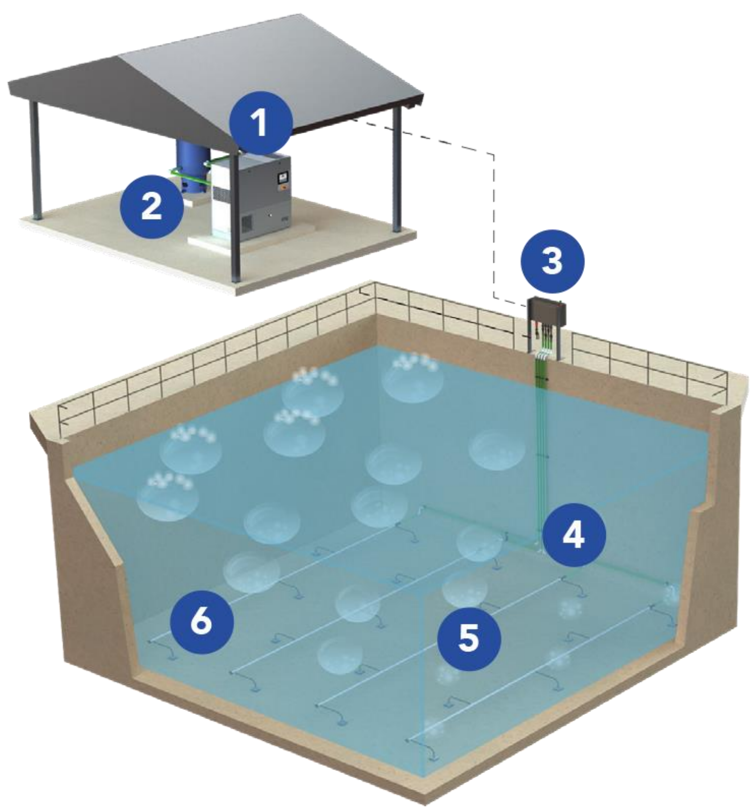
Jet aeration and mixing systems have many advantages for use in EQ basins. The fine bubbles produced by the jets results in a very high oxygen transfer efficiency and design alpha values of 0.9 or higher. The high oxygen transfer efficiency of these systems requires less air and lower energy usage compared to diffused or mechanical aeration systems to provide the same oxygen transfer rate. The pumped recirculation of the EQ basin contents through the manifold nozzles produces high velocity mixing throughout the entire tank with a small amount of pumped liquid and low energy usage. Typically, a jet aeration and mixing system has no moving parts within the EQ basin, with the jet motive pumps and blowers located outside of the basin. Mixing and aeration are independently controlled with jet aeration and mixing to maximize energy efficiency. As a result, jet aeration and mixing systems have a long life cycle and are very easy to operate and maintain. Jet aeration and mixing systems can also be supplied with submersible pumps installed on guiderails within the basin to reduce construction costs.

The primary disadvantage of jet aeration and mixing systems is that the jet aeration and mixing manifold may become plugged with debris from the recirculated wastewater. However, these systems are often supplied with an optional built-in backflush assembly to allow self-cleaning without entering or draining the basin. The backflush assembly uses additional piping and valves to allow the jet motive pumps to reverse flow through the jet aeration and mixing header to unclog plugged nozzles and discharge the wastewater back into the EQ basin.

### 5.1.2 Compressed Gas Mixing + Diffused Aeration

Compressed gas mixing and diffused aeration can be coupled together to provide mixing and aeration for EQ basins. As shown in **Figure 5.2**, compressed gas mixing uses a system of valves and piping manifolds with nozzles near the tank floor to fire programmed, short-duration bursts of compressed air to mix the basin from the bottom up. Compressed air for mixing is supplied by a compressor (1) and receiver tank (2) located outside of the basin. A valve module (3) located outside of the basin controls the pressure, frequency, duration, and sequence of firing to ensure complete mixing throughout the basin. All moving parts associated with the compressed gas mixing system are located outside of the basin. The compressed gas mixing system is specifically designed to produce large air bubbles to rapidly mix the contents of the basin. Due to the large bubble size used with this system, there is essentially no oxygen transferred to the basin contents, which allows mixing and aeration to be controlled independently.

*Figure 5.2 – Compressed Gas Mixing System Example Diagram*




*Source: EnviroMix, Inc. (https://enviro-mix.com/technology/)*

Aeration for the EQ basin is supplied using a diffused aeration system consisting of a network of fine bubble diffusers and blowers to supply the air needed. The diffuser grid is installed near the basin floor along with the headers and nozzles for the compressed gas mixing system. Turbo blowers or positive displacement blowers may be used to supply the air required.

This combination of systems to provide mixing and aeration has many advantages including:

- Separate control of aeration and mixing, allowing energy efficient aeration when needed
- No moving parts within the EQ basin
- All compressed gas mixing system components within the basin are non-clogging and essentially maintenance free
- Energy efficient mixing; compressor and valve module are the only components requiring power
- Mixing intensity is adjustable based on basin liquid level and solids content

The primary disadvantage of this system is that it still requires in-basin maintenance to clean and periodically replace aeration diffusers. This would require complete shutdown of the EQ basin which could potentially subject the WWTF to peak flow events exceeding its hydraulic capacity if multiple EQ basins or compartments are not provided. In addition, the energy efficiency provided by the compressed gas mixing system is negated when the diffused aeration system is operated to maintain aerobic conditions. Aeration is expected to be required for the EQ basin under most operating conditions.

### 5.1.3 Submerged Mechanical Mixers + Diffused Aeration

Like the compressed gas mixing and diffused aeration alternative, separate control of mixing and aeration within the EQ basin can also be provided using submerged mechanical mixers along with a diffused aeration system. Submerged mechanical mixers may be mounted on guiderails along the sides of the EQ basin to provide the mixing energy required. A network of fine bubble diffusers may be provided within the basin along with turbo or positive displacement blowers to provide the aeration required to maintain aerobic conditions. This option typically results in lower capital cost than other alternatives, but it is best suited for small basins due to the limited range of submerged mechanical mixers. When used in large EQ basins, unmixed dead zones resulting in reduced storage capacity due to deposited solids are expected with this alternative. This alternative is also typically less favorable than other mixing and aeration systems due to the increased maintenance requirements associated with submerged mechanical mixers and the cleaning and replacement requirements for diffusers. This alternative is also less energy efficient than jet aeration and mixing as well as compressed gas mixing coupled with diffused aeration.

### 5.1.4 Mechanical Aerators

Mechanical aerators may be installed within the EQ basin to provide both mixing and aeration. Floating mechanical aerators are typically used in this alternative, which may be anchored in position using several methods:

- Mooring cables connected to the floating aerator and shore mounted posts or anchors
- Fixed mooring posts installed within the basin which allow the floating aerator to move up or down with the liquid level
- Pivotal mooring which uses an articulating arm fixed to the floating aerator and a shore mounted anchor point which allows the arm to pivot with varying liquid level

Unlike other alternatives, mixing and aeration cannot be controlled separately with mechanical aerators. This alternative typically has the lowest capital cost due to its simplicity, but it also typically the least energy efficient and most maintenance intensive. Similar to submerged mechanical mixers, unmixed dead zones are very common with mechanical aerators. Unmixed dead zones may also develop very low DO concentrations and result in increased odor problems with mechanical aerators.

### 5.1.5 Selected Alternative

Based on the comparison of advantages and disadvantages of each option described above, and comparison of capital costs, jet aeration and mixing is recommended to ensure complete mixing and efficient aeration is provided. The capital cost for the jet aeration and mixing system is estimated to be approximately $600,000, which would include three 33 hp external jet motive pumps, optional backflush assembly, two 100 hp turbo blowers, VFDs for pumps and blowers, and associated piping. This estimated capital cost was nearly equivalent to the compressed gas mixing and diffused aeration alternative, which had an estimated capital cost of approximately $625,000. Mechanical aerators may be utilized for significantly lower capital cost. However, this is not recommended due to increased O&M requirements, lower energy efficiency, and a high likelihood of reduced storage capacity and increased odor issues related to deposition of solids in mixing dead-zones.

## 5.2 Flow and Level Control

Flow and level control are vital components of all flow equalization facilities. Regardless of whether inline or offline flow equalization is provided, flow meters, level indicating instruments, and control valves or weirs must be provided.

Flow measurement is recommended to be provided on the total plant influent flow prior to equalization, and on the secondary process influent following flow equalization. The WWTF influent flow rate is currently monitored using a Parshall flume. A Parshall flume is an extremely simple, low-cost, accurate method to measure open channel flows. A Parshall flume may continue to be used on the WWTF influent line to measure the plant influent flow rate prior to any recycle or waste flows. However, it is known that the existing Parshall flume regularly experiences flooding during high flow events and is undersized for future peak influent flows to the facility. As a result, the City wishes to avoid future use of a Parshall flume for influent flow measurement. It should be noted, expansion of the influent pumping station firm capacity and implementation of flow EQ facilities is expected to prevent flooding of the influent pump station and Parshall flume.

The existing Parshall flume has a maximum capacity of 21.39 MGD based on a 2-ft Parshall flume (throat width) and a maximum head of 2.5 feet. If the City continues to use a single Parshall flume, a new flume with a throat width of at least 30 inches would be needed to provide influent flow measurement for the 2025 and 2040 peak influent flow rates of 22.5 MGD and 28.3 MGD, respectively. A new Parshall flume may be installed immediately downstream of each new mechanical bar screen if a new screening facility is constructed ahead of the influent pumping station. The use of a Parshall flume immediately downstream of mechanical screens provides downstream water level control for each screen and provides redundancy for influent flow measurement by providing multiple Parshall flumes. If desired, electromagnetic flow meters may be provided on the force mains from the influent pump station to measure influent flow in lieu of the Parshall flume. Additional flow measurement devices would be required on any recycle and waste streams sent to the influent pump station so that these flows may be subtracted from the total flow rate to meet regulatory influent flow monitoring requirements. Secondary process influent flows are recommended to be monitored using an electromagnetic flow meter(s) due to their accuracy, reliability, and because this flow occurs in a closed conduit.

Level monitoring is recommended to be provided within the flow EQ basin using either an ultrasonic level sensor, a radar level sensor, or a system of floats. An ultrasonic level sensor is recommended because of its ease of use, accuracy, and reliability. The EQ basin level measurement will be used to provide on/off controls for pumping to and/or from the EQ basin, or valve open/close control depending on whether pumped flow or gravity flow arrangements are provided.

## 5.3 Other Appurtenances

### 5.3.1 Electrical Controls and Instrumentation

Electrical controls will be located in a small building at ground level near the EQ basin. Monitoring equipment such as water level indicators, pump status, blower status, valve status, flow monitors, DO monitors, etc. will be located within this structure as appropriate. All control system components such as PLC, HMI graphic display, fiber ethernet switch, and other components will also be housed in this structure. Local controls will be available at this structure, and all monitoring and controls capabilities will also be communicated back to the WWTF’s main SCADA system for remote monitoring and operation from the administration building.

### 5.3.2 Basin Access

Access to the top and interior of the EQ basin must be provided to allow periodic cleaning and maintenance activities. For all alternatives, it is recommended that a top walkway be provided around the circumference of the basing for cleaning and inspection purposes. Access to the top walkway should be provided by an exterior staircase. Access to the interior of the tank should be provided by a stainless steel, aluminum, or fiberglass ladder.

### 5.3.3 Washdown System

A series of yard hydrants and water cannons are recommended to be positioned at multiple locations around the EQ basin to allow for periodic cleaning. Water for washdown of the EQ basin is recommended to be provided via the plant non-potable water system. A water booster pump system may be needed to supply the necessary pressure for water cannons to achieve adequate tank cleaning.

### 5.3.4 Valves and Piping

All control valves necessary for operation of the flow EQ basin will be located either above grade or in a below grade vault with an access hatch, vault drain or sump pump, and electrical appurtenances necessary for control and status monitoring. Control valves for EQ basin effluent flow rate control should be either modulating plug valves or control pinch valves. All piping for flow to and from the EQ basin is recommended to be ductile iron pipe with restrained joints where necessary. All above grade piping shall be either flanged or welded. All piping for jet aeration and mixing equipment is recommended to be stainless steel and/or FRP.

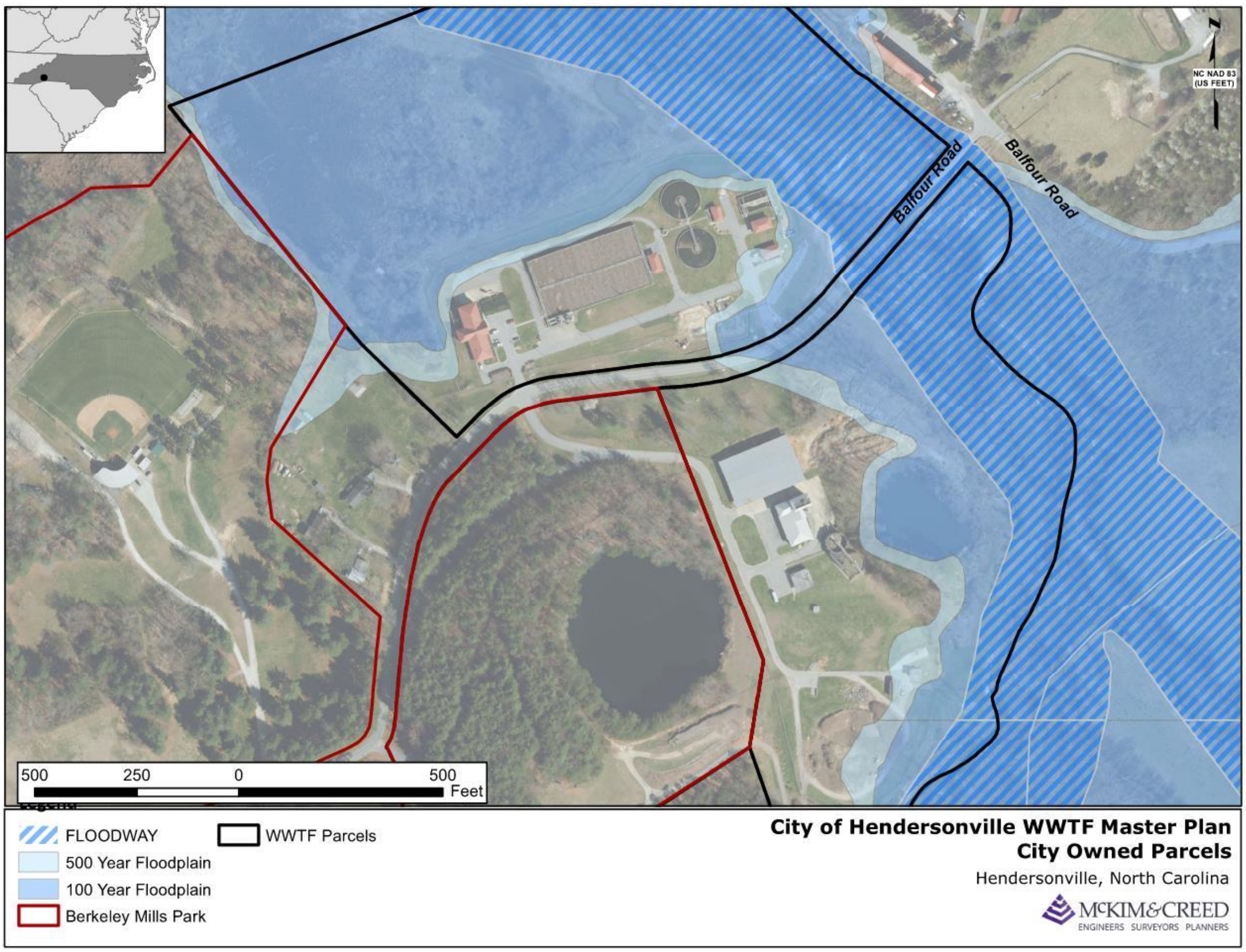
# 6. LOCATION & TYPE OF FLOW EQUALIZATION FACILITIES

The location and type (inline or offline) of flow equalization facilities selected both have significant impacts on the capital and O&M costs of the project due to their impacts on pumping and piping requirements. These two considerations were evaluated together due to their interrelated nature. Four alternatives were developed based on two potential locations for flow EQ facilities and the two types of flow EQ possible. The sections below provide general descriptions of the considerations required for location and type of flow EQ facilities, and analysis of the four alternatives evaluated.

## 6.1 Potential Locations of Flow EQ Facilities

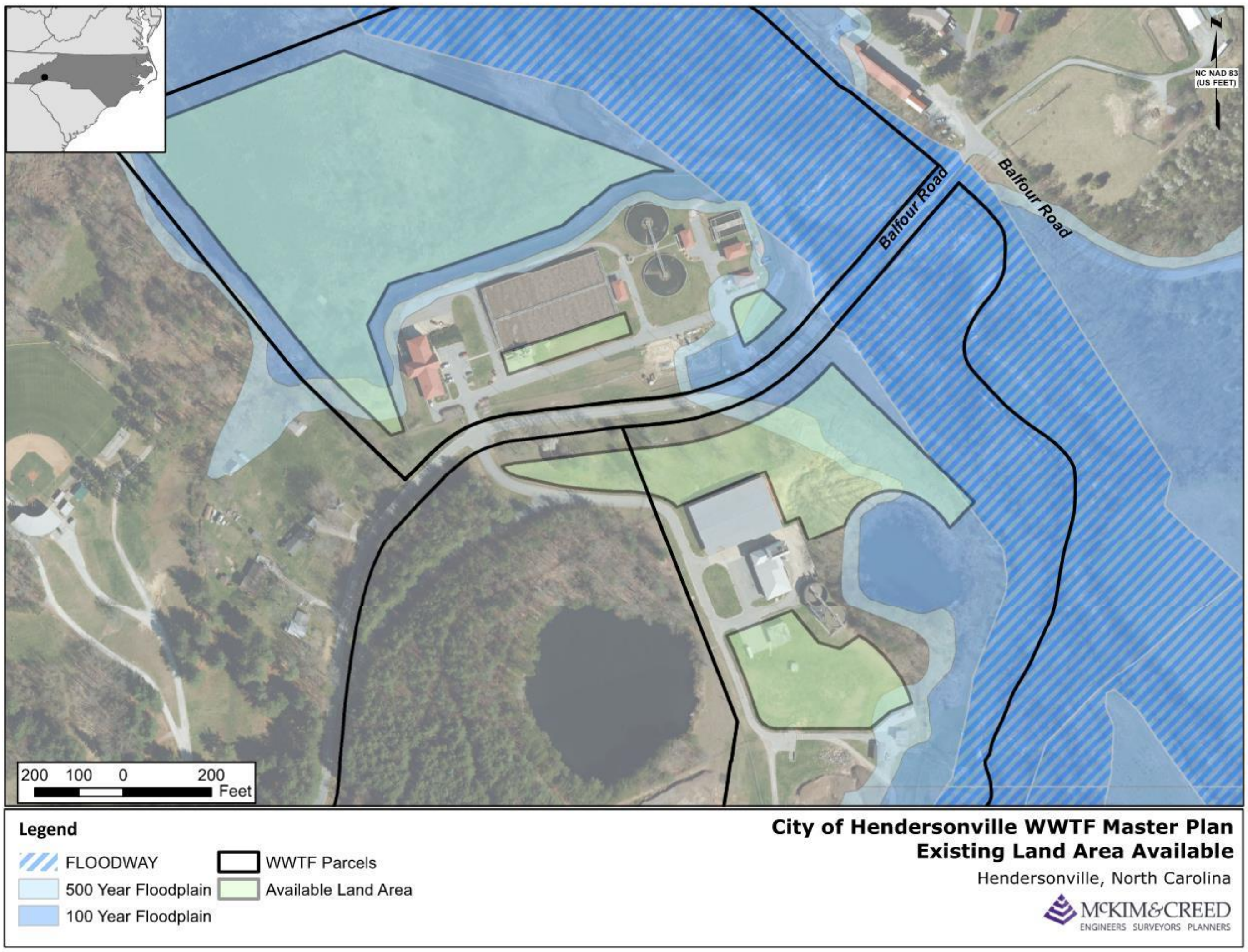
The City of Hendersonville owns 53.64 acres on the existing WWTF site, along with 54.83 acres adjoining the WWTF property, as shown in **Figure 6.1** below. The 54.83 acre City-owned property directly to the west of the WWTF is Berkeley Mills Park, which consists of a baseball/softball field, greenways, and natural greenspace. Berkeley Mills Park was assumed to be unavailable for potential siting of the proposed flow equalization facilities due to its heavy public use and its value to the local community. However, if necessary and acceptable to the City, sufficient area for future flow equalization facilities exists on the Berkeley Mills Park property, immediately east of the baseball field adjacent to the WWTF site. This area is mostly wooded, is higher in elevation than the WWTF site, and is outside of the existing 100-year flood zone for Mud Creek. Geotechnical conditions are unknown at this location, however, they are expected to be more favorable than geotechnical conditions at the existing WWTF site since this area is outside of the flood zone.

*Figure 6.1 – City of Hendersonville Owned Property at the WWTF*




On the 53.64 acre WWTF property, a majority of the available land area north of Balfour Road is located within the 100-year flood zone of Mud Creek as shown in **Figure 6.2** below. This area is approximately 10 to 20 feet lower in ground elevation than the existing WWTF site. The area north of the existing WWTF within the 100-year flood zone is also suspected to include potential wetland areas that would require environmental permitting for both temporary construction disturbance and permanent site disturbance. Construction of flow EQ facilities north of the existing WWTF site would require a large amount of site fill to raise it above the 100-year floodplain elevation, would likely require pile foundations for the EQ basin and other water-bearing structures, and is expected to require environmental and flood zone permitting. Despite these potential challenges, this land area is the only viable location for flow EQ facilities on the existing WWTF site north of Balfour Road. Therefore, it is prudent that this location be evaluated for siting of the proposed EQ facilities.

*Figure 6.2 – Available Land Area on WWTF Property*



Additional land area is available on the existing WWTF property south of Balfour Road on the site of the old WWTF, as shown in **Figure 6.2**. This site south of Balfour Road is now used for solids handling processes to treat the waste activated sludge produced from the WWTF. The City's yard waste and storm debris mulching operations are also located on the old plant site at the southern-most portion of the property, where the old oxidation ditch was previously located prior to demolition. Available land area for flow EQ facilities on this site are located to the east of the existing covered storage shelter, and between the old administration building and the City's mulching operations area. The area to the east of the existing covered storage shelter is recommended to be reserved for future solids handling processes such as thickened WAS storage and a dried biosolids storage and truck load-out station.

The area located between the old administration building and the City's mulching operations area is expected to be a viable location for proposed flow EQ facilities. The site elevation at this location is above the elevation of the existing WWTF across Balfour Road, is located outside of the 100-year flood zone of Mud Creek, and is expected to have adequate geotechnical conditions. It is worth noting that several old treatment process structures were located in this area prior to demolition, including a trickling filter and secondary clarifier. Remnants of structural foundations and other features are expected to be encountered below grade in this area, which may impact construction costs for site excavation and foundation preparation. The old sludge pump station building still stands in this area and would require demolition for the EQ basin construction. Based on discussions with City staff, the old administration building in this area is still used by field operations crews as a meeting space and staging area. The City prefers that the old administration building remain in place if feasible.

Based on the analysis of City owned lands described above, two land areas are recommended for further evaluation as potential sites for the proposed flow EQ facilities. The two sites recommended for further evaluation are as follows:

1. North of the existing WWTF located within the 100-year flood zone of Mud Creek on the WWTF property
2. South of Balfour Road on the WWTF property at the old plant site, located between the old administration building and the City's mulching operations area

To further clarity option 1 above, the recommended location of the proposed flow EQ facilities for this alternative is immediately north of the current administration building. This location north of the existing WWTF is recommended for the following reasons:

1. To avoid conflicts with future expansions of process basins, including aeration basins, secondary clarifiers, and tertiary filters

2. To avoid conflicts with the existing Duke Energy transmission easements located on the eastern portion of the WWTF property
3. To locate proposed facilities at the furthest extents of the existing flood zone to reduce potential impacts to flood elevations and construction feasibility
4. To limit site fill requirements as much as feasible

## 6.2 Inline vs. Offline Flow Equalization

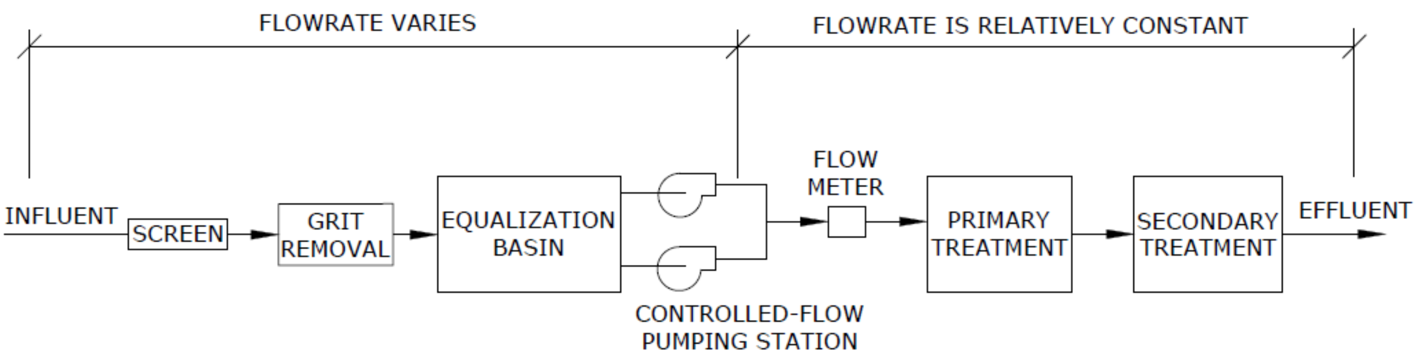
Flow equalization may be provided in either inline or offline arrangements, with both arrangements preferably located after screening and grit removal to reduce O&M requirements and odor issues caused by ragging and the accumulation of grit, scum, and large solids. Inline and offline flow equalization arrangements are differentiated by their location in the WWTF's flow path, the pumping facilities and piping required, and their impacts on constituent mass load equalization in the influent wastewater. Both options were considered in this evaluation to identify their impacts on treatment process operation and performance, and their impacts on capital and O&M costs. Regardless of which type of flow EQ is chosen, it is recommended that the City follow the standard convention to provide screening and grit removal ahead of the EQ basin. To accommodate this recommendation, the following assumptions were made:

- The existing influent pump station will be expanded to increase firm pumping capacity to accommodate the peak flow rates determined from the 2-year storm hydrographs
- Influent screening would be relocated upstream of the existing influent pumping station to protect the influent pumps from ragging and accelerated wear
    - This is based on condition assessment recommendations made in Technical Memorandum #1 of this Master Plan
- Grit removal would be relocated adjacent to the new flow equalization basin to increase capacity and improve performance compared to the existing aerated grit chambers
    - This is based on condition assessment recommendations made in Technical Memorandum #1 of this Master Plan

### 6.2.1 Inline Flow Equalization

With inline flow equalization, 100% of influent wastewater is sent to the EQ basin after screening and grit removal, before moving on to the other treatment units in the plant. A typical flow diagram for inline flow equalization is shown below in **Figure 6.3**. Inline flow equalization is typically used to provide influent flow and mass loading equalization of diurnal patterns to improve downstream treatment process efficiency and settling performance. Inline flow equalization can also be used to provide wet weather flow equalization if sized appropriately.

*Figure 6.3 – Typical Flow Diagram for Inline Flow Equalization*




Based on the flow diagram shown above, inline flow equalization would require pumping and piping improvements to redirect process flow to a new EQ basin prior to the aeration basins. All process flow to and from the EQ basin must be metered to properly monitor and control the EQ storage volume available and the flow rate to the downstream processes. It is typical for solids handling waste flows and filter backwash flows to be sent to the inline EQ basin prior to retreatment. Influent flow to the EQ basin must be pumped, regardless of which location alternative is selected due to site constraints. Effluent flow from the EQ basin may utilize either gravity flow or pumped flow depending on the EQ basin site elevation. If gravity flow is used, it is recommended that an automated control valve be used on the effluent line to modulate its position and control the flowrate based on feedback from an effluent flow meter. If the EQ effluent is pumped, it is recommended that VFD driven pumps be used to accurately control the flow rate based on feedback from an effluent flow meter. **Table 6.1** below summarizes the expected flow rates associated with inline flow equalization alternatives based on the 2-year storm hydrographs and the recommended WWTF hydraulic capacity limits described earlier.

*Table 6.1 – Inline Flow Equalization Influent and Effluent Flow Rates*

| Design Year | 2-Year Storm Peak Flow Rate (MGD) | Maximum EQ Basin Influent Flow Rate (MGD) | Maximum EQ Basin Effluent Flow Rate (MGD) |
|---|---|---|---|
| 2025 | 22.5 | 22.5 | 15 |
| 2040 | 28.3 | 28.3 | 19.5 |

The advantages of inline flow equalization include:

- Flow equalization can be provided for both diurnal peaks as well as peak wet weather flows

- When sized for peak wet weather flows, ensures the hydraulic capacity of the WWTF is not exceeded, and prevents SSOs
- Mass loading equalization is provided under normal and peak conditions, which stabilizes pH and dilutes shock loadings and inhibiting substances
- Improves biological process and settling performance because solids loading is stabilized and shock loadings are diluted, resulting in reduced energy demand for aeration and improved effluent quality
- Tertiary filtration rates and solids loading to tertiary filtration is reduced, resulting in improved performance and more uniform filter backwash requirements
- Improved settling and filtration performance reduces effluent turbidity, resulting in reduced disinfection dose requirements and reduced energy demand for UV disinfection

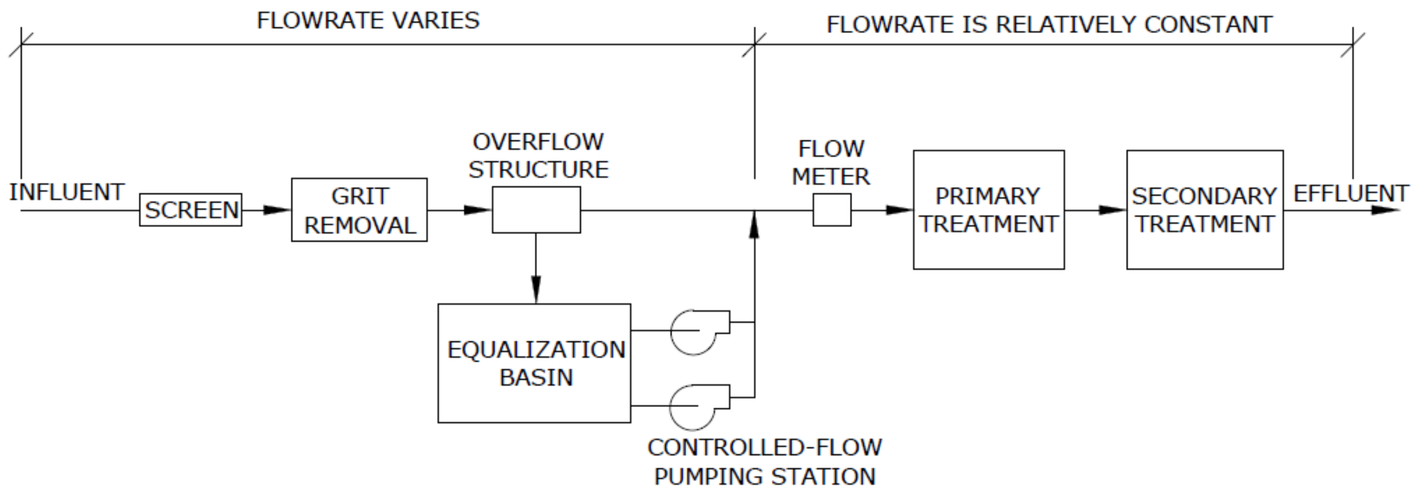
Disadvantages of inline flow equalization include:

- More pumping energy is required to constantly pump flow to flow equalization under normal operating conditions
- Additional operation and maintenance is required for constant pumping and use of EQ basin mixing and aeration equipment
- May be more costly to integrate into the existing WWTF's flow path

### 6.2.2 Offline Flow Equalization

Contrary to inline flow equalization, offline flow equalization is primarily used to equalize large flow peaks associated with wet weather to ensure the hydraulic capacity of the facility is not exceeded. Offline flow equalization does not provide equalization of influent loading and therefore does not provide any benefits to the secondary treatment process or settling performance during normal flow conditions. As the name implies, offline flow equalization facilities do not directly receive influent wastewater. A typical flow diagram for offline flow equalization is shown below in **Figure 6.4**. With offline flow equalization, an overflow structure or pumping arrangement diverts excess influent flow to the EQ basin under peak conditions. The diverted wastewater is stored until the peak flow conditions have subsided and is then returned to the normal flow stream for further treatment using controlled rate pumping or gravity flow.

*Figure 6.4 – Typical Flow Diagram for Offline Flow Equalization*




Typically, it is preferrable for offline flow equalization facilities to utilize a gravity flow in, pumped flow out arrangement to limit pumping energy requirements for peak wet weather flows. However, this arrangement is not expected to be feasible for the City of Hendersonville's WWTF due to the depth of the incoming gravity sewer outfall and existing influent pumping station, and the site topography near the existing influent pumping station. Gravity flow into an EQ basin at the WWTF site would require a new offline EQ basin to be constructed entirely below grade within existing floodplain north of Balfour Road. As a result, offline flow equalization alternatives for the City of Hendersonville's WWTF would still require all influent flow to be pumped to a new overflow structure to divert excess flow to a new EQ basin.

Based on the flow diagram shown above, all influent flow must be pumped to a new overflow structure ahead of the existing aeration basins. Plant recycles from solids handling process and filter backwash are recommended to be returned to either the overflow structure or directly to the Equalization basin if preferred. Influent flows exceeding the desired downstream treatment flow rate would overtop a diversion weir in the overflow structure and be sent to the EQ basin by either gravity flow or pumped flow depending on site selection. Offline flow EQ would still require two flow meters to allow monitoring of plant influent flow and flows to the downstream processes. EQ basin effluent flow is recommended to be controlled by an automated control valve or by VFD driven pumps, depending on site selection, based on flow signal from the downstream flow meter.

**Table 6.2** below summarizes the expected flow rates associated with offline flow equalization alternatives based on the 2-year storm hydrographs and the recommended WWTF hydraulic capacity limits described earlier. As seen below, the maximum effluent flow expected from an offline EQ basin is significantly lower than an inline EQ basin. An offline EQ basin only needs to be able to be completely drained within a 24-

hour period, therefore effluent flow capacity may be based on the volume of the basin. This illustrates the benefit of providing gravity flow to an offline EQ basin, and pumped flow out to limit energy usage.

*Table 6.2 – Offline Flow Equalization Influent and Effluent Flow Rates*

| Design Year | 2-Year Storm Peak Flow Rate (MGD) | Maximum EQ Basin Influent Flow Rate (MGD) | Maximum EQ Basin Effluent Flow Rate (MGD) |
|---|---|---|---|
| 2025 | 22.5 | 7.5 | 3.0 |
| 2040 | 28.3 | 8.8 | 3.0 |

The advantages of offline flow equalization include:

- Ensures the hydraulic capacity of the WWTF is not exceeded, and prevents SSOs
- Limits pumping energy required and associated operating costs
- May be easier to integrate into the existing WWTF's flow path

The disadvantages of offline flow equalization include:

- Only provides flow equalization for wet weather flow events, does not provide mass loading equalization
- No improvements to treatment performance under normal operating conditions
- No protection against shock mass loading and inhibitory substances

## 6.3 Alternatives Analysis

Four alternatives were developed for the proposed flow EQ facilities based on the two potential locations and the two flow EQ types described earlier. The four alternatives where developed based on the matrix comparison of location and EQ type as seen in **Table 6.3** below.

*Table 6.3 – Matrix of Flow EQ Facilities Alternatives*

| EQ Type / Location | Existing WWTF Site | Old Plant Site |
|---|---|---|
| Inline EQ | Alternative 1 | Alternative 2 |
| Offline EQ | Alternative 3 | Alternative 4 |

**Figure 6.5** and **Figure 6.6** below show preliminary locations of the proposed circular prestressed concrete EQ basin, piping, and other associated structures for each of the four alternatives. Alternatives 1 and 2 are shown in **Figure 6.5**, and alternatives 3 and 4 are shown in **Figure 6.6**. Analysis of alternatives and recommendations for improvements to the influent pumping station, screening, and grit removal are provided in Technical Memorandum #2 of this Master Plan. The capital and O&M costs for improvements to influent pumping, screening, and grit removal are not included in the following cost estimates for each

flow EQ alternative. Cost estimates for recommended improvements to influent pumping, screening, and grit removal are included in Technical Memorandum #2 of this Master Plan.

*Figure 6.5 – Inline EQ Alternatives*

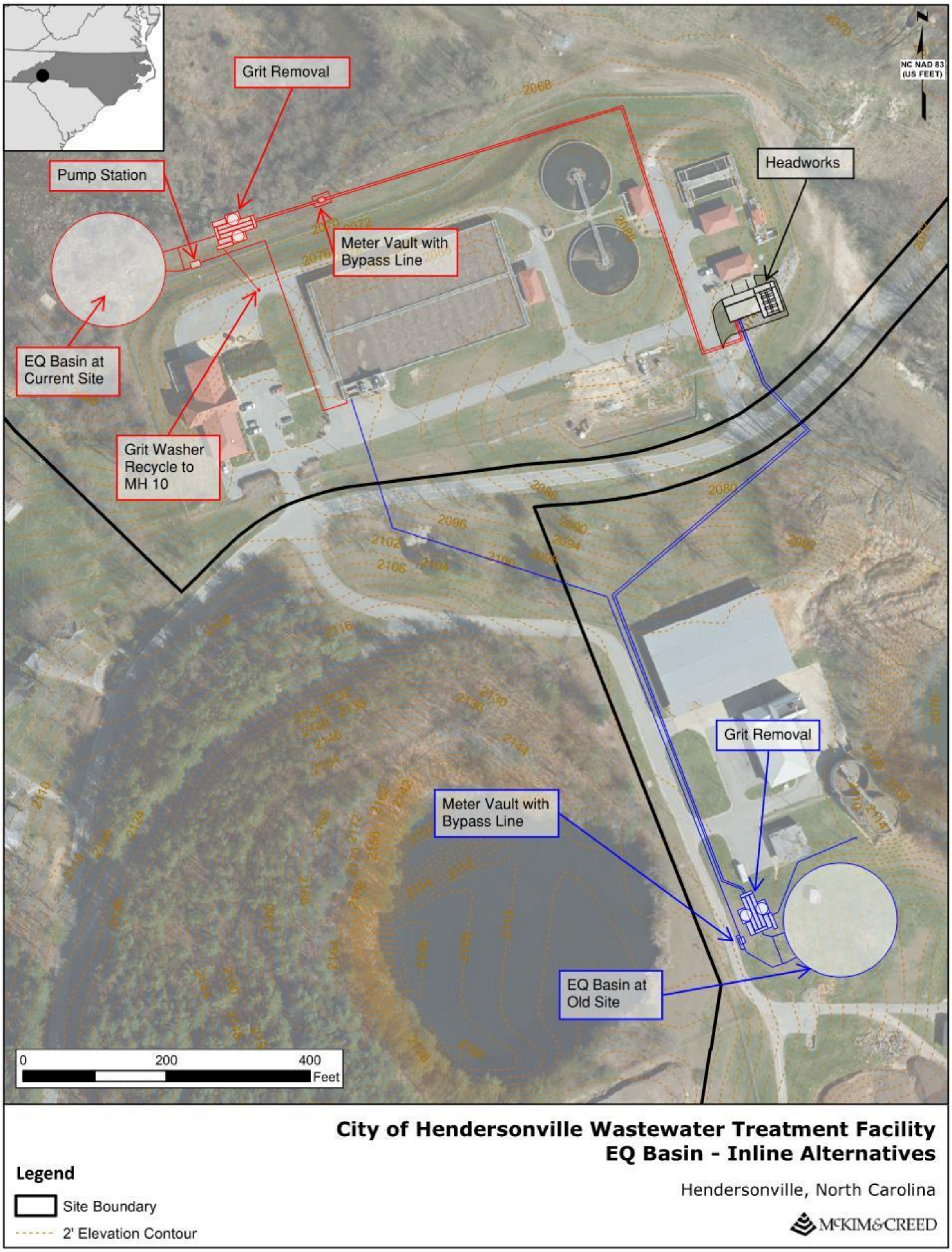
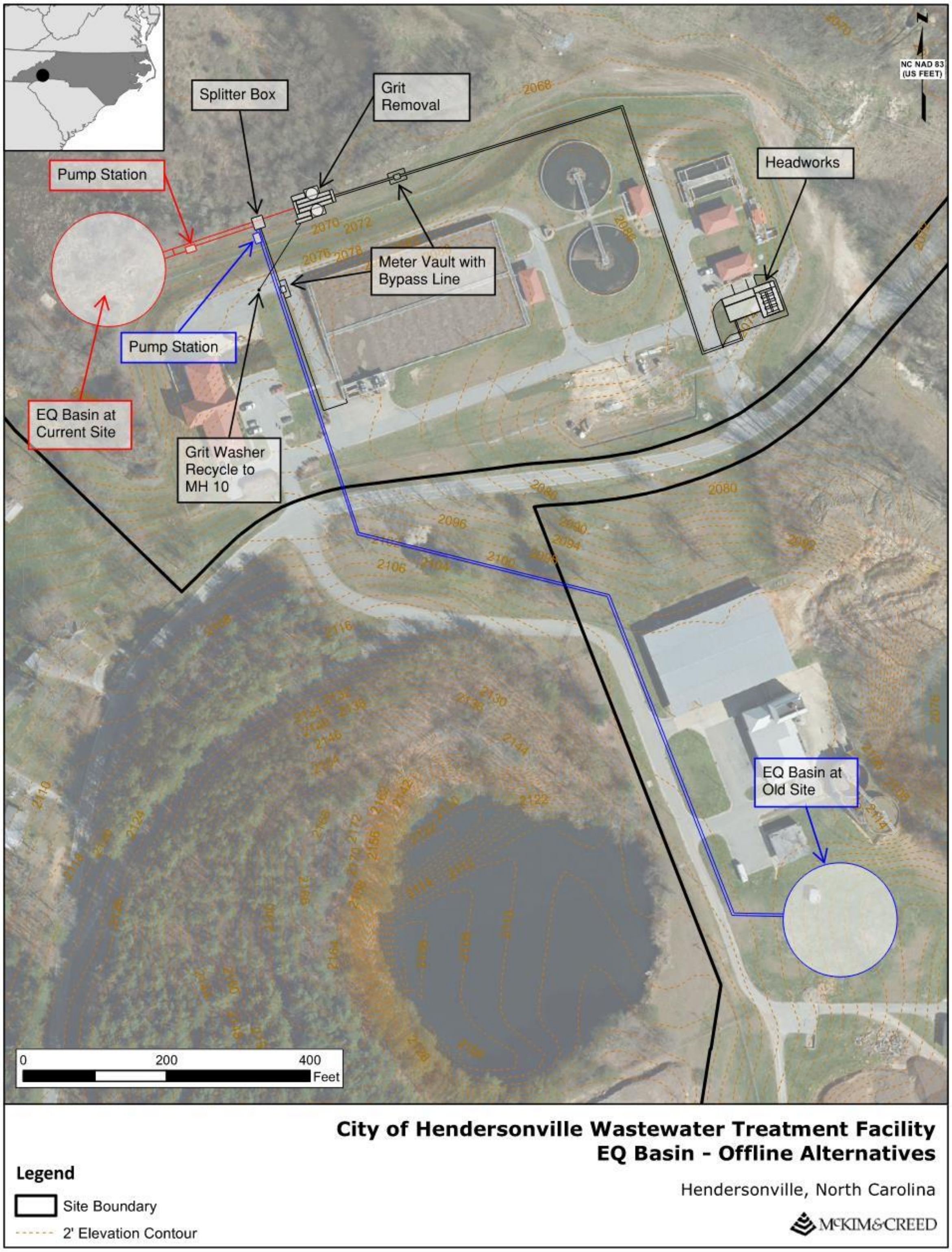


*Figure 6.6 – Offline EQ Alternatives*



### 6.3.1 Alternative 1: Inline EQ – Existing WWTF Site

As shown in red in **Figure 6.5** above, this alternative consists of a new inline EQ basin located on the existing WWTF site immediately north of the existing administration building. The 3.0 MG prestressed concrete EQ basin is proposed to be constructed with a finished floor elevation of approximately 2,076 feet, with a high water level (HWL) elevation of approximately 2,096 feet. This floor elevation is recommended to keep the proposed facilities located above the 100-year floodplain elevation. Under this alternative, flow from the influent pumping station is proposed to be directed to the new grit removal structure and EQ basin via two 24-inch diameter force mains. The proposed alignment for the new 24-inch diameter force mains is approximately 1,000 linear feet in length. Under this alternative, effluent from the EQ basin must be pumped to the aeration basins since gravity flow is not feasible based on site elevation. Per **Table 6.1**, an EQ effluent pump station with a firm capacity of 15 MGD in 2025 and 19.5 MGD in 2040 is proposed. VFD driven submersible low-head pumps are recommended for the EQ effluent pump station due to their low cost and ease of maintenance and replacement. The EQ effluent pump station is proposed to be a CIP concrete structure on pile foundations constructed adjacent to the EQ basin. EQ effluent will be pumped at a controlled rate through two 24-inch diameter force mains approximately 300 linear feet in length, each.

Significant site fill is required at this location to raise the existing grade up to an elevation of approximately 2,076 feet. It is assumed that all site fill required for the proposed location of Alternative 1 would be exported fill from other off-site locations. The proposed EQ basin and grit removal structures are assumed to be constructed on pile foundations in this location, similar to the existing process structures at the WWTF. The estimated volume of site fill required for construction at this location is approximately 19,000 CY, assuming a shrinking factor of 1.3 once fill is placed and compacted.

Pumping horsepower requirements were calculated for this alternative for both influent pumped to the EQ basin, and effluent pumped from the EQ basin. Theoretical pumping horsepower requirements were estimated at 2025 and 2040 design horizons for both AADF and peak flow conditions. The pumping horsepower requirements dictated the estimated pump sizing for each scenario, as well as a large majority of the expected energy usage. The total estimated firm pumping horsepower requirements, including both influent and effluent pumping, for each condition are summarized in **Table 6.4** below.

*Table 6.4 – Alternative 1 Pumping Horsepower Requirements*

| Condition | 2025 | 2040 |
|---|---|---|
| AADF | 67 hp | 95 hp |
| Peak Flow | 369 hp | 499 hp |

Capital costs were estimated for this alternative based on the recommended circular prestressed concrete tank, equipment costs for the EQ basin appurtenances recommended earlier in this document, structural costs for the EQ effluent pump station, site grading costs, piping costs, installation costs, and typical assumptions for design and indirect construction costs. The total opinion of probable project cost for Alternative 1 is presented below in **Table 6.5**. All estimated costs presented in this section are shown in September 2021 dollars. Detailed cost information is included at the end of this technical memorandum in **Appendix A**.

*Table 6.5 – Alternative 1 Estimated Opinion of Probable Project Cost*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $1,000,000 |
| 2 | Mechanical | $200,000 |
| 3 | Electrical | $200,000 |
| 4 | Instrumentation | $100,000 |
| 5 | Structural | $3,722,000 |
| 6 | Civil | $1,094,000 |
| 7 | Mobilization & Demobilization | $253,000 |
| 8 | Indirect Costs | $297,000 |
| 9 | General Conditions & Contractor Markup | $1,442,000 |
| 10 | 30% Contingency | $2,493,000 |
| 11 | Engineering, Legal, & Administration | $2,003,000 |
| | **Total Cost Opinion** | **$12,804,000** |

Annual operating and maintenance costs were estimated for 2025 and 2040 conditions to enable the calculation of the alternative's net present value (NPV). Annual operating and maintenance costs were estimated based on the energy usage required for pumping, aeration, and mixing, and based on a typical assumption for annual maintenance costs of the equipment. Annual energy usage for pumping was estimated based on the present year's AADF, which was 4.23 MGD for 2025, and 5.90 MGD for 2040. Annual energy usage for mixing and aeration of the EQ basin contents was estimated based on the assumption that the jet mixing pumps and the aeration blower would operate continuously. The estimated annual O&M costs for Alternative 1 are summarized below in **Table 6.6** and **Table 6.7**.

*Table 6.6 – Alternative 1 – Annual O&M Costs – 2025*

| Item | Annual Cost |
|---|---|
| Maintenance | $20,000 |
| Electricity | $105,000 |
| **TOTAL** | **$125,000** |

*Table 6.7 – Alternative 1 – Annual O&M Costs – 2040*

| Item | Annual Cost |
|---|---|
| Maintenance | $20,000 |
| Electricity | $116,000 |
| **TOTAL** | **$136,000** |

### 6.3.2 Alternative 2: Inline EQ – Old Plant Site

As shown in blue in **Figure 6.5**, Alternative 2 consists of a new inline EQ basin located on old plant site south of the former administration building. The 3.0 MG circular prestressed concrete EQ basin is recommended to be constructed with a finished floor elevation of approximately 2,095 feet and a HWL of 2,115 feet. Construction at this location allows for all effluent from the EQ basin to flow by gravity to the aeration basins on the existing WWTF site. Flow from the influent pumping station would be directed to a new grit removal structure adjacent to the new EQ basin via two 24-inch diameter force mains. The proposed alignment for the influent force mains is approximately 1,000 linear feet in length. Effluent from the EQ basin under this alternative will flow via gravity through one 36-inch diameter gravity line to the aeration basins. Effluent flow control is proposed to be accomplished using modulating plug valves or electronic or air-actuated control pinch valves based on feedback from an effluent electromagnetic flow meter. The 36-inch diameter effluent gravity line alignment is approximately 1,200 linear feet in length.

The two 24-inch diameter EQ influent force mains and the one 36-inch diameter EQ effluent gravity line will each be required to cross Balfour Road which is an NCDOT secondary route. Each crossing will require bore and jack installations inside a 48-inch diameter steel casing pipe for approximately 50 linear feet.

The site elevation at the old plant site immediately south of the former administration building ranges from approximately 2,105 to 2,095 feet. Approximately 5 to 10 feet of excavation is expected to be required in this location to construct the proposed EQ basin at a finished floor elevation of 2,095 feet. As a result, the circular prestressed concrete tank is proposed to be partially buried, which will slightly increase tank construction cost. Coordination with prestressed concrete tank manufacturers indicated that a minimum working area of approximately 15-feet would be required around the tank at the base of the excavation. Site work for the EQ basin at the old plant site is expected to require some rock excavation based on the old rock quarries in the immediate vicinity. Without any geotechnical information in this area, it was assumed that common earth excavation would be encountered up to an average depth of 4 feet, and rock excavation would be encountered at depths beyond 4 feet. Based on these assumptions approximately 11,700 CY of excavation is expected to be required, with 4,700 CY consisting of common earth excavation, and 7,000 CY consisting of rock excavation. Approximately 4,200 CY of excavated material was assumed to be backfilled and compacted around the EQ basin following construction. The remaining 7,500 CY of excavated material was assumed to be hauled off-site for disposal. If desired, excess excavated materials can be utilized to fill and level additional areas on the existing WWTF site or the old plant site to limit disposal costs related to haul-off. Any site fill within the 100-year floodplain at either location will require a floodplain development permit and potentially require a CLOMR or LOMR.

Pumping horsepower requirements were calculated for this alternative for the influent pumped to the EQ basin. There is no pumping energy requirement for effluent transferred from the EQ basin under this alternative since all effluent will flow by gravity to the downstream processes. Theoretical pumping horsepower requirements were estimated at 2025 and 2040 design horizons for both AADF and peak flow conditions. The pumping horsepower requirements dictated the estimated pump sizing for each scenario, as well as a large majority of the expected energy usage. The total estimated firm pumping horsepower requirements for each condition are summarized in **Table 6.8** below.

*Table 6.8 – Alternative 2 Pumping Horsepower Requirements*

| Condition | 2025 | 2040 |
|---|---|---|
| AADF | 72 hp | 102 hp |
| Peak Flow | 419 hp | 555 hp |

The total opinion of probable project cost for Alternative 2 is presented below in **Table 6.9**. The OPPC was developed utilizing the same process as Alternative 1. All estimated costs presented in this section are shown in September 2021 dollars. The estimated annual O&M costs for Alternative 2 are summarized in **Table 6.10** and **Table 6.11**.

*Table 6.9 – Alternative 2 Estimated Opinion of Probable Project Cost*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $778,000 |
| 2 | Mechanical | $156,000 |
| 3 | Electrical | $156,000 |
| 4 | Instrumentation | $78,000 |
| 5 | Structural | $1,493,000 |
| 6 | Civil | $1,965,000 |
| 7 | Demo | $65,000 |
| 8 | Mobilization & Demobilization | $188,000 |
| 9 | Indirect Costs | $221,000 |
| 10 | General Conditions & Contractor Markup | $1,071,000 |
| 11 | 30% Contingency | $1,852,000 |
| 12 | Engineering, Legal, & Administration | $1,488,000 |
| | **Total Cost Opinion** | **$9,511,000** |

*Table 6.10 – Alternative 2 – Annual O&M Costs – 2025*

| Item | Annual Cost |
|---|---|
| Maintenance | $16,000 |
| Electricity | $107,000 |
| **TOTAL** | **$123,000** |

*Table 6.11 – Alternative 2 – Annual O&M Costs – 2040*

| Item | Annual Cost |
|---|---|
| Maintenance | $16,000 |
| Electricity | $119,000 |
| **TOTAL** | **$135,000** |

### 6.3.3 Alternative 3: Offline EQ – Existing WWTF Site

Alternative 3, as shown in red in **Figure 6.6**, is nearly identical to Alternative 1 with the exception of the EQ facilities operating in an offline arrangement to only equalize wet weather flows exceeding the hydraulic capacity of the WWTF. Under this alternative, all influent wastewater will be pumped to grit removal and then to a new overflow structure to control the flow split between EQ storage and flow to downstream processes. Under normal operation, all wastewater from the overflow structure will be sent to the aeration basins and downstream processes for normal treatment. Under wet weather conditions, the City will have the capability to control the amount of flow sent to the aeration basins, and the amount of flow sent to the EQ basin for storage. Control of the flow split at the overflow structure can be accomplished using an electrically actuated weir gate on the EQ influent overflow weir. Control of the electrically actuated weir gate can be controlled automatically using feedback from the downstream flow meter measuring the flow rate sent to the aeration basins.

All flow leaving the overflow structure will be by gravity, regardless of whether it is directed to the aeration basins or to the EQ basin for storage. Due to site elevation at the existing WWTF, effluent from the EQ basin will require pumping to transfer it to the aeration basins under Alternative 3. EQ basin effluent would be pumped from a new EQ basin effluent pump station through a single 16-inch diameter force main with a total length of approximately 150 linear feet. Gravity flow from the overflow structure to the EQ basin will require a single 24-inch diameter pipe, also with a total length of approximately 150 linear feet. Gravity flow from the overflow structure to the aeration basins will require a single 36-inch diameter pipe with a total length of approximately 200 linear feet.

Site grading requirements for Alternative 3 are identical to Alternative 1. Pumping horsepower requirements were calculated for this alternative for all influent pumped to the overflow structure and for

EQ basin effluent pumping. Theoretical pumping horsepower requirements were estimated at 2025 and 2040 design horizons for both AADF and peak flow conditions. The pumping horsepower requirements dictated the required pump sizing for each scenario, as well as a large majority of the expected energy usage. The total estimated firm pumping horsepower requirements, including both influent and effluent pumping, for each condition are summarized in **Table 6.12** below.

*Table 6.12 – Alternative 3 Pumping Horsepower Requirements*

| Condition | 2025 | 2040 |
|---|---|---|
| AADF | 63 hp | 86 hp |
| Peak Flow | 325 hp | 432 hp |

The total opinion of probable project cost for Alternative 3 is summarized in **Table 6.13**. All estimated costs presented in this section are shown in September 2021 dollars.

*Table 6.13 – Alternative 3 Estimated Opinion of Probable Project Cost*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $916,000 |
| 2 | Mechanical | $184,000 |
| 3 | Electrical | $184,000 |
| 4 | Instrumentation | $92,000 |
| 5 | Structural | $3,822,000 |
| 6 | Civil | $1,016,000 |
| 7 | Mobilization & Demobilization | $249,000 |
| 8 | Indirect Costs | $292,000 |
| 9 | General Conditions & Contractor Markup | $1,420,000 |
| 10 | 30% Contingency | $2,453,000 |
| 11 | Engineering, Legal, & Administration | $1,971,000 |
| | **Total Cost Opinion** | **$12,599,000** |

Annual O&M costs were calculated for Alternative 3 based on an assumption of how frequently the offline flow EQ facilities would be used during an average year. This assumption was based on the average number of days per year that the WWTF influent exceeded the 4.8 MGD maximum month permitted capacity of the existing WWTF. The average number of days per year with influent flow exceeding 4.8 MGD was based on influent flow data from 2014 through 2019. Based on this data, there were 124 days out of a possible 2,191 total calendar days where influent flow was greater than or equal to 4.8 MGD. This equates to an annual percent chance of 5.66%, or 20.7 days per year on average. This average number of days per year is similar to the average number of days per year with precipitation exceeding 1.0 inches per day for the Asheville/Hendersonville area of 17 days (per NOAA climate data) over the same date range. As a result, it was assumed that offline flow EQ facilities would be operated at least 21 days per year to provide some level of flow equalization. The resulting annual O&M costs for Alternative 3 based on these assumptions are summarized in **Table 6.14** and **Table 6.15** below.

*Table 6.14 – Alternative 3 – Annual O&M Costs – 2025*

| Item | Annual Cost |
|---|---|
| Maintenance | $19,000 |
| Electricity | $26,000 |
| **TOTAL** | **$45,000** |

*Table 6.15 – Alternative 3 – Annual O&M Costs – 2040*

| Item | Annual Cost |
|---|---|
| Maintenance | $19,000 |
| Electricity | $35,000 |
| **TOTAL** | **$54,000** |

### 6.3.4 Alternative 4: Offline EQ – Old Plant Site

Alternative 4, as shown in blue in **Figure 6.6**, is nearly identical to Alternative 2 with the exception of the EQ facilities operating in an offline arrangement to only equalize wet weather flows exceeding the hydraulic capacity of the WWTF. Under this alternative, all influent wastewater will be pumped to grit removal and then to a new overflow structure on the existing WWTF site to control the flow split between EQ storage and flow to downstream processes. Under normal operation, all wastewater from the overflow structure will be sent to the aeration basins and downstream processes for normal treatment. Under wet weather conditions, the City will have the capability to control the amount of flow sent to the aeration basins, and the amount of flow sent to the EQ basin for storage. Control of the flow split at the overflow structure can be accomplished using an electrically actuated weir gate on the EQ influent overflow weir. Control of the electrically actuated weir gate can be controlled automatically using feedback from the downstream flow meter measuring the flow rate sent to the aeration basins.

Under Alternative 4, the new EQ basin will be located on the old plant site south of the former administration building, so all flow from the overflow structure to the EQ basin must be pumped through a 24-inch diameter force main approximately 1,250 linear feet in length. Gravity flow from the EQ basin effluent to the aeration basins will require a single 16-inch diameter pipe with a total length of approximately 1,250 linear feet. Gravity flow from the overflow structure to the aeration basins will require a single 36-inch diameter pipe with a total length of approximately 200 linear feet. The 24-inch diameter EQ influent force main and 16-inch diameter EQ effluent gravity line will both be required to cross Balfour Road which is an NCDOT secondary route. These road crossings will require bore and jack installations. A 48-inch diameter steel casing pipe was assumed to be required for the 24-inch diameter EQ influent force main, and a 36-inch diameter steel casing pipe was assumed to be required for the 16-

inch diameter EQ effluent gravity line. Each bore and jack installation was assumed to be approximately 50 linear feet in length.

Site grading requirements for Alternative 4 are nearly identical to Alternative 2, with the exception of site grading required on the existing WWTF site for the overflow structure and EQ influent pump station. Site fill on the existing WWTF site for the new overflow structure and EQ influent pump station will require approximately 2,500 CY. The estimated volume of fill required includes an assumed shrinkage factor of 1.3 to account for compaction of fill materials during placement.

Pumping horsepower requirements were calculated for this alternative for all influent pumped to the overflow structure and for EQ basin influent pumping. Theoretical pumping horsepower requirements were estimated at 2025 and 2040 design horizons for both AADF and peak flow conditions. The pumping horsepower requirements dictated the estimated pump sizing for each scenario, as well as a large majority of the expected energy usage. The total estimated firm pumping horsepower requirements for each condition are summarized in **Table 6.16** below.

*Table 6.16 – Alternative 4 Pumping Horsepower Requirements*

| Condition | 2025 | 2040 |
|---|---|---|
| AADF | 74 hp | 106 hp |
| Peak Flow | 356 hp | 472 hp |

The total opinion of probable project costs for Alternative 4 is summarized below in **Table 6.17**. Annual O&M costs were estimated for Alternative 4 using the same assumptions as those described for Alternative 3. The annual O&M costs for Alternative 4 are summarized below in **Table 6.18** and **Table 6.19**. All estimated costs presented in this section are shown in September 2021 dollars.

*Table 6.17 – Alternative 4 Estimated Opinion of Probable Project Cost*

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $988,000 |
| 2 | Mechanical | $198,000 |
| 3 | Electrical | $198,000 |
| 4 | Instrumentation | $99,000 |
| 5 | Structural | $1,660,000 |
| 6 | Civil | $2,004,000 |
| 7 | Demo | $65,000 |
| 8 | Mobilization & Demobilization | $209,000 |
| 9 | Indirect Costs | $246,000 |
| 10 | General Conditions & Contractor Markup | $1,192,000 |
| 11 | 30% Contingency | $2,058,000 |
| 12 | Engineering, Legal, & Administration | $1,654,000 |
| | **Total Cost Opinion** | **$10,571,000** |

*Table 6.18 – Alternative 4 – Annual O&M Costs – 2025*

| Item | Annual Cost |
|---|---|
| Maintenance | $20,000 |
| Electricity | $26,000 |
| **TOTAL** | **$46,000** |

*Table 6.19 – Alternative 4 – Annual O&M Costs – 2040*

| Item | Annual Cost |
|---|---|
| Maintenance | $20,000 |
| Electricity | $35,000 |
| **TOTAL** | **$55,000** |

## 6.4 Comparison of Alternatives and Recommendations

A summary of the capital costs, annual O&M costs, O&M NPV, and total NPV is provided in **Table 6.20** below. All estimated costs presented in this section are shown in September 2021 dollars. These estimated costs are recommended to be revisited and updated regularly to capture changes in market conditions prior to project conception to allow for budgets to be updated appropriately. As shown in the comparison below, Alternative 2: Inline EQ – Old Plant Site has the lowest total NPV, $97,000 less than the next lowest alternative, Alternative 4: Offline EQ – Old Plant Site. In terms of capital cost, Alternative 2 also has the lowest cost option, which is $1,060,000 less than Alternative 4. As expected, both inline EQ alternatives (Alternative 1 and Alternative 2) had significantly higher annual O&M costs due to the higher horsepower requirements for influent pumping under normal conditions. Capital costs for both alternatives on the existing WWTF site (Alternative 1 and Alternative 3) were approximately $2MM to $3MM more than capital costs for the alternatives located at the old plant site. This difference is directly attributable to the costs for site fill and pile foundations if the EQ facilities are constructed on the existing WWTF site.

*Table 6.20 – Cost Comparison of Flow EQ Alternatives*

| Alternative | Capital Costs | 2025 Annual O&M Cost | 2040 Annual O&M Cost | O&M NPV | Total NPV[1] |
|---|---|---|---|---|---|
| Alternative 1: Inline EQ – Existing WWTF Site | $12,804,000 | $125,000 | $136,000 | $1,294,000 | **$11,884,000** |
| Alternative 2: Inline EQ – Old Plant Site | $9,511,000 | $123,000 | $135,000 | $1,279,000 | **$9,145,000** |
| Alternative 3: Offline EQ – Existing WWTF Site | $12,599,000 | $45,000 | $54,000 | $489,000 | **$10,909,000** |
| Alternative 4: Offline EQ – Old Plant Site | $10,571,000 | $46,000 | $55,000 | $499,000 | **$9,242,000** |

*[1]Total NPV based on capital and O&M costs assuming the EQ facilities are constructed in 2025.*

It is important to note that while the offline EQ alternatives had significantly lower annual O&M costs, this is based on the assumed number of days that the offline EQ facilities would be utilized. Offline EQ facilities may be used more frequently than was assumed in this analysis, which would increase annual O&M costs and widen the NPV gap between offline EQ and inline EQ alternatives. It should also be noted that annual O&M cost savings related to inline EQ alternatives are not included in the table above due to their unpredictable nature. Inline EQ alternatives provide diurnal influent load equalization and reduce on-peak energy demands associated with air demands from the biological process. Finally, the City could mitigate some of the increased annual O&M costs under the inline EQ alternatives by cycling the EQ basin aeration blower and EQ basin jet mixing pumps on and off as needed to reduce energy usage. VFDs are recommended for the jet mixing pumps and EQ basin blowers to provide improved energy efficiency and reduced operating costs under all alternatives. The capital cost estimates presented above assumed VFDs are provided for the associated pumps and blowers, however their impacts on energy savings cannot be predicted reliably therefore no energy savings are assumed in the O&M cost estimates.

Based on the information presented in this section, it is recommended that the City proceed with Alternative 2, which consists of a new inline flow EQ facility located at the old plant site, south of Balfour Road. Alternative 2 is recommended for the following reasons:

- Lowest total net present value
- Provides wet weather flow equalization, diurnal flow equalization, and diurnal influent loading equalization
- Expected to improve overall treatment process control and result in fewer effluent limit violations
- Diurnal loading equalization is expected to reduce on-peak energy demands associated with air demands from the biological process
- Expected to improve secondary clarifier settling performance due to load equalization
- Expected to improve tertiary filter performance and reduce filter backwash frequency
- Expected to reduce peak UV disinfection dose requirements and associated peak energy demand

The proposed inline flow EQ facilities are recommended to be constructed concurrently with improvements to the influent pumping station, screening, and grit removal processes. Permitting requirements for the recommended improvements are expected to include:

- NPDES permit major modification due to the addition of a new unit process
- NPDES Authorization to Construct (ATC) Permit
- NCDEQ approval of erosion and sedimentation control (E&SC) plan (land disturbance >1 acre)
- NCDOT EA16.1 encroachment agreement for pipeline crossings of Balfour Road (SR 1508)

# 7. FLOW EQUALIZATION FACILITY DESIGN CRITERIA

## 7.1 Flow Rates

As noted above, an inline flow equalization basin is recommended for the City of Hendersonville's WWTF to provide wet weather flow equalization as well as diurnal flow and loading equalization. The projected flows to the WWTF, to the EQ basin, and from the EQ basin are summarized below in **Table 7.1**.

*Table 7.1 – Flow Equalization Basin Design Flow Rates*

| Design Year | 2-Year Storm Peak Influent Flow Rate (MGD) | Maximum EQ Basin Influent Flow Rate (MGD) | Maximum EQ Basin Effluent Flow Rate (MGD) | Average EQ Basin Effluent Flow Rate at AADF (MGD) |
|---|---|---|---|---|
| 2025 | 22.5 | 22.5 | 15 | 4.23 |
| 2040 | 28.3 | 28.3 | 19.5 | 5.9 |

## 7.2 Flow Equalization Basin

A partially buried circular prestressed concrete tank is proposed for in-line flow equalization storage. All influent flow to the WWTF is recommended to be screened, pumped to the old plant site for grit removal, and flow by gravity into the inline EQ basin. The inline EQ basin will provide both diurnal flow equalization as well as storm flow equalization. The circular prestressed concrete tank will have a sloped floor and a center drain sump with flow control valves and an effluent flow meter. A circumferential concrete walkway with handrails is recommended to be provided at the top of the tank to allow easy access for maintenance. EQ basin effluent will flow by gravity to the existing aeration basin influent channel for further treatment.

An EQ tank bypass is proposed to be provided following grit removal to allow the EQ basin to be removed from service for periodic maintenance and cleaning. Yard hydrants and water cannons are recommended to be installed at multiple locations around the perimeter of the top of the tank for periodic cleaning using plant non-potable water. Online monitoring and controls capabilities are recommended to be provided for EQ basin level and wastewater dissolved oxygen concentration. Design parameters for the proposed EQ basin are summarized in **Table 7.2** below.

*Table 7.2 – Flow Equalization Basin Design Parameters*

| Parameter | Units | Value |
|---|---|---|
| Number of Tanks | - | 1 |
| Diameter | ft | 160 |
| Design Sidewater Depth | ft | 20 |
| Design Freeboard | ft | 1 |
| Design Volume | MG | 3.0 |
| Overflow Capacity | MGD | 19.5 |
| Minimum Working Depth | ft | 3 |
| Design Working Volume | MG | 2.5 |
| Minimum Floor Slope | % | 2 |
| Level Measurement | - | Ultrasonic |
| DO Measurement | - | Luminescent DO Probe |

## 7.3 Jet Aeration and Mixing System

Jet aeration and mixing equipment is recommended to be provided within the proposed EQ basin to prevent solids deposition and limit odor generation. The jet aeration and mixing system will consist of three aeration and mixing headers within the EQ basin. Each aeration and mixing header is recommended to be constructed of 304 SS with air piping to extend up and over the EQ basin walls to connect to a common header, and 12-inch diameter pump suction and discharge piping to extend through the tank walls for connection to an external jet motive pump. Each aeration and mixing header will be provided with a dry-pit horizontal centrifugal pump mounted outside of the basin under a cover shelter. Recommended design parameters for the jet motive pumps are summarized in **Table 7.3**. Air will be provided by two 100 hp positive displacement blowers connected to a common air header to feed all three aeration and mixing headers. One of the two positive displacement blowers is to operate as a 100% in-place spare. Recommended design parameters for the positive displacement blowers are provided in **Table 7.4**. All common air header piping is recommended to be constructed of 304 SS.

Control of the jet aeration and mixing system is recommended to be based off of EQ basin level feedback and EQ basin wastewater DO concentration feedback. Automatic jet motive pump speed control is recommended to be provided based on feedback from the EQ basin level sensor. An operator adjustable pump speed setpoint may be established for multiple EQ basin depth bands. For example, the operator may choose to set each pump's speed to 100% when the water level in the EQ basin is between 16 and 20 feet deep, 75% between 12 and 16 feet deep, 50% between 8 to 12 feet deep, and 25% between 4 and 8 feet deep. Level feedback will also control pump on-off when the water level in the EQ basin has reached the minimum working level for the pumps. Automatic blower control is recommended to be provided based on feedback from the EQ basin DO probe. An operator adjustable DO setpoint may be established with a feedback loop to automatically increase or decrease blower speed based on DO concentration. A minimum DO concentration of 1.0 mg/L in the mixed basin contents is recommended at all times per the Ten State Standards. Operators may adjust the setpoint up or down as needed to balance energy usage with odor concerns.

*Table 7.3 – Flow Equalization Basin Jet Motive Pump Design Criteria*

| Parameter | Units | Value |
|---|---|---|
| Pump Type | - | Horizontal Centrifugal |
| Number of Units | - | 3 |
| Rated Capacity, each | gpm | 3,500 |
| Rated Head | ft | 21 |
| Rated Power, each | hp | 33 |
| Drive Type | - | Variable Frequency Drive |
| Pump Control | - | On-Off-Remote; automated operation based on EQ basin level and operator adjustable speed setpoints per basin level range |

*Table 7.4 – Flow Equalization Basin Jet Aeration Blower Design Criteria*

| Parameter | Units | Value |
|---|---|---|
| Blower Type | - | Positive Displacement |
| Number of Units | - | 2 (1 duty, 1 standby) |
| Rated Capacity, each | SCFM | 2,000 |
| Rated Power, each | hp | 100 |
| Drive Type | - | Variable Frequency Drive |
| Blower Control | - | On-Off-Remote; automated operation based on operator adjustable DO concentration setpoint |
| Dissolved Oxygen Setpoint | mg/L | 1.0 |

## 7.4 Flow Rate Control

Flow rate control measures will be provided to monitor and control the EQ basin effluent flow rate to the secondary treatment process. An electromagnetic flow meter will be provided on the 36-inch EQ basin effluent gravity line within a meter vault located near the EQ basin. The electromagnetic flow meter will provide feedback to adjust the opening of EQ effluent control valves to maintain effluent flows no greater than 15 MGD under the 2025 design conditions, and 19.5 MGD under the 2040 design conditions. The flow meter will be located downstream from the EQ basin bypass line to ensure all flow to the secondary treatment process is measured. This requires that the EQ effluent flow meter be sized to provide flow measurement up to the 2040 non-equalized peak flow rate of 28.3 MGD. A flow meter bypass line will be provided with manual, buried plug valves installed upstream and downstream of the electromagnetic flow meter to allow it to be removed for maintenance while still maintaining flow to the downstream processes. Flow rate control design parameters are summarized below in **Table 7.5**.

*Table 7.5 – Flow Rate Control Design Parameters*

| Parameter | Units | Value |
|---|---|---|
| Number of Flow Meters | - | 1 |
| Type | | Electromagnetic Flow Meter |
| Diameter | in | 30 |
| Peak Flow | MGD | 28.3 |
| Peak Velocity | ft/s | 8.9 |

## 7.5 EQ Effluent Valves

Effluent flow from the EQ basin is recommended to be controlled by modulating control valves based on feedback from the effluent electromagnetic flow meter. At least two modulating control valves are recommended to be provided on the EQ basin effluent line to provide redundancy. The control valves are recommended to be located within a valve vault adjacent to the EQ basin. Recommended design criteria for the EQ basing effluent control valves are summarized in Table 7.6.

*Table 7.6 – EQ Basin Effluent Control Valves Design Parameters*

| Parameter | Units | Value |
|---|---|---|
| Number of Control Valves | - | 2 |
| Type | - | Modulating Plug Valve |
| Diameter | in | 24 |
| Actuator Type | - | Electrically Actuated |
| End Connection | - | Flanged, ANSI Class 125 |
| Maximum Flow, each | MGD | 9.75 |

# APPENDIX A:
## SUPPORTING COST INFORMATION

## Total Net Present Value

| Alternative | Capital Costs | 2025 Annual O&M Cost | 2040 Annual O&M Cost | O&M Net Present Value | Total NPV |
|---|---|---|---|---|---|
| **Alternative 1: Inline EQ - Existing WWTF Site** | $12,804,000 | $125,000 | $136,000 | $1,294,000 | **$11,884,000** |
| **Alternative 2: Inline EQ - Old Plant Site** | $9,511,000 | $123,000 | $135,000 | $1,279,000 | **$9,145,000** |
| **Alternative 3: Offline EQ - Existing WWTF Site** | $12,599,000 | $45,000 | $54,000 | $489,000 | **$10,909,000** |
| **Alternative 4: Offline EQ - Old Plant Site** | $10,571,000 | $46,000 | $55,000 | $499,000 | **$9,242,000** |

| CAPEX Cost Estimate Components & Assumptions | | | | |
|---|---|---|---|---|
| **#** | **Category** | **Description** | **Assumption** | |
| 1 | Equipment | Includes major and anicillary equipment listed in budgetary proposals from vendors. | - | |
| | **SUBTOTAL A** | *= Equipment Budgetary Proposal Cost* | | |
| 2 | Mechanical Equipment Installation | | 20% | of Subtotal A. |
| 3 | Electrical Installation Costs | | 20% | of Subtotal A. |
| 4 | Instrumentation Installation Costs | | 10% | of Subtotal A. |
| 5 | Structural | Detailed calcs specific to the alternative. | | |
| 6 | Civil | Detailed calcs specific to the alternative. Includes site work, grading, subgrade preparation, etc. | | |
| 7 | Demo | Detailed calcs specific to the alternative. Includes demo of existing buildings, concrete slabs, pavements, underground utilities, etc. | | |
| 8 | Mobilization & Demobilization | | 4% | of Subtotal A + sum of items 2 through 7 |
| | **SUBTOTAL B** | *= Subtotal A + Sum of items 2 through 8* | | |
| 9 | Permits | | 1% | of Subtotal B. |
| 10 | Risk & Liability Insurance | | 1.5% | of Subtotal B. |
| 11 | Performance & Payment Bonds | | 2% | of Subtotal B. |
| | **SUBTOTAL C** | *= Subtotal B + sum of items 8 through 10* | | |
| 12 | General Conditions | | 6% | of Subtotal C. |
| 13 | Contractor's OH & P | OH = overhead, P = profit | 15% | of Subtotal C. |
| | **SUBTOTAL D** | *= Subtotal C + sum of items 11 through 12* | | |
| 14 | Contingency | | 30% | of Subtotal D. |
| | | **OPINION OF PROBABLE CONSTRUCTION COST (OPCC)** | *= Subtotal D + line 14.* | |
| 15 | Engineering, Legal, & Administration | | 25% | of Subtotal D minus sum of items 9 -11 |
| | | **TOTAL CAPITAL COST** | *= OPCC + Item 15* | |

## Alternative 1: Inline EQ - Existing WWTF Site

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $1,000,000 |
| 2 | Mechanical | $200,000 |
| 3 | Electrical | $200,000 |
| 4 | Instrumentation | $100,000 |
| 5 | Structural | $3,722,000 |
| 6 | Civil | $1,094,000 |
| 7 | Mobilization & Demobilization | $253,000 |
| 8 | Indirect Costs | $297,000 |
| 9 | General Conditions & Contractor Markup | $1,442,000 |
| 10 | 30% Contingency | $2,493,000 |
| 11 | Engineering, Legal, & Administration | $2,003,000 |
| | **Total Capital Cost** | **$12,804,000** |

## Alternative 2: Inline EQ - Old Plant Site

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $778,000 |
| 2 | Mechanical | $156,000 |
| 3 | Electrical | $156,000 |
| 4 | Instrumentation | $78,000 |
| 5 | Structural | $1,493,000 |
| 6 | Civil | $1,965,000 |
| 7 | Demo | $65,000 |
| 8 | Mobilization & Demobilization | $188,000 |
| 9 | Indirect Costs | $221,000 |
| 10 | General Conditions & Contractor Markup | $1,071,000 |
| 11 | 30% Contingency | $1,852,000 |
| 12 | Engineering, Legal, & Administration | $1,488,000 |
| | **Total Capital Cost** | **$9,511,000** |

## Alternative 3: Offline EQ - Existing WWTF Site

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $916,000 |
| 2 | Mechanical | $184,000 |
| 3 | Electrical | $184,000 |
| 4 | Instrumentation | $92,000 |
| 5 | Structural | $3,822,000 |
| 6 | Civil | $1,016,000 |
| 7 | Mobilization & Demobilization | $249,000 |
| 8 | Indirect Costs | $292,000 |
| 9 | General Conditions & Contractor Markup | $1,420,000 |
| 10 | 30% Contingency | $2,453,000 |
| 11 | Engineering, Legal, & Administration | $1,971,000 |
| | **Total Capital Cost** | **$12,599,000** |

## Alternative 4: Offline EQ - Old Plant Site

| Item | Description | Cost ($) |
|---|---|---|
| 1 | Equipment | $988,000 |
| 2 | Mechanical | $198,000 |
| 3 | Electrical | $198,000 |
| 4 | Instrumentation | $99,000 |
| 5 | Structural | $1,660,000 |
| 6 | Civil | $2,004,000 |
| 7 | Demo | $65,000 |
| 8 | Mobilization & Demobilization | $209,000 |
| 9 | Indirect Costs | $246,000 |
| 10 | General Conditions & Contractor Markup | $1,192,000 |
| 11 | 30% Contingency | $2,058,000 |
| 12 | Engineering, Legal, & Administration | $1,654,000 |
| | **Total Capital Cost** | **$10,571,000** |

| OPEX Cost Estimate Components & Assumptions | | | | |
|---|---|---|---|---|
| # | Category | Description | Assumption | |
| 1 | Analysis Period | Assumes that the Net Present Worth (NPW) Analysis Period is the same for all alternatives. | 20 | years |
| 2 | Design Life | Entails the assumptions that a) all system have the same useful life, and b) the project is constructed at one time. | 20 | years |
| 3 | Net Present Value Discount Rate | | 4% | |
| 4 | Annual Inflation Rate | | 3% | |
| 5 | 2021 Labor Cost | The cost of operator labor associated with the proposed alternative (including benefits) | $ 25.00 | / hour |
| 6 | 2040 Labor Cost | The cost of operator labor associated with the proposed alternative (including benefits) assuming inflation rate of 3% | $ 45.15 | / hour / operator |
| 7 | Maintenance Cost | | 2% | of capital equipment cost |
| 8 | Electricity Costs | | $0.06 | / kWhr |

## 2021 Annual O&M Costs

| Alternative | Item | Annual Cost |
|---|---|---|
| Alternative 1: Inline EQ - Existing WWTF Site | Maintenance | $20,000 |
| | Electricity | $105,000 |
| | **TOTAL** | **$125,000** |
| Alternative 2: Inline EQ - Old Plant Site | Maintenance | $16,000 |
| | Electricity | $107,000 |
| | **TOTAL** | **$123,000** |
| Alternative 3: Offline EQ - Existing WWTF Site | Maintenance | $19,000 |
| | Electricity | $26,000 |
| | **TOTAL** | **$45,000** |
| Alternative 4: Offline EQ - Old Plant Site | Maintenance | $20,000 |
| | Electricity | $26,000 |
| | **TOTAL** | **$46,000** |

## 2040 Annual O&M Costs

| Alternative | Item | Annual Cost |
|---|---|---|
| Alternative 1: Inline EQ - Existing WWTF Site | Maintenance | $20,000 |
| | Electricity | $116,000 |
| | **TOTAL** | **$136,000** |
| Alternative 2: Inline EQ - Old Plant Site | Maintenance | $16,000 |
| | Electricity | $119,000 |
| | **TOTAL** | **$135,000** |
| Alternative 3: Offline EQ - Existing WWTF Site | Maintenance | $19,000 |
| | Electricity | $35,000 |
| | **TOTAL** | **$54,000** |
| Alternative 4: Offline EQ - Old Plant Site | Maintenance | $20,000 |
| | Electricity | $35,000 |
| | **TOTAL** | **$55,000** |

## Opinion of Probable Project Costs

**Project:** WWTF Master Plan
**Project Number:** 06496-0009
**Prepared By:** McKim & Creed, Inc.
**Date:** September 2021
**Checked By:** ZMT
**Subject:** **Alternative 1: Inline EQ - Existing WWTF Site**

| ITEM | DESCRIPTION | QUANTITY | UNIT | UNIT COST | EXTENSION | ITEM SUB TOTAL |
|---|---|---|---|---|---|---|
| | | | | | **Net Present Value of Capital and O&M Costs** | **$11,884,000** |
| **1** | **Equipment** | | | | **$1,000,000** | |
| | Aeration and Mixing | 1 | LS | $600,000 | $600,000 | |
| | Electromagnetic Flow Meters | 1 | LS | $58,000 | $58,000 | |
| | Flow Meter Vaults | 2 | EA | $60,000 | $120,000 | |
| | Pumps | 3 | EA | $49,000 | $147,000 | |
| | Variable Frequency Drives (VFD) | 3 | EA | $25,000 | $75,000 | |
| | | | | | **Subtotal A:** | **$1,000,000** |
| **2** | **Installation Costs** | | | | **$5,569,000** | |
| | Mechanical Equipment Installation | 1 | LS | 20.0% | $200,000 | |
| | Electrical Installation Costs | 1 | LS | 20.0% | $200,000 | |
| | Instrumentation Installation Costs | 1 | LS | 10.0% | $100,000 | |
| | Structural | 1 | LS | $3,722,000 | $3,722,000 | |
| | Civil | 1 | LS | $1,094,000 | $1,094,000 | |
| | Mobilization & Demobilization | 1 | LS | 4.0% | $253,000 | |
| | | | | | **Subtotal B:** | **$6,569,000** |
| **3** | **Indirect Costs** | | | | **$297,000** | |
| | Permits | 1 | LS | 1.0% | $66,000 | |
| | Risk & Liability Insurance | 1 | LS | 1.5% | $99,000 | |
| | Performance & Payment Bonds | 1 | LS | 2.0% | $132,000 | |
| | | | | | **Subtotal C:** | **$6,866,000** |
| **4** | **General Conditions & Contractor Markup** | | | | **$1,442,000** | |
| | General Conditions | 1 | LS | 6.0% | $412,000 | |
| | Contractor's OH & P | 1 | LS | 15.0% | $1,030,000 | |
| | | | | | **Subtotal D:** | **$8,308,000** |
| **5** | **Contingency** | **1** | **LS** | **30%** | **$2,493,000** | |
| | | | | | **Opinion of Probable Construction Cost:** | **$10,801,000** |
| **6** | **Engineering, Legal, and Administration** | **1** | **LS** | **25.0%** | **$2,003,000** | |
| | | | | | **Opinion of Probable Project Cost:** | **$12,804,000** |
| **7** | **2025 O&M Costs** | | | | **$125,000** | |
| | Maintenance | 1 | LS | 2.0% | $20,000 | |
| | Electricity | 1,734,930.00 | kWhr | $0.06 | $105,000 | |
| **8** | **2040 O&M Costs** | | | | **$136,000** | |
| | Maintenance | 1 | LS | 2.0% | $20,000 | |
| | Electricity | 1,921,328 | kWhr | $0.06 | $116,000 | |
| **9** | **Net Present Value of Capital and O&M Costs** | | | | | **$11,884,000** |

# Opinion of Probable Project Costs

| | |
|---|---|
| **Project:** | WWTF Master Plan |
| **Project Number:** | 06496-0009 |
| **Prepared By:** | McKim & Creed, Inc. |
| **Date:** | September 2021 |
| **Checked By:** | ZMT |
| **Subject:** | **Alternative 2: Inline EQ - Old Plant Site** |

| ITEM | DESCRIPTION | QUANTITY | UNIT | UNIT COST | EXTENSION | ITEM SUB TOTAL |
|---|---|---|---|---|---|---|
| | | | | | **Net Present Value of Capital and O&M Costs** | **$9,145,000** |
| **1** | **Equipment** | | | | **$778,000** | |
| | Aeration and Mixing | 1 | LS | $600,000 | $600,000 | |
| | Electromagnetic Flow Meters | 1 | LS | $58,000 | $58,000 | |
| | Flow Meter Vaults | 2 | EA | $60,000 | $120,000 | |
| | | | | | **Subtotal A:** | **$778,000** |
| **2** | **Installation Costs** | | | | **$4,101,000** | |
| | Mechanical Equipment Installation | 1 | LS | 20.0% | $156,000 | |
| | Electrical Installation Costs | 1 | LS | 20.0% | $156,000 | |
| | Instrumentation Installation Costs | 1 | LS | 10.0% | $78,000 | |
| | Structural | 1 | LS | $1,493,000 | $1,493,000 | |
| | Civil | 1 | LS | $1,965,000 | $1,965,000 | |
| | Demo | 1 | LS | $65,000 | $65,000 | |
| | Mobilization & Demobilization | 1 | LS | 4.0% | $188,000 | |
| | | | | | **Subtotal B:** | **$4,879,000** |
| **3** | **Indirect Costs** | | | | **$221,000** | |
| | Permits | 1 | LS | 1.0% | $49,000 | |
| | Risk & Liability Insurance | 1 | LS | 1.5% | $74,000 | |
| | Performance & Payment Bonds | 1 | LS | 2.0% | $98,000 | |
| | | | | | **Subtotal C:** | **$5,100,000** |
| **4** | **General Conditions & Contractor Markup** | | | | **$1,071,000** | |
| | General Conditions | 1 | LS | 6.0% | $306,000 | |
| | Contractor's OH & P | 1 | LS | 15.0% | $765,000 | |
| | | | | | **Subtotal D:** | **$6,171,000** |
| **5** | **Contingency** | **1** | **LS** | **30%** | **$1,852,000** | |
| | | | | | **Opinion of Probable Construction Cost:** | **$8,023,000** |
| **6** | **Engineering, Legal, and Administration** | **1** | **LS** | **25.0%** | **$1,488,000** | |
| | | | | | **Opinion of Probable Project Cost:** | **$9,511,000** |
| **7** | **2025 O&M Costs** | | | | **$123,000** | |
| | Maintenance | 1 | LS | 2.0% | $16,000 | |
| | Electricity | 1,772,265.87 | kWhr | $0.06 | $107,000 | |
| **8** | **2040 O&M Costs** | | | | **$135,000** | |
| | Maintenance | 1 | LS | 2.0% | $16,000 | |
| | Electricity | 1,968,302 | kWhr | $0.06 | $119,000 | |
| **9** | **Net Present Value of Capital and O&M Costs** | | | | | **$9,145,000** |

## Opinion of Probable Project Costs

| | |
|---|---|
| **Project:** | WWTF Master Plan |
| **Project Number:** | 06496-0009 |
| **Prepared By:** | McKim & Creed, Inc. |
| **Date:** | September 2021 |
| **Checked By:** | ZMT |
| **Subject:** | **Alternative 3: Offline EQ - Existing WWTF Site** |

| ITEM | DESCRIPTION | QUANTITY | UNIT | UNIT COST | EXTENSION | ITEM SUB TOTAL |
|---|---|---|---|---|---|---|
| | | | | | **Net Present Value of Capital and O&M Costs** | **$10,909,000** |
| **1** | **Equipment** | | | | **$916,000** | |
| | Aeration and Mixing | 1 | LS | $600,000 | $600,000 | |
| | Electromagnetic Flow Meters | 1 | LS | $58,000 | $58,000 | |
| | Flow Meter Vaults | 2 | EA | $60,000 | $120,000 | |
| | Pumps | 3 | EA | $26,000 | $78,000 | |
| | Variable Frequency Drive (VFD) | 3 | EA | $20,000 | $60,000 | |
| | | | | | **Subtotal A:** | **$916,000** |
| **2** | **Installation Costs** | | | | **$5,547,000** | |
| | Mechanical Equipment Installation | 1 | LS | 20.0% | $184,000 | |
| | Electrical Installation Costs | 1 | LS | 20.0% | $184,000 | |
| | Instrumentation Installation Costs | 1 | LS | 10.0% | $92,000 | |
| | Structural | 1 | LS | $3,822,000 | $3,822,000 | |
| | Civil | 1 | LS | $1,016,000 | $1,016,000 | |
| | Mobilization & Demobilization | 1 | LS | 4.0% | $249,000 | |
| | | | | | **Subtotal B:** | **$6,463,000** |
| **3** | **Indirect Costs** | | | | **$292,000** | |
| | Permits | 1 | LS | 1.0% | $65,000 | |
| | Risk & Liability Insurance | 1 | LS | 1.5% | $97,000 | |
| | Performance & Payment Bonds | 1 | LS | 2.0% | $130,000 | |
| | | | | | **Subtotal C:** | **$6,755,000** |
| **4** | **General Conditions & Contractor Markup** | | | | **$1,420,000** | |
| | General Conditions | 1 | LS | 6.0% | $406,000 | |
| | Contractor's OH & P | 1 | LS | 15.0% | $1,014,000 | |
| | | | | | **Subtotal D:** | **$8,175,000** |
| **5** | **Contingency** | **1** | **LS** | **30%** | **$2,453,000** | |
| | | | | | **Opinion of Probable Construction Cost:** | **$10,628,000** |
| **6** | **Engineering, Legal, and Administration** | **1** | **LS** | **25.0%** | **$1,971,000** | |
| | | | | | **Opinion of Probable Project Cost:** | **$12,599,000** |
| **7** | **2025 O&M Costs** | | | | **$45,000** | |
| | Maintenance | 1 | LS | 2.0% | $19,000 | |
| | Electricity | 427,248.99 | kWhr | $0.06 | $26,000 | |
| **8** | **2040 O&M Costs** | | | | **$54,000** | |
| | Maintenance | 1 | LS | 2.0% | $19,000 | |
| | Electricity | 573,530 | kWhr | $0.06 | $35,000 | |
| **9** | **Net Present Value of Capital and O&M Costs** | | | | | **$10,909,000** |

# Opinion of Probable Project Costs

**Project:** WWTF Master Plan
**Project Number:** 06496-0009
**Prepared By:** McKim & Creed, Inc.
**Date:** September 2021
**Checked By:** ZMT
**Subject:** **Alternative 4: Offline EQ - Old Plant Site**

| ITEM | DESCRIPTION | QUANTITY | UNIT | UNIT COST | EXTENSION | ITEM SUB TOTAL |
|---|---|---|---|---|---|---|
| | | | | | **Net Present Value of Capital and O&M Costs** | **$9,242,000** |
| 1 | **Equipment** | | | | **$988,000** | |
| | Aeration and Mixing | 1 | LS | $600,000 | $600,000 | |
| | Electromagnetic Flow Meters | 1 | LS | $58,000 | $58,000 | |
| | Flow Meter Vaults | 2 | EA | $60,000 | $120,000 | |
| | Pumps | 3 | EA | $40,000 | $120,000 | |
| | Variable Frequency Drive (VFD) | 3 | EA | $30,000 | $90,000 | |
| | | | | | **Subtotal A:** | **$988,000** |
| 2 | **Installation Costs** | | | | **$4,433,000** | |
| | Mechanical Equipment Installation | 1 | LS | 20.0% | $198,000 | |
| | Electrical Installation Costs | 1 | LS | 20.0% | $198,000 | |
| | Instrumentation Installation Costs | 1 | LS | 10.0% | $99,000 | |
| | Structural | 1 | LS | $1,660,000 | $1,660,000 | |
| | Civil | 1 | LS | $2,004,000 | $2,004,000 | |
| | Demo | 1 | LS | $65,000 | $65,000 | |
| | Mobilization & Demobilization | 1 | LS | 4.0% | $209,000 | |
| | | | | | **Subtotal B:** | **$5,421,000** |
| 3 | **Indirect Costs** | | | | **$246,000** | |
| | Permits | 1 | LS | 1.0% | $55,000 | |
| | Risk & Liability Insurance | 1 | LS | 1.5% | $82,000 | |
| | Performance & Payment Bonds | 1 | LS | 2.0% | $109,000 | |
| | | | | | **Subtotal C:** | **$5,667,000** |
| 4 | **General Conditions & Contractor Markup** | | | | **$1,192,000** | |
| | General Conditions | 1 | LS | 6.0% | $341,000 | |
| | Contractor's OH & P | 1 | LS | 15.0% | $851,000 | |
| | | | | | **Subtotal D:** | **$6,859,000** |
| 5 | **Contingency** | **1** | **LS** | **30%** | **$2,058,000** | |
| | | | | | **Opinion of Probable Construction Cost:** | **$8,917,000** |
| 6 | **Engineering, Legal, and Administration** | **1** | **LS** | **25.0%** | **$1,654,000** | |
| | | | | | **Opinion of Probable Project Cost:** | **$10,571,000** |
| 7 | **2025 O&M Costs** | | | | **$46,000** | |
| | Maintenance | 1 | LS | 2.0% | $20,000 | |
| | Electricity | 431,132.72 | kWhr | $0.06 | $26,000 | |
| 8 | **2040 O&M Costs** | | | | **$55,000** | |
| | Maintenance | 1 | LS | 2.0% | $20,000 | |
| | Electricity | 581,034 | kWhr | $0.06 | $35,000 | |
| 9 | **Net Present Value of Capital and O&M Costs** | | | | | **$9,242,000** |