City of Northglenn
11701 Community Center Drive
Northglenn, Colorado 80233

CITY OF NORTHGLENN

Request for Proposals
Section 36 Sub Area Plan
[Proposal No. 2023-034]

ADDENDUM NO. 1
DATED: January 17, 2024

TO: PROSPECTIVE BIDDERS

The following adds to, supplements, amends or clarifies by way of explanation, portions of the Contract Documents, Specifications, and Drawings for the above named project.

NOTE: It will be the responsibility of the Bidder to acknowledge receipt of Addenda on the Bid Form as part of his/her submitted proposal. Failure to do so may be grounds for the City to reject the proposal.

Contract Award Date Change

To accommodate changes in the City Council meeting schedule the contract award date is moved to March 18, 2024

Attachments

Attached to this addendum you will find:

- Attachment A – RFP Question and Response Form
- Attachment B – Northglenn Water Master Plan
    - o This is to provide further information related to water infrastructure in Northglenn.
- Attachment C – Wastewater Treatment Plant Master Plan
    - o This is to provide further information related to wastewater infrastructure in Northglenn.

City of Northglenn
Sara Dusenberry
Senior Planner

ALL ITEMS IN CONFLICT WITH THIS ADDENDUM ARE HEREBY DELETED.

END OF ADDENDUM NO. 1

RFP Questions and Responses

1. Are we required to use EPS for the market analysis?
    a. No, you may choose to work with them for the market analysis. Their work with the city on the fiscal analysis does not preclude them from working with a firm on the market analysis.
2. Is the site rail served?
    a. No, RTD purchased the rail for a future expansion of the N-Line.
3. What is the impact of the oil and gas site on the surrounding areas?
    a. The site is operated by Kerr McGee. Their approved permit indicated a 30-year pumping timeframe.
4. With two similar RFPS out does the city desire to have separate consultants?
    a. No, the city is open to hiring the same consultant for both projects.

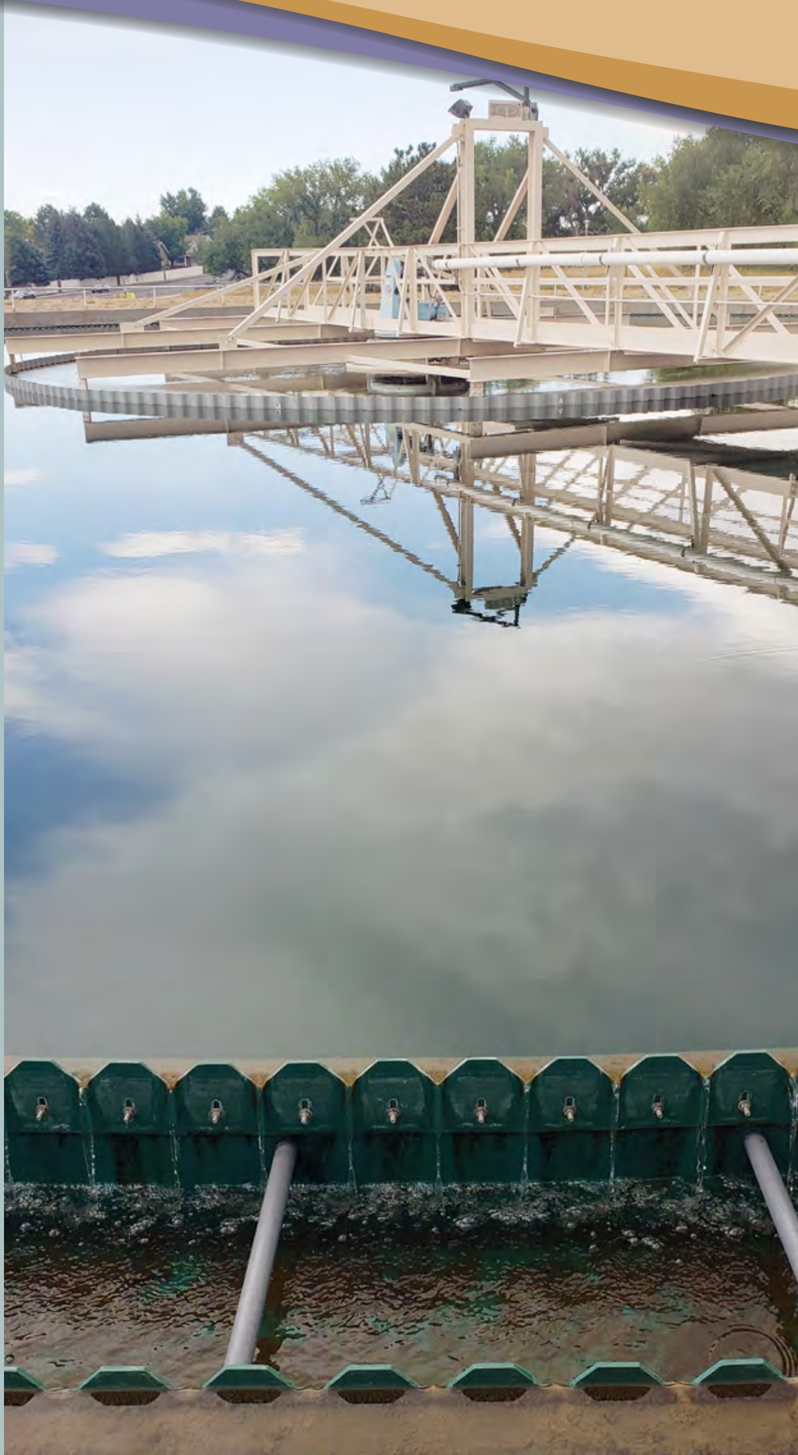
Attachment B

FINAL REPORT

CONSULTING ENGINEERS

City of Northglenn
Water Treatment Plant Master Plan Update

August 2020



Prepared For:

City of Northglenn

WATER TREATMENT PLANT
MASTER PLAN UPDATE

FOR THE

CITY OF NORTHGLENN

JVA, Inc.
1319 Spruce Street
phone: 303-444-1951
Boulder, CO 80302
fax: 303-444-1957

JVA Project No. 1061e

AUGUST 2020

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EXECUTIVE SUMMARY

INTRODUCTION

The purpose of the City of Northglenn (City) Water Treatment Plant Master Plan Update is to develop a comprehensive planning document providing guidance for the City's water treatment systems to continue to reliably serve high quality drinking water to the existing and future service area. This Master Plan should be viewed as a dynamic working document, reviewed annually, and updated as conditions in the City's service area change. The capital improvement plan (CIP) will assist the City in prioritizing projects and developing annual budgets. Recommendations identified in this Master Plan should be considered as conceptual only. Additional details and potential alternatives should be further investigated and analyzed in the preliminary design engineering phase of each project. The Master Plan includes historical water production and demands, water demand projections, an inventory of existing facilities, a capital improvements plan, and information on project financing.

PLANNING CONDITIONS

The City provides domestic water services primarily to residential customers, and a few commercial customers, within its service area, which encompasses approximately 4,800 acres. The City's current population is approximately 38,694 people. Since 2010, the average annual water tap growth for the City has been approximately 1.0 percent. Buildout is expected to occur within the next 20 years with a total of approximately 11,650 taps.

The City's WTP raw water and finished water production data from 2015 to 2019 was used to estimate historical water demand. The historical yearly average and maximum month daily average water production is 3.92 and 7.01 MGD, respectively, with a peak day flow of 8.32 MGD. Using a 0.5 percent growth rate, future water production rates were estimated for a 20-year period. The future average and peak day water production rates are 4.4 and 9.7 MGD, respectively.

WATER QUALITY AND REGULATORY REQUIREMENTS

The existing water supply sources for the City consists of surface water from the Clear Creek watershed. Water is collected through the Berthoud Pass Ditch, the Croke Canal, and the Church Ditch and stored in Standley Lake. The City shares the water stored in Standley Lake with other cities including Thornton and Westminster. From Standley Lake, the City's water is conveyed to the Terminal Reservoir (TR) prior to treatment and distribution.

Primary Drinking Water Standards

Primary drinking water standards include enforceable maximum contaminant levels (MCLs) and not enforceable maximum contaminant level goals (MCLGs). Primary contaminants are defined in the Colorado Primary Drinking Water Regulations (CPDWR) along with their respected limits. The City must meet all MCLs to maintain compliance with the CPDWR. The contaminants are divided into inorganics, volatile & synthetic organic compounds, radionuclides, disinfectants & byproducts, and lead & copper.

The inorganics group consists of elemental metals and nitrogen containing compounds. Finished water sampled for inorganics in 2012 and 2018 were all below the MCLs. Volatile and synthetic organic compounds have not historically been found in the City's finished drinking water, and so were not further investigated. Radionuclides are unstable forms of elements that can occur in natural or man-made deposits. The City's finished water was tested for radionuclides in 2012. All results were far below MCLs.

Disinfection is required for inactivating viruses, bacteria, and protozoa. The CPDWR sets a maximum residual chlorine level of 4.0 mg/L to protect consumers from drinking harmful amounts of disinfectants and to reduce production of disinfection by products (DBPs). The two major groups of DBPs are Total Trihalomethanes (TTHMs) and Haloacetic Acids (HAA5s). Historical data indicates that DBP levels are well below the MCL.

The CPDWR sets action levels for lead and copper concentrations for distribution systems. If concentrations exceed the action level limit, the City would be required to comply with additional requirements, which may include public education, corrosion control treatment, additional sampling, source water treatment, and/or lead service line replacement. Historical lead and copper levels from 2014 through the 2018 indicate that all test results are below the action level.

Secondary Drinking Water Standards

Secondary drinking water contaminants primarily affect the aesthetic qualities (taste and odor) relating to the public's acceptance of drinking water. The CPDWR defines secondary maximum contaminant levels (SMCLs), but they are not enforceable. They are intended to represent reasonable goals to reduce public health implications and water aesthetic degradation. The City's most recent raw water test results indicate no excursions except for manganese. The SMCL for manganese is 0.05 mg/L and in 2019 the average manganese level was 0.06 mg/L with a peak of 0.4 mg/L.

Future Considerations

The City is currently investigating the water quality of the Laramie-Fox Hills (LFH) aquifer for ASR. The LFH aquifer could provide approximately 2,000 acre-feet (AF) of storage to help mitigate drought conditions. The LFH water quality is poor and changing sources and blending of sources may result in unpredictable water treatment challenges. Pretreatment of the LFH aquifer water should be considered if ASR is planned for implementation.

Within the next five years, CDPHE anticipates modifications to the lead and copper rule (LCR). The LCR was first introduced in 1991 by the EPA to control lead and copper in drinking water to reduce public health impacts associated with these constituents. The EPA has considered regulatory options to further improve the existing rule, including lead service line replacement, improving optimal corrosion control treatment requirements, consideration of a health-based benchmark, the potential role of point-of-use filters, clarifications or strengthening of tap sampling requirements, increased transparency, and public education requirements. However, the most significant upcoming change to the LCR will be the separation of the lead and copper rule into two separate regulations. These changes are not expected to affect the City, since there are no recorded lead or copper violations.

Per- and polyfluoroalkyl substances (PFAS) are a large group of synthetic fluorinated organic chemicals that are soluble, mobile, and recalcitrant to chemical and biological processes. The two most dominant groups of PFAS consist of perfluorooctanyl sulfonate (PFOS) and perfluorooctanoic acid (PFOA). Elevated exposure to PFAS compounds (primarily by way of ingestion of drinking water) have been associated with developmental effects during pregnancy such as low infant birth weights and skeletal variations, effects on the immune system such as changes in antibody production and immunity, liver effects including tissue damage, cancer, and thyroid hormone disruption. In May 2016, EPA established drinking water health advisories of 70 parts per trillion (ppt) (0.07 micrograms per liter (µg/L)) for the combined concentrations of PFOS and PFOA. Above these levels, EPA recommends drinking water systems take steps to assess contamination, inform consumers, and limit exposure. Although the EPA has not issued an MCL for drinking water for PFOS and PFOA, several states have established drinking water and groundwater guidelines. Colorado has yet to establish these guidelines, however, the State has embarked on a sampling project in which utilities may volunteer to have their water sampled. The City has participated in the State sampling program and the total PFAS are 1.95 ppt, well below the advisory level.

Water Treatment Facility Evaluation

The City relies on surface water from the Clear Creek watershed. Raw water is collected from the Berthoud Pass Ditch, Croke Canal and Church Ditch systems that deliver water to Standley Lake. From Standley Lake, water is conveyed to the WTP’s 40 million-gallon (MG) terminal reservoir through a 48-inch transmission line. Water may also be delivered to the terminal reservoir through the Farmer’s Highline Canal. From the terminal reservoir, raw water enters the pretreatment system, which consists of chemical addition, rapid mix and flocculation. Pretreated water is then conveyed to the clarification system for settling and solids removal. Clarified water is then treated with chlorine prior to entering the conventional filtration system. Filtered water enters two clearwells (A & B) in series to enhance chlorine contact time prior to being pumped to the distribution system. Solids from the clarifiers, as well as backwash water from the filters, are sent to a residual solids collection system, which consists of two settling ponds and a pumping station. The dilute residual solids are pumped to the sanitary sewer system. Unit treatment process capacities are as defined in the following table.

Unit Treatment Process Capacity

Process	Number of Units	Capacity Per Unit	Total Capacity
Raw Water Storage	1 reservoir	40 MG	40 MG
Rapid Mix	1 tank	14 MGD	14 MGD
Flocculation[1]	2 trains	6.2 MGD	12.4 MGD
Clarification[2]	2 basins	9.6 MGD	19.2 MGD
Filtration[3]	8 filters	2.5 MGD	20 MGD
Disinfection	Clearwell A	0.340 MG	0.884 MG
	Clearwell B	0.544 MG	
Low Zone High Service Pumps, CW B	4 pumps	3.3 MGD	31.1 MGD
High Zone High Service Pumps, CW B	4 pumps	2.0 MGD	
Low Zone High Service Pumps, CW A	3 pumps	3.3 MGD	

[1]Based on minimum detention time of 30 minutes and 2 ft freeboard
[2]Based on maximum surface loading rate of 0.7 gpm/ft^2
[3]Capacity per unit given by City – 5 gpm/sf

Each unit process was evaluated in this Master Plan. The following highlights the assessment and limiting factors.

Raw Water

The raw water mainline PRVs reduce over 60 psi of pressure and the City has expressed interest in replacing one of the PRVs with an in-line micro hydro turbine. Staff have noted inconsistent readings from the TR level monitoring system. Installation of a bubbler or pressure transducer at the TR outlet is being considered.

Pretreatment

There is only one rapid mix chamber with one mixer. This is a single point of failure in the WTP. It is suggested that the City purchase a backup mixer to be stored on site.

Flocculation

The flocculation basins were designed for a 7 MGD capacity for each train. At the time of the original design, a 20 to 30-minute detention time range was required per the State design criteria, which has since been updated to a 30-minute standard. Including the volume of the splitter box, the detention time per train is 24.2 minutes at a depth of 14.1 feet at 14 MGD, which meets the CDPHE standard in 2014. Using the updated 2017 State regulation and a 2 ft freeboard, the maximum flow rate through both basins while still achieving a 30 min detention time is approximately 12.4 MGD.

The velocity of the flocculated water through the two 30-inch pipes leaving the process is approximately 2.21 ft/s each at 7 MGD for a combined total of 14 MGD. The CDPHE requirement is greater than 0.5 ft/s and less than 1.5 ft/s. The maximum velocity of 1.5 ft/s is reached at 9.5 MGD.

The velocity of the flocculated water in the 30-inch pipes is within acceptable ranges for the current and projected treatment flows. It is recommended that the City request a variance for the flocculation time and velocity during the preparation of the next Basis of Design Report (BDR) to retain the 14 MGD capacity rating.

The drainpipe from the flocculation basins is prone to clogging and is hydraulically restricted where the 6-inch drain line from the flocculators ties into the 4-inch line that conveys flows to the waste ponds. The drains from the flocculation basins are used infrequently, however the restriction is a maintenance issue. Replacement of the existing 4-inch line is recommended.

Clarification

The treatment process includes two 110 ft diameter clarifiers that are performing well. The combined capacity exceeds 14 MGD. During the 2017 Sanitary Survey, CDPHE staff inspected the clarifiers and observed that the effluent weirs on both clarifiers were uneven as seen from varying rates of flow at different locations around each clarifier. The department recommended that the clarifiers be evaluated to determine the severity of potential short-circuiting. The City implemented the State recommendations in 2019 and recoated the rake arms, skim coated the floors and replaced the launders and weirs.

Powder Activated Carbon Contact Chamber

A powder activated carbon (PAC) contact chamber is located after clarification and before filtration. The PAC chamber is no longer used. If needed, the PAC chamber volume can be incorporated for additional CT for disinfection.

Filtration

Settled water from the clarifiers is distributed onto eight (8) conventional tri-media filters. The water level in the filters is set to be maintained at 42 inches above the media by a modulating valve located before rapid mix. The filter media has a depth of 30 inches and is comprised of anthracite, silica sand, and garnet sand. The original gravel underdrain system has been replaced with the Leopold® IMS® (Integral Media Support) cap in 2001. The City has filter to waste (FTW), but the system is not automated so it is rarely used. New valves and actuators are needed to incorporate the FTW into the backwash scheme.

The butterfly valves and actuators in the filter gallery are approximately 40-years old and in need of replacement. Most of the existing filter actuators are pneumatic and should be replaced with the best available technology. With exception to the recently installed FTW piping, there is significant corrosion to the remaining filter piping in the gallery, especially the piping closer to the ground level. All the pipes should be sandblasted and re-coated.

The filter backwash and surface wash are controlled using the original filter control consoles (FCC) consisting of old relays and controls that are no longer supported by the manufacturer. Operation staff are interested in filter automation by integrating a filter building PLC with user interface and SCADA.

There is a strong chlorine smell within the filter building. A new ventilation system is recommended to provide better air circulation.

Disinfection

Filtered water flows through two clearwells in series. The clearwells can operate independently, allowing for temporary shutdown of either clearwell during maintenance. Clearwell A and B are baffled and have respective volumes of 340,000 and 544,000 gallons. Primary disinfection is provided by a sodium hypochlorite system. A 10 percent sodium hypochlorite solution is injected into the clarified water, prior to filtration, in a chlorine injection vault upstream of the PAC chamber.

The maximum flow rate through both clearwells was assessed using a target pH of 8.2 and chlorine residual of 1.2 mg/L. Using the required giardia log removal rate as the control point and winter conditions, the maximum flow rate through the clearwells in series is 15 MGD.

There is only one point of chlorine injection located in an aging and deteriorating manhole. Replacement of the injection point vault is recommended.

High Service Pumps

The distribution system is divided into the low zone, intermediate zone, and high zone. At the WTP, finished water can be pumped either to the low zone and/or high zone. The typical operation scheme is to pump all treated water to the low zone storage tanks and utilize the system booster station to convey finished water to the high zone. Intermediate zones are typically back fed through a PRV from the high zone.

The WTP has four low zone (150 hp) and four high zone high service pumps (150 hp) located after clearwell B, and three low zone pumps (125 hp) located after clearwell A. Clearwell B pumps are original to the Plant while the clearwell A pumps were installed in 2010. None of the pumps are operated with VFDs.

VFDs were purchased for the clearwell B low zone pumps in 2018 and are unusable because of harmonic issues. The pumping system should be evaluated to assess the pump conditions and review the vibration or harmonic issue. Once the vibration issue is rectified the City should proceed with the VFD installations.

The low zone pumps from Clearwell A have not been used since installation. The City will be operating the pumps in 2020 to flush the system and test performance.

Chemical Feed and Storage Systems

The sodium permanganate, alum, polymer, sodium hydroxide, and sodium hypochlorite systems are located within a single chemical building. Sodium hypochlorite and sodium permanganate are each stored in separate rooms. Alum, polymer, and sodium hydroxide share a common room.

The sodium permanganate, alum, polymer, chlorine and caustic feed systems all use the same model of chemical feed pump with interchangeable parts which allows for flexibility and redundancy. However, the WTP only has a single shelf spare to serve as a redundant pump for all systems. CDPHE Design Criteria requires that standby units for chemical feeders must be provided. The alum storage is in flux and the current in process improvement of installing six new tanks should be completed along with the construction of secondary containment. In addition, secondary containment improvements are needed for the sodium hydroxide and polymer systems.

The chemical building is original to the WTP and has served its useful life of over 40 years. Building HVAC and electrical components have aged and access to tanks and equipment is inadequate. It is recommended that a more thorough evaluation be conducted for the entire chemical feed and storage system and building. This evaluation should include occupancy classifications, chemical storage requirements, fail/safe controls, electrical/control elements, ventilation, fire suppression, safety showers and a review of applicable CDPHE, NFPA and building codes.

Residuals Handling

The solids handling system consists of two recycle ponds (north and south) and a pump station. Filter backwash water, solids from the clarifiers and plant drains all flow by gravity to the north pond, which is hydraulically connected to the south pond. The water/solids mixture from the north pond is currently pumped to the sanitary sewer collection system.

The existing residuals handling system is outdated and poorly functioning. The residuals handling system needs to be improved to allow for recycling of the backwash water and reduce waste volumes sent to the sanitary sewer collection system. This can be achieved though the addition of settling basins or a gravity thickener providing high quality decant water and the implementation of a residuals dewatering process using mechanical equipment such as a screw press or rotary fan press. The residual handling system will require a new building, settling basins and associated infrastructure. Opportunities for beneficial reuse of the residual solids should also be explored.

Instrumentation and Controls

The control system is comprised of Allen-Bradley PLCs and an Intellution iFix 6.0 SCADA HMI software with stand-alone Historian server. This includes a redundant iFix system with active fail-over. They currently have a combination of Micologix, Compact Logix, and Control logix PLC’s.

The WTP has an existing 480 volt, 2000A electric service entrance Main Switchgear. This Main Switchgear is connected to a switchboard which in turn feeds the other power distribution equipment in the plant. All the service entrance equipment, except for the newer automatic transfer switch (ATS), appear to be original plant equipment.

At the WTP, the filter controls limit the operation of the filter and backwashes. With new controls, new programs may be developed for filter runs and backwashes that allow the operators to optimize the process. In addition, the electrical service entrance equipment is well past its safe life expectancy. It would be important to replace this equipment under a controlled planned scenario. This equipment creates a single point of failure that would completely shut down the WTP.

Staff were also interested in electrical upgrades for the off-site booster pump station. The booster pump station is fed with 480 volts, 400A, three phase power. This power connects to a main fused disconnect in the existing MCC located inside the station. At some time after the initial installation a generator and automatic transfer switch (ATS) were added to the station. This installation modified the MCC by taking power off the main MCC fused disconnect and routed cables outside the station to the ATS and then back from the ATS to the bussing in the MCC. This is how the MCC is today. Currently this MCC only has breakers (no starters) feeding the various loads within the station. The booster pump station MCC is old and has been modified too many times to be safe. It is also not supported by any manufacturer. This MCC should be replaced with a new Panelboard.

CAPITAL IMPROVEMENTS PLAN

In working with staff, the following projects where identified for inclusion in the capital improvements plan. Costs are shown as 2020 dollars. Smaller projects at budgets less than $150,000 are estimated based on industry experience and standards. Larger project costs are detailed in the opinion of cost matrixes provided in Appendix B. Projects projected for the 5 to 10-year horizon have been shown with a completion year of 2030.

Project No. 1 – AWIA – Risk Resilience Assessment & Emergency Response Plan

Risk Resilience Assessment

Year to Complete: June 2021

Anticipated Cost: $50,000

Emergency Response Plan

Year to Complete: December 2021

Anticipated Cost: $50,000

Project No. 2 – Hydropower Generation

Year to Complete: 2030

Feasibility & Interconnect Study: $50,000

Anticipated Cost: $1.0 million

Project No. 3 – TR Level Indicator

Year to Complete:2021

Anticipated Cost: $40,000

Project No. 4 – CL17 and Streaming Current Monitor (SCM)

Year to Complete: 2022

Anticipated Cost: $30,000

Project No. 5 – Chemical Feed and Storage Improvements

Higher Priority Improvements (Chemical Pumps & Secondary Containment)
Year to Complete: 2021
Anticipated Cost: $320,000

Chemical Systems Comprehensive Evaluation
Year to Complete: 2022
Anticipated Cost: $60,000

Project No. 6 – Rapid Mix Improvements (Spare Mixer)

Year to Complete: 2021
Anticipated Cost: $50,000

Project No. 7 – Flocculator Basin Improvements (4-inch Drain)

Year to Complete:2022
Anticipated Cost: $80,000

Project No. 8 – Chlorine Vault Improvements

Year to Complete: 2024
Anticipated Cost: $100,000

Project No. 9 – Filter Improvements

Year to Complete: 2023 to 2026
Anticipated Cost: $ 1.1 million

Project No. 10 – High Service Pump Improvements

Pumping System Evaluation
Year to Complete: 2021
Anticipated Cost: $40,000

Low Zone Pumps – VFD Installation (4)
Year to Complete: 2022
Anticipated Cost: $100,000

Project No. 11 – Residuals Handling Improvements

Year to Complete:2021-2022
Anticipated Cost: $3.6 million

Project No. 12 – Electrical and Controls Improvements

Year to Complete: 2025
Anticipated Cost
WTP Main Service Entrance Switchgear - $400,000
Filter Building MCC - $350,000
Booster Pump Station MCC - $150,000
Filter Control Console ($60,000 per x 4) = $240,000

FUNDING OPTIONS

The Colorado Water Conservation Board (CWCB) offers several loans and grants for water-related projects and offer interest loans at 2.15 to 2.75 percent for 20 years. Interest rates are dependent on income rates within the City. CWCB grants could possibly be used to fund a conduit turbine project or any other water conservation project. In addition, the Colorado Water Resource and Power Development Authority (CWRPDA), has a Small Hydropower Loan program that currently offers 30-year loans for projects up to $5-million dollars for governmental agencies.

The State Drinking Water Revolving Fund (DWRF) provides low interest loans to governmental entities for the construction of water projects for public health and compliance purposes. Possible loan types include direct loans of up to $3.0 million, current APR of 2.5 percent for 20 years, and leveraged loans, which are generally provided to investment grade borrowers with larger projects greater than $3.0 million at bond market interest rate for 20 years.

The Department of Local Affairs (DOLA) Energy and Mineral Impact Assistance Fund (EIAF) was created to assist political subdivisions that are socially and/or economically impacted by the development, processing, or energy conversion of minerals and mineral fuels. Loan types include administrative grants up to $25,000 for planning, preliminary engineering and architectural design projects, Tier I grants of up to $200,000 that can be used for a variety of public purposes including planning, engineering and design studies, and capital projects, and Tier II grants of up to $1.0 million for use on a wide variety of community development projects to improve quality of life in communities.

SECTION 1 – INTRODUCTION

JVA, Inc. was retained by the City of Northglenn (City) to prepare this Water Treatment Plant Master Plan Update (Plan). The plan was last updated in 2009 by HDR Engineering, Inc and since that time there have been system improvements including the addition of raw water piping, chemical feed, rapid mix, flocculation, a clear-well expansion and second high service pump station. In addition, improvements to the SCADA system, chemical storage and filtration systems have also been implemented.

PROJECT PURPOSE AND GOALS

The purpose of this Plan is to evaluate the capacity of each unit process and provide recommended system improvements and budgetary costs. To that end, in this report we have reviewed demand projections, inventoried the treatment facility, assessed system performance, provided near term recommendations, and developed a capital improvement plan (CIP). This Plan is a dynamic working document that provides a roadmap for future improvements based on aging infrastructure, equipment performance and reliability, and regulatory compliance. This Plan should be reviewed periodically and updated as conditions within the City's service area and at the WTP change. Recommendations identified in this Plan should be considered and used only as conceptual for planning purposes. A summary of the key report elements are as follows.

- ***Water Production Evaluation*** – Historical WTP flow data from 2015 through September 2019 was analyzed. Current annual average, peak and seasonal per capita water demand was estimated for customers in the service area. Peaking factors for peak day summer and winter conditions were also calculated.

- ***Water Demand Projections*** – Peak water demand projections were developed for the service area through the five-year, ten-year, and twenty-year planning horizon. These figures are based on historical population growth and a City unit growth scenario for buildout.

- ***Performance Evaluation*** – A system evaluation was prepared to examine the current treatment and hydraulic capacities, and limitations for each unit process.

- ***Capital Improvement Plan*** – Conceptual level opinion of probable costs (OPC) are prepared as part of the capital improvement plan (CIP) for recommended projects identified during the planning efforts. Projects are prioritized based on City Staff input and demand projections.

- ***Project Financing*** **–** The City may desire to use alternate funding packages for project implementation. A summary of eligibility and application requirements for several funding sources are provided.

SECTION 2 – PLANNING CONDITIONS

The planning service area and projections are defined in this section. Current water production, demands and population are defined along with projected growth. The City's Integrated Water Resource Plan is being completed concurrently with this Plan and a brief summary is also provided in this section.

PLANNING AND SERVICE AREA

The City currently serves a population of approximately 39,000 people with 10,295 water taps. The service area is located within the City boundary and encompasses approximately 4,800 acres of land. The City does not plan to expand domestic water service outside the existing service area boundaries. The service area, development sites and major infrastructure are depicted in Figure 1.

The City has four remaining areas of development prior to reaching build out. The first area, Karl's Farm, is under contract for a mixed-use development consisting of single family, multi-family, apartments and commercial units. The Karl's Farm development is approximately 64 acres and will add approximately 800 units to the service area. The second area is the 112th Station Area, which is located along York Street between 112th Avenue and 120th Avenue. This area is not currently under contract but is expected to add approximately 216 units. The Civic Center development is expected to have 154 units, and the size of the Market Place development is unknown but is assumed to include 185 units. A summary of buildout taps/units is provided in Table 1.

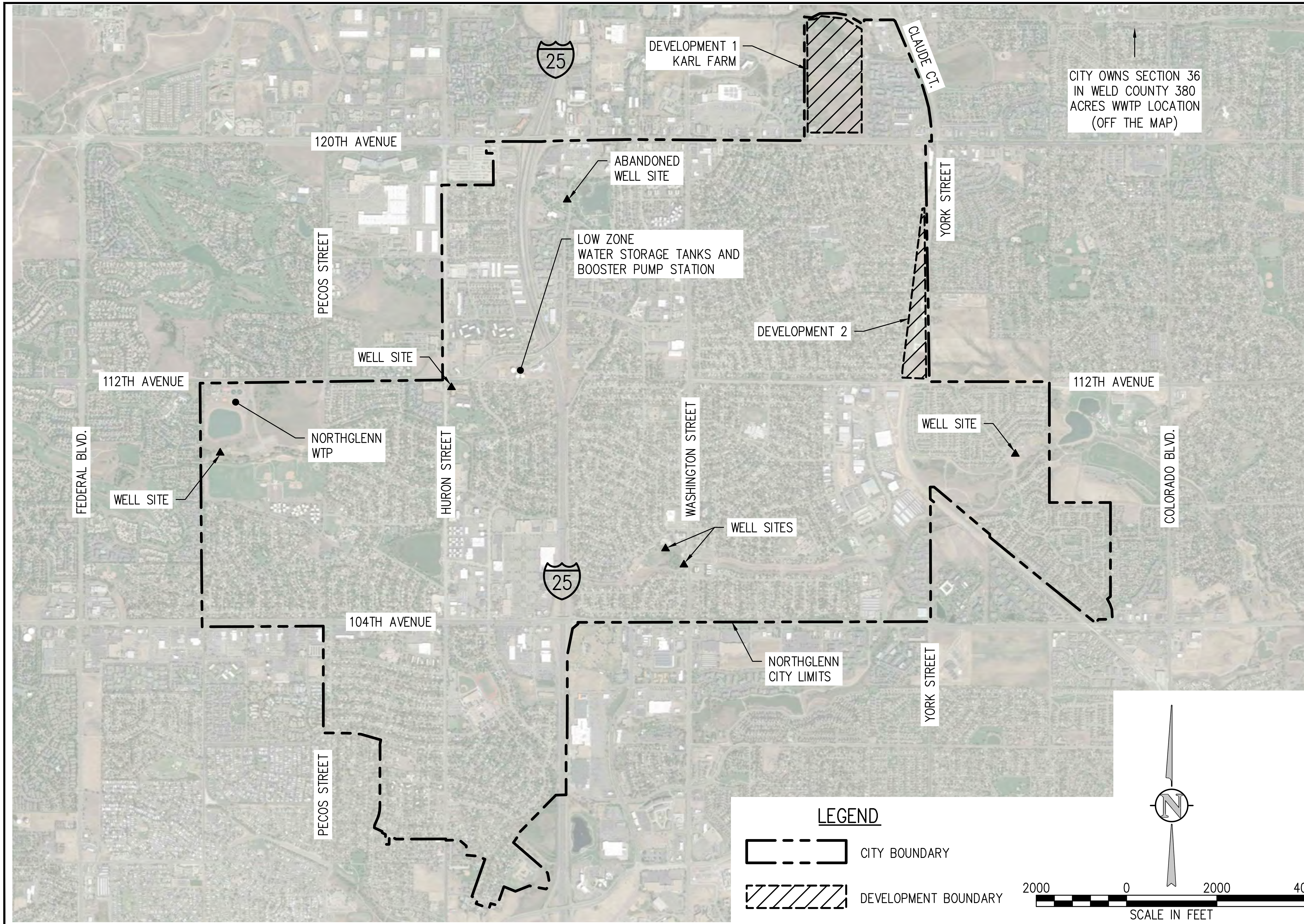
Table 1. Summary of Tap Development

Area	Water Taps
Current	10,295
Karl's Farm	800
112th Station Area	216
Civic Center	154
Market Place	185
Total	**11,650**

INTEGRATED WATER RESOURCE PLAN

The City is working with Element Water Consulting (Element) on an Integrated Water Resource Plan (IWRP) to help determine water rights and water storage needs. The IWRP will analyze the demands of the system and identify projects and water resources to sustain the community through drought conditions.

Element has identified water storage as the top priority. The City does not have enough storage in Standley Lake or the Terminal Reservoir, which limits full use of the City's water resource portfolio. An option being explored is Aquifer Storage Recovery (ASR). Treated water would be



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CONSULTING ENGINEERS

FIGURE 1 - SERVICE AREA
CITY OF NORTHGLENN WATER MASTER PLAN UPDATE
AUGUST 2020

N:\1061e – Northglenn WTP Master Plan Update\Drawings\Exhibits-Figures\Water Master Plan Update\1061e – Figure – Service Area.dwg, 8/27/2020 – 11:17 AM, LLG pumped into the Denver Basin Aquifer, specifically Laramie Fox Hills (LFH) and the Lower Arapahoe for storage and later extraction as needed for treatment. The City owns two existing wells that may be used for the ASR process, but they are over 40 years old and in poor condition. The LFH aquifer water quality can be poor and treating this comingled water could be a future issue. The ASR analysis is being performed by HRS Water Consultants, Inc. and being coordinated in the IWRP. The potential water quality concerns are further detailed in Section 3 of this report.

The IWRP also investigates other storage options and water rights from different supplies and focuses on advanced conservation to reduce water demand. For more information, please reference the IWRP report produced by Element.

Water demand projections developed for the IWRP are also discussed and compared later in this Section.

PLANNING PERIOD

A 20-year planning period was selected for this Plan, which is projected to include full build-out of the service area. Adjacent communities and districts boarder the City and the growth boundary cannot expand. As indicated, there are only a few remaining development areas. Full buildout will occur when all plated lots within the service area boundary have been developed. The expected number of taps at buildout is approximately 11,650.

LAND USE AND ZONING

Zoning information for the City was provided by the Unified Development Ordinance (UDO). The City categorizes land use based on four zoning districts: residential, mixed use and commercial, nonresidential and planned development units (PD). Each zoning district is further defined into zoning classifications. Residential districts are made up of single family and multifamily lots and makes up approximately 94 percent of the City’s water taps, while 5.5 percent of water taps are in mixed use and commercial districts. The remaining 0.5 percent of taps are City-owned.

HISTORICAL TRENDS

Historical trends for population growth and water production are described below. The historical data is used as the basis for future projections.

POPULATION

Historical population data and estimates for the City were obtained from various sources, including the American Community Survey (ACS) and the US Census Bureau (Census), and are summarized in Table 2. The Census reports population estimates every 10 years while ACS data has been collected annually since 2010. Based on the ACS and Census data, the City has experienced an average annual growth rate of 1.0 to 1.2 percent since 2010, with growth rates peaking between 2014 and 2016 and a decline in the last 3 years. Figure 2 shows the historical population graphically. The 2019 population for Northglenn was 38,694.

Table 2. Historical Population

Year	ACS Population		US Census	
	Population Estimate	Annual Growth Rate (AGR)	Population Estimate [1]	Annual Growth Rate (AGR)
2010	35,895		35,789	1.3%
2011	36,438	1.5%		
2012	36,949	1.4%		
2013	37,463	1.4%		
2014	38,534	2.9%		
2015	38,865	0.9%		
2016	39,126	0.7%		
2017	38,955	-0.4%		
2018	38,870	-0.2%	39,010	1.1%
Average Annual Growth Rate		1.0 %		1.2%

Note:
[1] All information provided by the Census Bureau's Population Estimates Program (PEP)

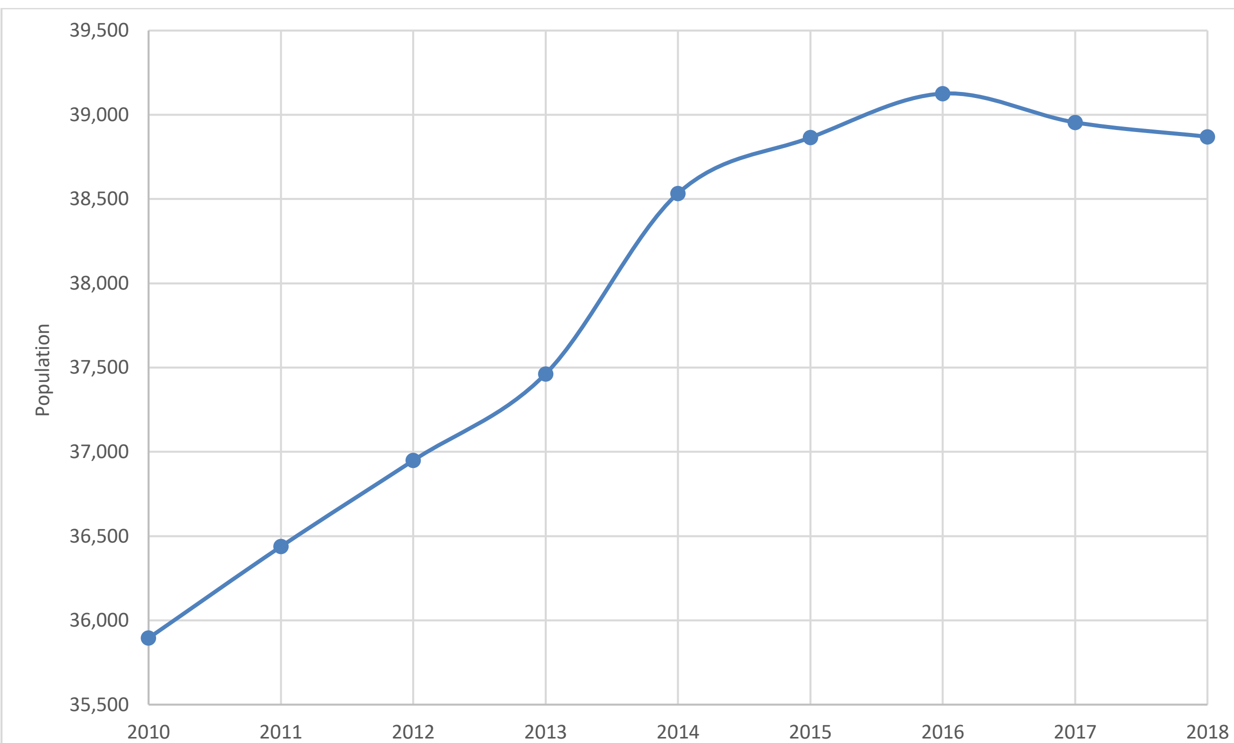


Figure 2. Historical Population

WATER PRODUCTION

Based on historical water production from January 2015 to September 2019, the WTP produced an average of 3.92 million gallons per day (MGD). The average summer and winter water production rates are 5.11 MGD and 2.55 MGD, respectively. The peak day production rate over the past 5 years is 8.68 MGD, which occurred in 2018, and the average annual peak day production is 8.32 MGD. The City calculates daily flow to the distribution system by subtracting clarifier waste flows and backwash waste flows from the raw water total. Table 3 summarizes the annual and seasonal water production rates for the WTP. Summer months are defined as April through September, and winter months are October through March.

Table 3. Historical Water Production

Year	Total Annual Production	Maximum Month Production	Max Month	Maximum Month Average Day Flow	Annual Average Day Flow	Average Summer Flow	Average Winter Flow	Annual Peak Day Flow	Peaking Factor
	gallons	gallons		MGD	MGD	MGD	MGD	MGD	
2015	1,382,924,759	213,146,351	Aug	6.88	3.79	4.90	2.67	8.16	2.15
2016	1,436,566,985	231,718,126	Jul	7.47	3.93	5.16	2.69	8.61	2.19
2017	1,435,488,696	215,831,122	Jul	6.96	3.73	4.95	2.50	8.17	2.19
2018	1,451,233,946	213,172,982	Jul	6.88	3.98	5.45	2.49	8.68	2.18
2019	1,148,061,114	212,958,324	Aug	6.87	4.21	5.10	2.40	7.97	1.89
Average	1,370,855,100	217,365,381	Jul	7.01	3.92	5.11	2.55	8.32	2.12

A graphical representation of the monthly water production per year is presented in Figure 3. The historical monthly water production rates show low variability year to year and indicate that monthly water production has remained relatively consistent in the last 5 years. The historical data shows that the winter months experience less variability than the summer months.

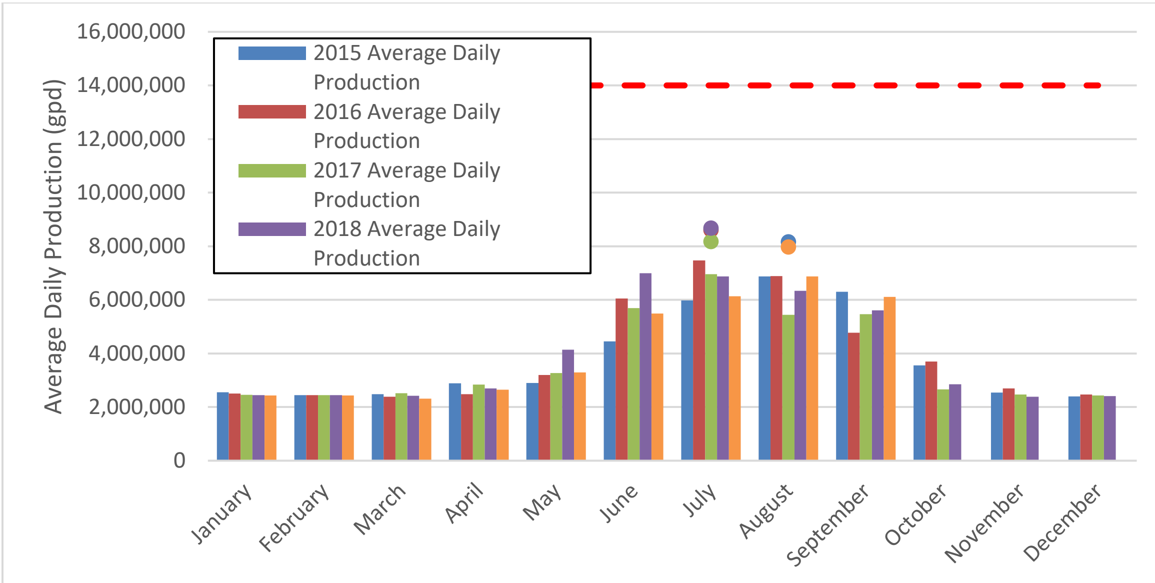


Figure 3. Summary of Monthly Historical Water Production

PROJECTIONS

Projections for population growth and water production rates are described in the following. A comparison with the IWRP water demand projections is also provided. The IWRP considers extreme water conservation and climate change effects in predicting demands.

POPULATION

The City's serviceable population is made up primarily of full-time residential units with no industrial users and a limited number of commercial taps. Projection data from the Colorado Division of Local Government, State Demography office predicts that the City will experience a 1.5 to 2.0 percent annual growth in population from 2020 to 2040. However, the City has nearly reached build-out capacity and does not plan to expand outside the existing service area boundaries. In addition, the City has experienced minimal tap growth in the past five years, with only 44 taps added between 2017 and 2019. Therefore, a conservative growth rate of 0.5 percent was used to estimate future population. This rate corresponds with the last five years of historical population growth. Using a growth rate of 0.5 percent per year, the number of residents, including the Karl's Farm and York Street developments, could reach approximately 43,000 people by 2040, as indicated in Figure 4.

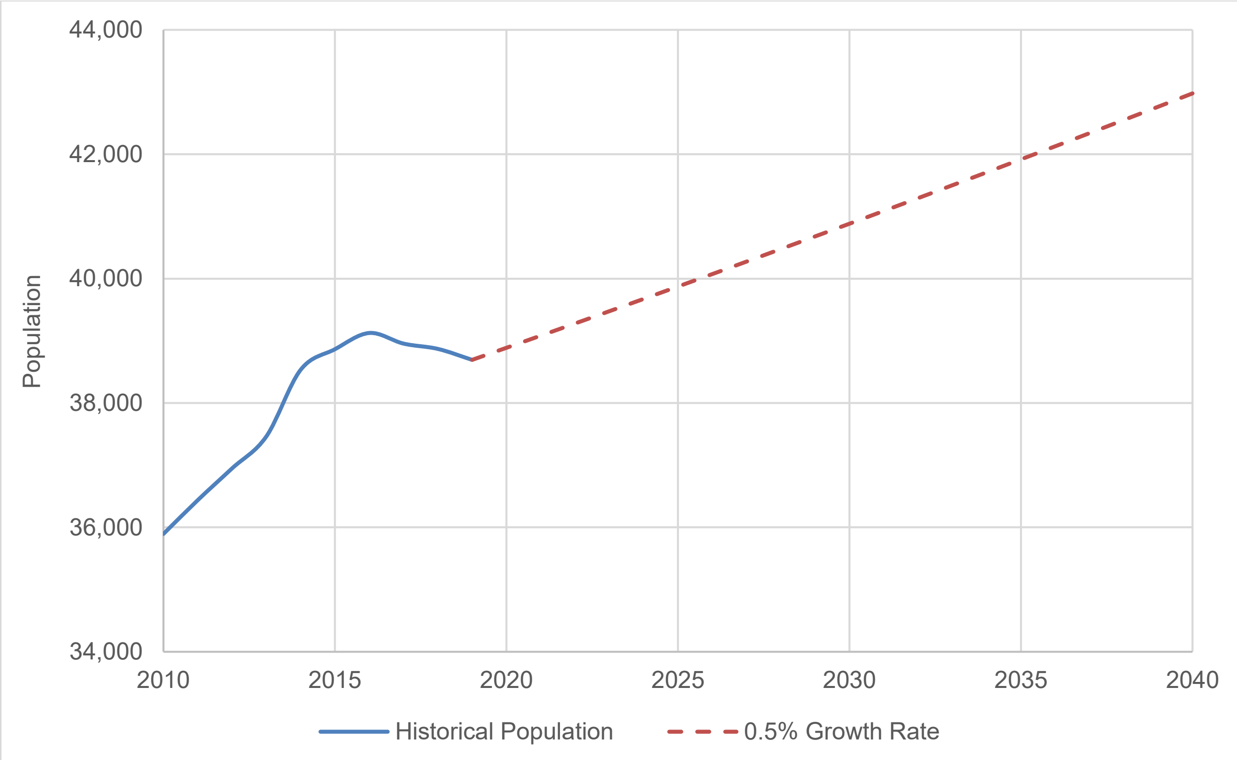


Figure 4. Population Projections

Water Production Projections

The current per capita water production rates were used to determine the projected water production through 2040. By dividing the current average and peak day water production rates by the current population, it was determined that the average annual water use per person is 101 gallons per capita per day (gal/cap/day) and the peak day water use is 224 gal/cap/day. By applying these rates to the projected population, the projected average and peak day production is 4.36 and 9.64 MGD by 2040.

The same method used to determine projected average day and peak day production was used to determine projected average seasonal production. The average summer water use per person is 132 gal/cap/day and 66 gal/cap/day during the winter months. By applying these rates to the projected population, the projected average summer and average winter production is 5.68 and 2.83 MGD by 2040. The seasonal water production projections are depicted in Figure 5.

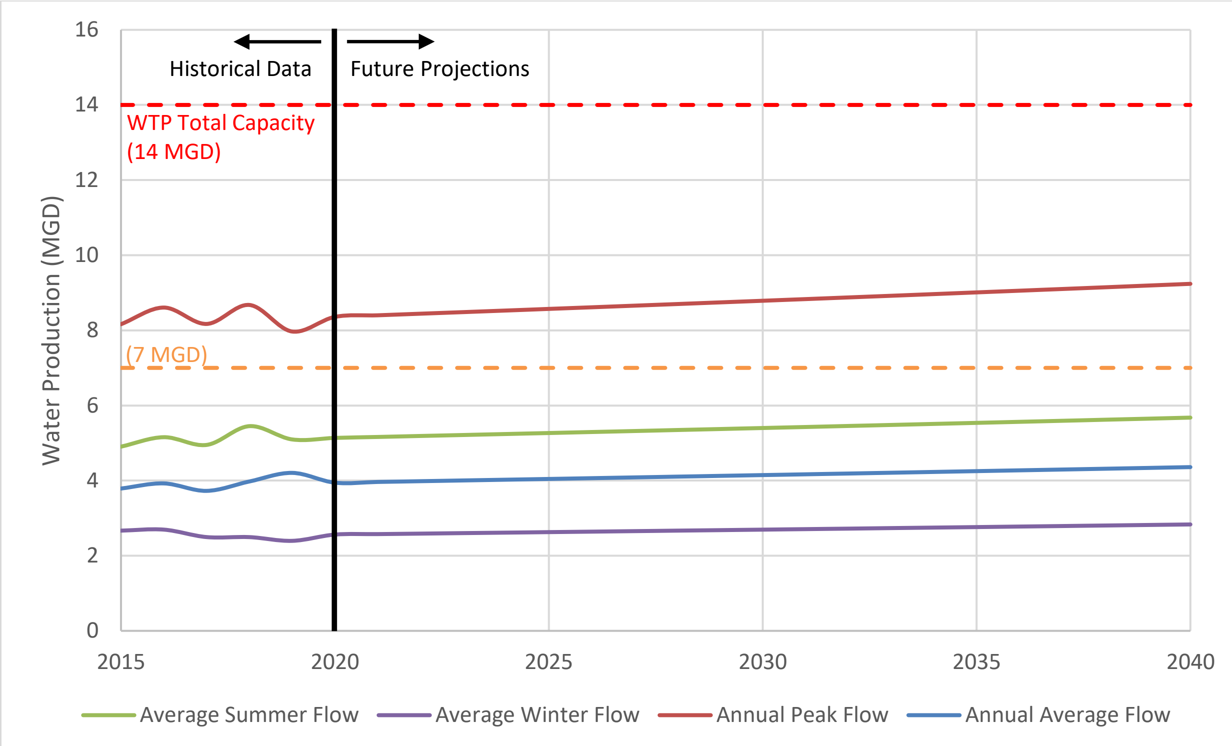


Figure 5. Historical and Projected Water Production Rates

IWRP Projections

The IWRP includes three different average annual water demand projections that are based off a 10% population growth projection and historical demands. The population growth projection includes the anticipated new units from the four developments, resulting in 1,355 new units and an estimated population of 42,162 people at full buildout in 2050. The IWRP's maximum projected average annual water demand from the three scenarios is 1,634 million gallons or approximately 4.48 MGD by 2050, similar to the buildout projection in this Plan. Peak day demands were not analyzed in the IWRP.

The IWRP further analyzes the seasonal variability of demands and more information on the IWRP's projections can be found in the final IWRP report. The values presented in the Plan are from a draft of the Task 1 Supply and Demand Conditions memo and may not accurately portray the values in the final report.

Summary

The projected demands were calculated based on an expected growth rate of 0.5 percent with buildout occurring by 2040. Based on the demands, future average and peak day water production rates are projected to be approximately 4.4 and 9.7 MGD. The projected peak day demand is well below the 14 MGD total capacity of the plant.

SECTION 3 –WATER QUALITY AND REGULATORY REQUIREMENTS

Presented in this section is a discussion of the City's raw and finished water quality and current regulatory framework of the Colorado Primary Drinking Water Standards. In addition, future water quality regulations that may impact the City are reviewed.

SOURCE WATER

The existing water supply sources for the City consists of surface water from the Clear Creek watershed. Water is collected through the Berthoud Pass Ditch, the Croke Canal, and the Church Ditch and stored in Standley Lake. The City shares the water stored in Standley Lake with other cities including Thornton and Westminster. From Standley Lake, the City's water is conveyed to the Terminal Reservoir (TR) prior to treatment and distribution. In accordance with Colorado's Source Water Assessment and Protection Program (SWAP), administered by the Colorado Department of Public Health and Environment (CDPHE), the cities of Northglenn, Westminster and Thornton collaborated to prepare the *Source Water Protection Plan for the Upper Clear Creek Water Shed and Standley Lake.* The plan was completed in 2010.

The City has historically tested its finished water for inorganic chemicals (IOCs), synthetic organic chemicals (SOCs), volatile (VOCs), nitrates (and nitrites), radionuclides, chlorine residual, disinfection byproducts (DBPs) consisting of TTHMs and haloacetic acids (HAA5s), fluoride, lead and copper.

PRIMARY DRINKING WATER CONTAMINANTS

Primary drinking water standards include enforceable maximum contaminant levels (MCLs) and not enforceable maximum contaminant level goals (MCLGs). Primary contaminants are defined in the Colorado Primary Drinking Water Regulations (CPDWR) along with their respected limits. The City must meet all MCLs to maintain compliance with the CPDWR.

Archived Consumer Confidence Reports (CCRs) provide test results for primary and secondary contaminants, lead and copper, and DBPs, but these tests were only performed on finished water. The CCRs are publicly available on the City's website. Any contaminant not shown was not detected during water quality testing over the past five years.

INORGANICS

The inorganics group consists of elemental metals and nitrogen containing compounds. Table 4 presents the MCLs, MCLGs, and laboratory test results for inorganics tested by the City. Finished water sampled for inorganics in 2012 and 2018 were all below the MCLs. Lead, Copper, and Fluoride will be discussed in later sections.

Table 4. Inorganics MCLs, MCLGs, and Test Results

Contaminant	MCLG (mg/L)	MCL (mg/L)	2012 Results (mg/L)	2018 Results (mg/L)
Antimony	0.006	0.006	0.00024	NT
Arsenic	0	0.010	0.00032	NT
Barium	2	2	0.00005	0.00005
Chromium (total)	0.1	0.1	0.00061	NT
Selenium	0.05	0.05	0.00050	NT
Thallium	0.0005	0.002	0.00004	NT

VOLATILE ORGANIC COMPOUNDS

VOCs are chemicals that readily evaporate at normal temperatures and pressures. VOCs have not historically been found in the City's finished drinking water, and so they were not investigated as part of this effort. Testing the finished drinking water for VOCs is required once per year.

SYNTHETIC ORGANIC COMPOUNDS

SOCs are synthetic chemicals such as pesticides and fuel additives. Testing for SOCs is expensive and time consuming. SOCs have not historically been found in the City's drinking water sources, and so they were not investigated as part of this effort. Testing for SOCs is required twice every three years by the City's Monitoring Schedule set by CDPHE. Two samples must be collected in the same calendar year but in different quarters. SOCs were sampled in April of 2019 and the next samples must be collected in 2022. A sampling schedule through 2030 is provided in Table 5 below.

Table 5. SOC Sampling Schedule

Sampling Year	Sampling Status
2019	Completed
2022	Scheduled
2025	Scheduled
2028	Scheduled
2031	Scheduled

*Based on current monitoring schedule, which is subject to change based on CDPHE

RADIONUCLIDES

Radionuclides are unstable forms of elements that can occur in natural or man-made deposits. The City's finished water was tested for radionuclides in 2012. All results were far below MCLs. Table 6 presents the MCLs, MCLGs, and laboratory test results for radionuclides. All radionuclides are measured in picocuries per liter (piCi/L) unless otherwise noted. Radionuclide testing is required every 9 years. The next radionuclide samples are due in 2021.

Table 6. Radionuclides MCLs, MCLGs, and Test Results

Contaminant	MCLG (piCi/L)	MCL (piCi/L)	Results (piCi/L)
Alpha particles	none ---------- zero	15	0.8
Beta particles and photon emitters	none ---------- zero	50	0.66
Radium 226 and Radium 228 (combined)	none ---------- zero	5	0.18
Uranium	zero	30 ug/L	0.0002 ug/L

DISINFECTANTS

Disinfection is required for inactivating viruses, bacteria, and protozoa. The City injects chlorine before filtration but does not include the filter volumes when calculating chlorine contact time (CT). CT is calculated using only the volume of the two baffled clearwells at the WTP. Any disinfectant remaining in the water following disinfection is referred to as residual.

The CPDWR sets a maximum residual chlorine level of 4.0 mg/L to protect consumers from drinking harmful amounts of disinfectants and to reduce production of disinfection by products (DBPs). Residual chlorine prevents organic growth and the spread of bacteria in distribution systems and the storage tanks. The City targets a chlorine residual of 1.2 mg/L leaving the WTP. The minimum requirement set by the Long Term 2 Enhanced Surface Water Treatment Rule is a residual of 0.2 mg/L at the farthest point in the distribution system. Residual chlorine in the distribution system was not reviewed as part of this Plan.

In an effort to improve disinfection practices around the State, CDPHE has recently implemented a new program named the Disinfection Outreach Verification Effort (DOVE). A DOVE outreach letter was sent to the City in 2014, however, a DOVE evaluation of the WTP has not been performed by CDPHE and has therefore not been reviewed as part of this Plan.

DISINFECTION BYPRODUCTS

Residual disinfectants can react with organic compounds remaining in the finished water to form disinfection byproducts (DBPs). The two major groups of DBPs are Total Trihalomethanes (TTHMs) and Haloacetic Acids (HAA5s). Table 7 presents the MCLs, MCLGs, and laboratory test results for DBPs.

Table 7. DBPs MCLs, MCLGs, and Test Results, 2016-2019

Contaminant	MCLG (mg/L)	MCL (mg/L)	Average Results (mg/L)	Maximum Results (mg/L)
HAA5	n/a	0.060	0.0207	0.0393
TTHMs	n/a	0.080	0.0397	0.0509

The DBPs concentrations from 2016 to 2019 from different distribution locations are shown in Figure 6 below. Treated water total organic carbon (TOC) is also shown for comparison. TOC is a measurement of the total organic content in the water and it is important because organic carbon compounds are precursors for the formation of DBPs when reacting with chlorine. TOC is common in surface waters. Historical data indicates that DBP levels are well below the MCL.

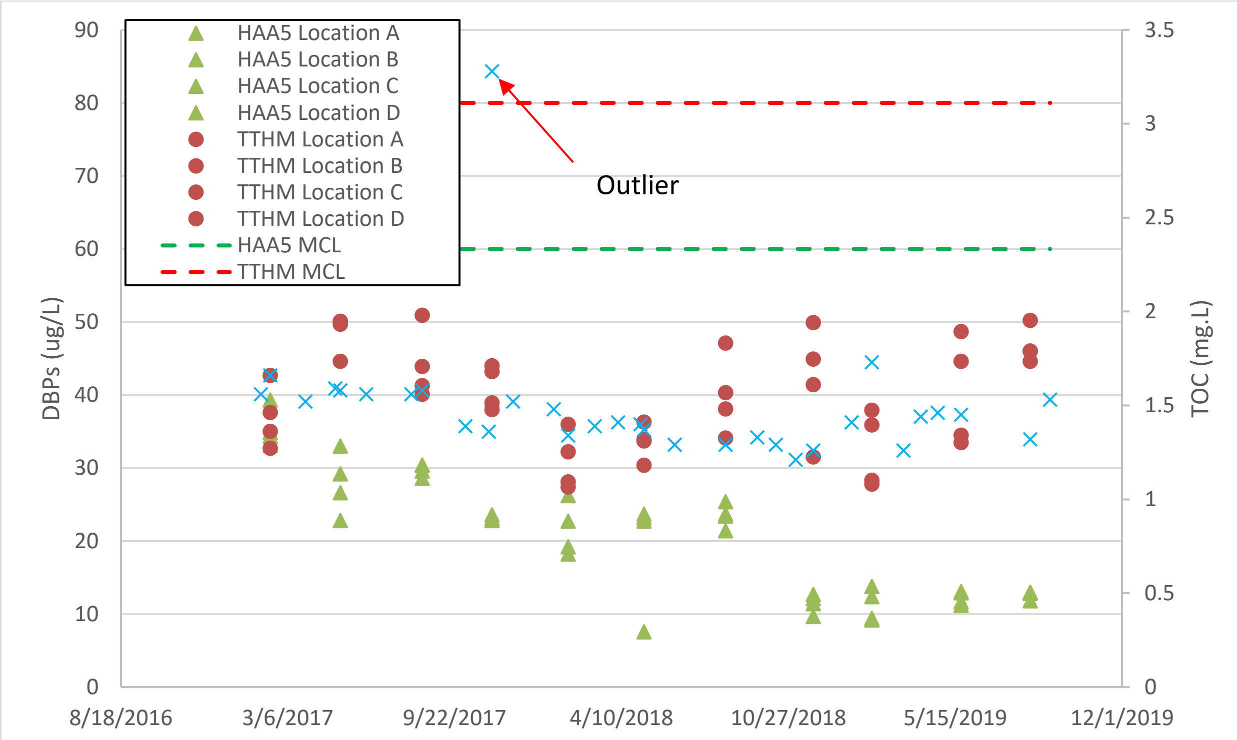


Figure 6. DBP and Finished Water TOC

LEAD AND COPPER

The CPDWR sets action levels for lead and copper concentrations for distribution systems. If concentrations exceed the action level limit, the City would be required to comply with additional requirements, which may include public education, corrosion control treatment, additional sampling, source water treatment, and/or lead service line replacement. Historical lead and copper levels from 2014 through the 2018 CCR are shown in Table 8 and indicate that all test results are below the action level.

Table 8. 2014 to 2018 Lead and Copper MCLs, MCLGs, and Test Results

Contaminant	MCLG (mg/L)	MCL (mg/L)	Average Results (mg/L)	Maximum Results (mg/L)	Minimum Results (mg/L)
Copper (90th Percentile)	1.3	Action Level=1.3	0.07	0.08	0.07
Lead (90th Percentile)	0.0	Action Level=0.015	0.0019	0.0020	ND

SECONDARY DRINKING WATER CONTAMINANTS

Secondary drinking water contaminants primarily affect the aesthetic qualities (taste and odor) relating to the public's acceptance of drinking water. The CPDWR defines secondary maximum contaminant levels (SMCLs), but they are not enforceable. They are intended to represent reasonable goals to reduce public health implications and water aesthetic degradation. Table 9 lists the most recent raw water test results and associated SMCLs.

Table 9. Select SMCLs and Test Results

Contaminant	SMCL (mg/L)	Average Concentrations (mg/L)	Maximum Concentrations (mg/L)
Fluoride	2.0 mg/L	0.48	0.49
Iron	0.3 mg/L	0.002	0.005
Manganese	0.05 mg/L	0.010	0.022
Total Dissolved Solids (TDS)	500 mg/L	170	215

IRON AND MANGANESE

Iron and manganese impact drinking water quality by producing a metallic taste on finished water and can precipitate in the distribution system, staining laundry and household fixtures if not removed. Oxidized iron and manganese can form a reddish-brown coating on distribution pipes which may slough off later into the water. The iron and manganese SMCLs are 0.3 and 0.05 mg/L, respectively. In 2019, the average raw water iron level was 0.03 mg/L, with a peak of 0.08 mg/L and the average manganese level was 0.06, with a peak of 0.4 mg/L. Raw and finished water iron and manganese concentrations are collected daily and weekly at the clarifier.

TOTAL DISSOLVED SOLIDS

Total dissolved solids (TDS) is a measurement of the total dissolved charged ions in the water. Water high in TDS can lead to excessive scaling and taste issues. If the high TDS is due to high concentrations of chloride and sulfide, the water may be corrosive to iron-based materials, as previously discussed. If the high TDS is due to high concentrations of bicarbonate and hardness ions, the water may be corrosive to copper piping. In addition, water high in TDS tastes salty. The SMCL for TDS is 500 mg/L. Raw water TDS ranges from 120 to 200 mg/L with an average of 150 mg/L.

FLUORIDE

Fluoride is regulated as a secondary contaminant. While low levels of fluoride can help prevent cavities in teeth, fluoride concentrations above 2 mg/L may result in cosmetic discoloration of permanent teeth (dental fluorosis) in children under nine years of age. The problem occurs only in developing teeth, before they erupt from the gums, so older children and adults are not at risk.

Naturally occurring fluoride levels in the City’s water supply are well below 2 mg/L and cosmetic discoloration due to fluoride concentrations is not a concern. Currently the City does not add any supplemental fluoride to the treatment process.

FUTURE CONSIDERATIONS

The City is currently investigating the water quality of the Laramie-Fox Hills (LFH) aquifer through existing wells, LF-4 and LF-5, for ASR. The LFH aquifer could provide approximately 2,000 acre-feet (AF) of storage to help mitigate drought conditions. The treated water volume is minimal compared to the overall aquifer and unlikely to impact water quality. Initial samples of the LFH water quality are presented in Table 10.

Table 10. LFH aquifer Test Results

Parameters	Units	LF-4	LF-5	WTP Average Raw Water	MCL/SMCL*
Alkalinity	CaCO3 mg/L	520	390	56	-
Ammonia Nitrogen	mg/L	0.73	0.84	-	-
Asbestos	MFL	-	0.18	-	-
Barium, Total	ug/L	64	45	0.28	2000
Bicarbonate Alkalinity	as mg/L of HCO3	630	460	-	-
Boron, Total	mg/L	0.26	0.29	-	-
Calcium, Total	mg/L	3	1.4	-	-
Carbonate	as mg/L of CO3	16	30	-	-
Chloride	mg/L	45	110	-	250*
Erucylamide	ug/L	22	-	-	-
Fluoride	mg/L	3.2	1.2	0.48	4.0, 2.0*
Iron, Total	mg/L	0.37	0.074	-	0.3*
Magnesium, Total	mg/L	0.73	0.32	-	-
Manganese, Total	ug/L	10	6.4	-	50*
Molybdenum, Total	ug/L	2.2	-	-	-
pH		8.6	9	-	6.5-8.5*
Potassium, Total	mg/L	1.9	1.4	-	-
Sodium, Total	mg/L	250	250	22.7	-
Specific Conductance	umho/cm	1100	1200	-	-
Strontium	mg/L	0.074	0.069	-	-
Sulfate	mg/L	-	28	-	250*
TDS	mg/L	650	670	-	500*
Total Organic Carbon	mg/L	2.2	1.5	2.25	-
Gross Beta Particle Activity	pCi/L	ND	ND	0.66	50
Radium 226	pCi/L	ND	ND	0.18[1]	5
Radium 228	pCi/L	ND	ND		
Gross Alpha	pCi/L	3.9	ND	0.80	15

*SMCLs are identified with an asterisk

[1]City takes combined radium for CCRs

We have summarized some of the impacts of using the LFH groundwater in the following.

- The pH level is above the SMCL. This may result in increased coagulant use or chemical addition to lower the pH.
- The TDS levels are also above SMCL. TDS is a difficult constituent to treat and could require reverse osmosis for reliable removal. High levels of TDS can cause hardness, deposits, colored water, staining, and a salty taste, that may be unpleasant to customers.

- The fluoride levels are much higher than the City's raw water. Fluoride in high concentrations can cause bone disease. This could require the addition of fluoride removal technologies.
- Total iron levels are above SMCL. The City currently doses potassium permanganate for iron and manganese removal and substantial process additions would not be anticipated.
- The alkalinity levels of the aquifer samples are much higher than the City's raw water. This could increase coagulant use and increase the amount of sodium hydroxide for pH control.
- Total sodium levels are much higher than the City's raw water quality. Sodium concentrations above 200 mg/L may result in a salty taste that is unpleasant to customers.

Changing sources and blending of sources may result in unpredictable water treatment challenges. Pretreatment of the LFH aquifer water should be considered if ASR is planned for implementation.

Within the next five years, CDPHE anticipates modifications to the lead and copper rule (LCR). The LCR was first introduced in 1991 by the EPA to control lead and copper in drinking water to reduce public health impacts associated with these constituents. The EPA has considered regulatory options to further improve the existing rule, including lead service line replacement, improving optimal corrosion control treatment requirements, consideration of a health-based benchmark, the potential role of point-of-use filters, clarifications or strengthening of tap sampling requirements, increased transparency, and public education requirements. However, the most significant upcoming change to the LCR will be the separation of the lead and copper rule into two separate regulations. These changes are not expected to affect the City, since there are no recorded lead or copper violations.

Per- and polyfluoroalkyl substances (PFAS) are a large group of synthetic fluorinated organic chemicals that are soluble, mobile, and recalcitrant to chemical and biological processes. The two most dominant groups of PFAS consist of perfluorooctanyl sulfonate (PFOS) and perfluorooctanoic acid (PFOA). PFOS and PFOA are human-made, fully fluorinated, organic compounds that are stable and resist typical environmental degradation processes, resulting in them building up in the environment. PFAS are manmade, chemicals that are heat, water, and lipid-resistant. Because of these qualities, they deter water, grease and oil, and are therefore used in many industrial applications, ranging from flame-retardants to stain-resistant carpets to Teflon® pans. Due to decades of ubiquitous use of these chemicals, PFAS are now detected throughout the environment in soil, air, water, household dust, and humans.

Elevated exposure to PFAS compounds (primarily by way of ingestion of drinking water) have been associated with developmental effects during pregnancy such as low infant birth weights and skeletal variations, effects on the immune system such as changes in antibody production and immunity, liver effects including tissue damage, cancer, and thyroid hormone disruption. In May 2016, EPA established drinking water health advisories of 70 parts per trillion (ppt) (0.07 micrograms per liter (µg/L)) for the combined concentrations of PFOS and PFOA. Above these levels, EPA recommends drinking water systems take steps to assess contamination, inform consumers, and limit exposure. Although the EPA has not issued an MCL for drinking water for PFOS and PFOA, several states have established drinking water and groundwater guidelines. Colorado has yet to establish these guidelines, however, the State has embarked on a sampling project in which utilities may volunteer to have their water sampled. The City participated in the

State sampling program in 2020 with the following results (PFOS 0.55 ppt, PFOS+PFOA 0.55 ppt, PFBS 0.56 ppt, PFHxA 0.84 ppt and a total PFAS of 1.95 ppt). The City potable supply is well below the EPA advisory level.

In addition, the State has also developed a PFAS Narrative Policy Work Group, whose goal is to develop policy by May of 2020 that will address PFAS contamination. The PFAS narrative policy will implement provisions in Regulations 31 and 41 and will describe how the department plans to implement the narrative provisions and the purpose of the work group is to gather input from stakeholders on the implementation efforts.

SECTION 4 – WATER TREATMENT FACILITY EVALUATION

The City's conventional water treatment plant treats raw water from the TR. The unit treatment processes include chemical addition, rapid mix with coagulation and flocculation, sedimentation, chlorination, mixed media filtration, and chlorine contact. Finished water is then pumped to the distribution system via high service pump stations. Solids handling consists of open pond storage of backwash and settled water treatment residuals which are intermittently pumped to the sanitary sewer. This section details each unit treatment process and includes process condition assessments and identification of performance limiting factors. A site plan is provided in Figure 7 and the process flow diagram is depicted in Figure 8.

The instrumentation and controls, electrical, and regulatory compliance of the WTP will also be evaluated. Each evaluation is based on discussions with City staff, field visits, recent construction projects, and existing reports. Capital improvement projects have been identified for each major system to address performance limiting factors and recommended improvements. The recommended capital improvement projects are discussed in Section 5.

EXISTING FACILITIES AND PROCESSES OVERVIEW

Existing facilities and processes include raw water conveyance and storage, pretreatment, clarification, filtration, disinfection, finished water pumping, and chemical systems. The capacity of each unit treatment process is listed in Table 11.

Table 11. Unit Treatment Process Capacity

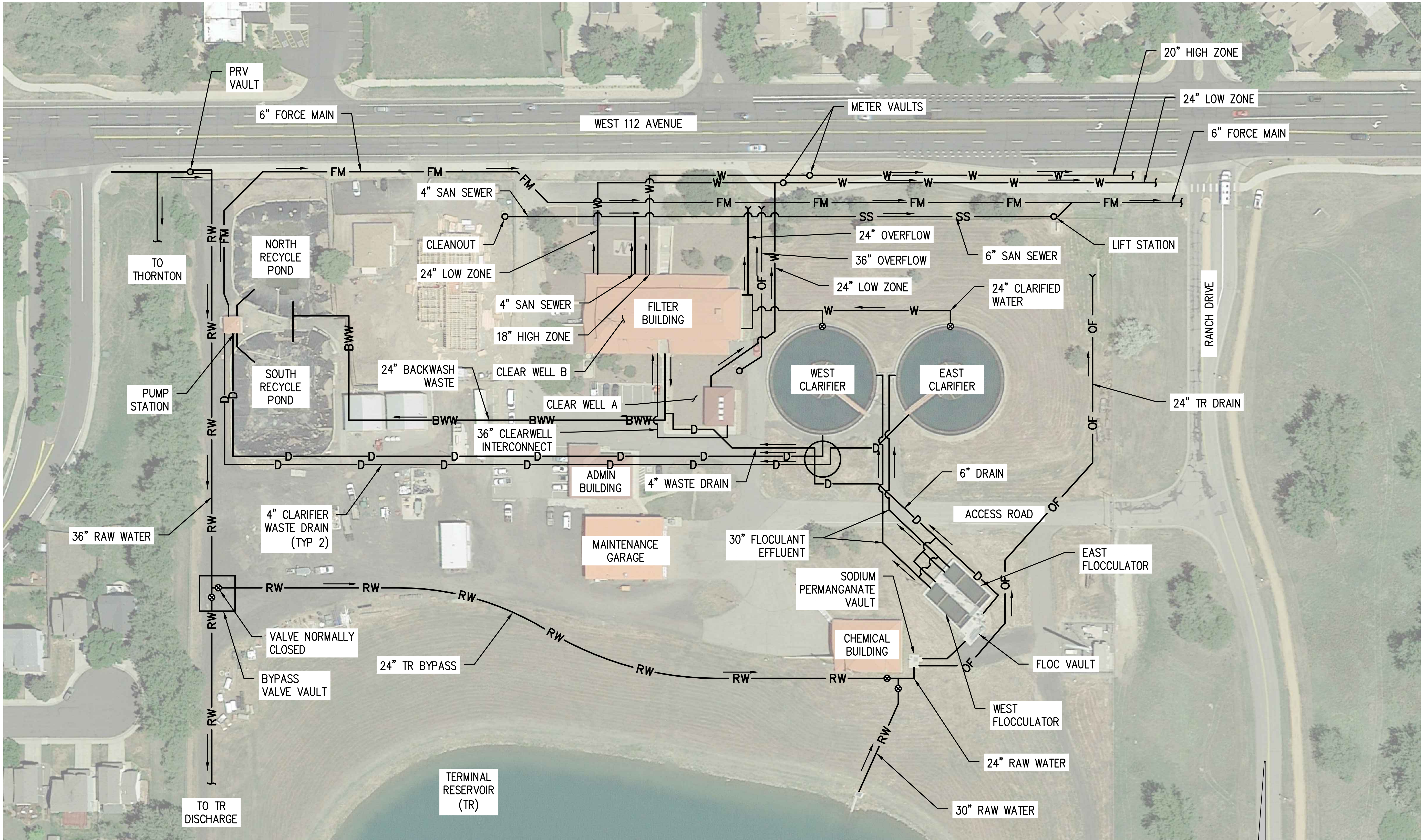
Process	Number of Units	Capacity Per Unit	Total Capacity
Raw Water Storage	1 reservoir	40 MG	40 MG
Rapid Mix	1 tank	14 MGD	14 MGD
Flocculation[1]	2 trains	6.2 MGD	12.4 MGD
Clarification[2]	2 basins	9.6 MGD	19.2 MGD
Filtration[3]	8 filters	2.5 MGD	20 MGD
Disinfection	Clearwell A	0.340 MG	0.884 MG
	Clearwell B	0.544 MG	
Low Zone High Service Pumps, CW B	4 pumps	3.3 MGD	31.1 MGD
High Zone High Service Pumps, CW B	4 pumps	2.0 MGD	
Low Zone High Service Pumps, CW A	3 pumps	3.3 MGD	

[1]Based on minimum detention time of 30 minutes and 2 ft freeboard

[2]Based on maximum surface loading rate of 0.7 gpm/ft^2

[3]Capacity per unit given by City – 5 gpm/sf

FIGURE 7 - WATER TREATMENT FACILITY SITE PLAN
CITY OF NORTHGLENN WATER MASTER PLAN UPDATE
AUGUST 2020



LEGEND

RW	RAW WATER	FW	FILTERED WATER
OF	OVERFLOW	BWW	BACKWASH WASTE
W	WATER	SS	SANITARY SEWER
D	DRAIN	FM	FORCE MAIN

N

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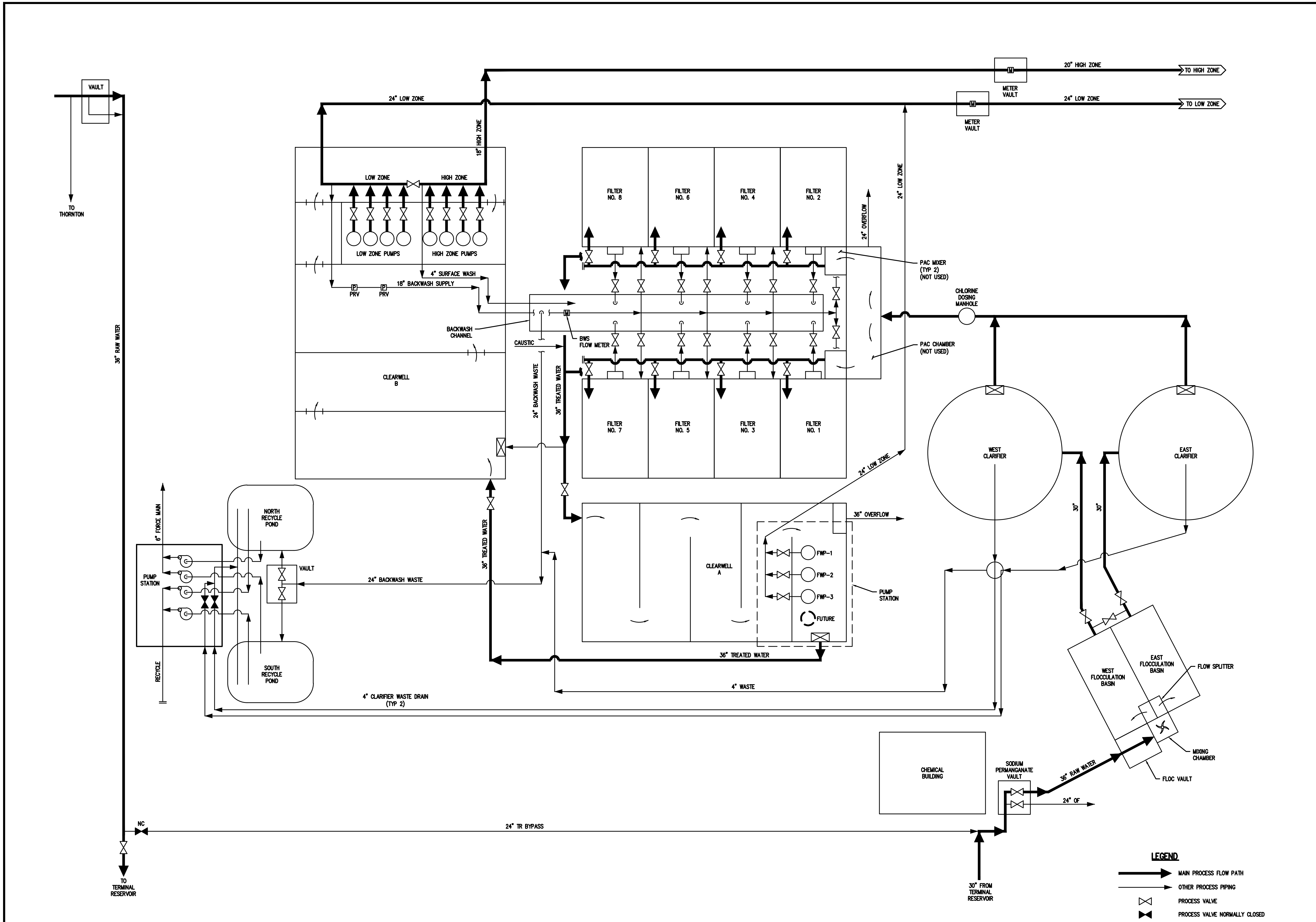

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FIGURE 8 - PROCESS FLOW DIAGRAM
CITY OF NORTHGLENN WATER MASTER PLAN UPDATE
AUGUST 2020

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RAW WATER

Raw water from Standley Lake is conveyed via a 48-inch transmission line aligned in 112th Avenue. Prior to reaching the WTP property, the main tees providing service to the City of Thornton to the south. At the northwest corner of the City's WTP property, the raw water transmission line enters a vault including PRVs, flowmeter and electrically actuated butterfly valves. The PRV vault is also equipped with a 12-inch bypass and 12-inch PRV. The PRVs reduce pressure from approximately 100 pounds per square inch (PSI) to approximately 30 PSI. The actuated butterfly valves are used to control flow and the water level in the TR. From the PRV vault, raw water is conveyed via a 36-inch main to the TR. Raw water can bypass the TR through a 24-inch reinforced concrete pipe connected to the sodium permanganate vault.

The capacity of the TR is 40 million gallons (MG). In addition to Standley Lake, raw water can be diverted from Farmer's Highline Canal (Canal) as an alternative source. The TR is used to store raw water for treatment as well as irrigation water for the local City park. An irrigation pump station is located on the southeast side of the TR and is equipped with a flow totalizer for measuring irrigation consumption. The TR level is maintained at 13 feet during the summer and 12 feet during the winter. Staff maintain a constant level in the TR through the SCADA setpoints

Raw water from the TR is conveyed to the sodium permanganate vault via a 30-inch DIP pipeline located at the northeast side of the TR. The TR outlet piping consists of a trash rack with 4-inch wide openings, emergency valves with hydraulic actuators, and a pressure transducer to measure the level of the TR. The pressure transducer is in the sodium permanganate vault downstream of the TR outlet.

CONDITION AND PERFORMANCE ASSESSMENT

The 48-inch raw water transmission line from Standley Lake is maintained annually by Staff. In 2020, several sections of the pipeline will be replaced. During this time, the Canal will be used to convey raw water from Standley Lake to the WTP. The Canal outlet structure was recently improved with a new flume and manual read. Half of the 36-inch raw water line from the PRV valve vault to the TR, located on the WTP property, was replaced in 2015 and the other half was replaced in 2019. The raw water line was installed with cathodic protection and is encased in concrete due to its vicinity to the dam toe. The 24-inch concrete bypass line, which was constructed in 1979, is original to the WTP and no major improvements have been done to the line.

The raw water mainline PRVs reduce over 60 psi of pressure and the City has expressed interest in replacing one of the PRVs with an in-line micro hydro turbine. A potential micro hydro unit is reviewed in Section 5.

LIMITING FACTORS

Operational staff have noted inconsistent readings from the TR level monitoring system. The pressure transducer is located within in the sodium permanganate vault and may be susceptible to interference due to dynamic losses in the water line. Installation of bubbler or pressure transducer at the TR outlet was reviewed with staff in the field.

Pretreatment

Pretreatment consists of chemical addition, rapid mix and flocculation. Sodium permanganate is injected into the 24-inch raw water line at the sodium permanganate vault prior to the floc vault. The addition of sodium permanganate is used to oxidize manganese for subsequent removal. From the sodium permanganate vault, the 24-inch line is expanded to a 36-inch line to allow for more contact time. Prior to entering the floc vault, the line is reduced back to 24-inch. In the floc vault, alum and coagulant aid are injected to enhance coagulation prior to rapid mix.

Rapid Mix

Rapid mix is used to quickly distribute the coagulant and polymer throughout the water for optimal coagulation. The rapid mix process consists of one concrete rapid mix chamber and one mechanical mixer. The current rapid mix chamber and mixer was installed in 2014. The basin has a volume of approximately 5,740 gallons at the high-water level. The design parameters for the rapid mix chamber are included in Table 12 below.

Table 12. Rapid Mix Chamber Design Parameters

Parameter	Rapid Mix Chamber
Quantity	1
Length (feet)	7.0
Width (feet)	7.0
Height (feet)	16.7
High Water Level Height (feet)	15.7
Operating Volume (gal)	5,740

The mixer is a 4-blade impeller with a vertical shaft agitator with a maximum rotation of 155 RPM and powered by a 40 HP motor with a VFD. Operation staff manually adjust the speed of the rapid mix to achieve optimum mixing. A summary of the design parameters for the mixer is included in Table 13 below.

Table 13. Mixer Design Parameters

Parameter	Mixer
Quantity	1
Motor Manufacturer	TECO Westinghouse
Motor Model Number	MAX-E2/841™
Power (hp)	40 w/ VFD
Motor RPM	1770
Frequency (Hz)	60
Nominal Efficiency (%)	94.1
Minimum Efficiency (%)	93
Impeller Manufacturer	Chemineer Inc.
Impellor Model Number	23GTP-40
Max Impellor RPM	155

Condition and Performance Assessment

Hydraulic detention time and velocity gradient were assessed to evaluate the performance of the rapid mix system. Typical detention time for rapid mix systems is 30 seconds for obtaining full mixing and dispersion of the chemicals. At the current peak day production of 8.7 MGD, the hydraulic detention time is 57 seconds and at the design capacity of 14 MGD, the hydraulic detention time is 35 seconds.

The velocity gradient is used to determine the intensity of the mixing system based on the power, or energy, that is applied to the water in the mixing chamber. Optimal velocity gradients range from 500 to $1{,}000^{s-1}$. Using the volume of water and imparted power, the rapid mix basin velocity gradient ranges are depicted in Table 14. Mixing speed or power imparted can be adjusted by staff with a VFD to maximize chemical dispersion and minimize over shearing the floc. The velocity gradient at both operating depths is well within the acceptable range.

Table 14. Rapid Mix Design Criteria

Parameter	Operating Depth with freeboard of 15.6 ft	Current Operating Depth of 16.23 ft	CDPHE Design Criteria
Hydraulic Detention Time @ 8.7 MGD	57 sec	59 sec	< 30 sec
Hydraulic Detention Time @ 14 MGD	35 sec	37 sec	< 30 sec
Velocity Gradient	877 sec^{-1}	861 sec^{-1}	500 – 1000 sec^{-1}
Capacity Based on Design Criteria			
Treatment Capacity	16.5 MGD	17.1 MGD	30 sec

The rapid mix chamber was newly constructed in 2014 and is in good condition. Operators have not noticed any performance issues since construction.

Limiting Factors

There is only one rapid mix chamber with one mixer. This is a single point of failure in the WTP. It is suggested that the City purchase a backup mixer to be stored on site. The WTP has a crane that will allow the replacement of the current mixer in the event of failure. A second option would be the installation of an inline mixer upstream of the rapid mix chamber for redundancy. Location and space for this static mixer maybe a challenge.

FLOCCULATION

The flocculation system consists of two treatment trains. Each train contains three stages divided by wooden baffles in an over/under configuration. The flocculation basins were constructed in 2014 and are uncovered concrete basins located to the east of the chemical building. Water from the rapid mix chamber enters a splitter box where flow is directed to each train. Flow to each basin is controlled by adjustable slide gates. The design rating for each train is 7.0 MGD with the ability to take one train out of service. Each train has a volume of approximately 116,000 gallons at the normal water level and contains baffling for three flocculation stages. The flocculation basin design parameters are included in Table 15.

Table 15. Flocculation Basin Design Parameters

Parameter	Flocculation Basin (each)
Quantity	2
Length (feet)	45
Width (feet)	22
Height (feet)	18
High Water Level Height (feet)	14.5
Operating Volume (gal)	116,000
Baffles	3
Water Flow Around Baffles	Over, under, over

The first two baffles are used to section the flocculation basin into three stages while the third baffle prevents short circuiting to the 30-inch outlet pipe. Horizontal shaft paddle wheel style flocculators are used to induce floc formation in the basins. Each stage contains a paddle wheel flocculator with a unique paddle and blade configuration to decrease the amount shear with each stage. The first stage has 4 paddles with 3 blades on each paddle. The second stage has 2 paddles with three blades on each paddle. The last stage has 2 paddles and 2 blades on each paddle. The rotational speed decreases with each stage. The motors for each stage are currently set at 1.97 RPM in Stage 1, 1.76 RPM in Stage 2, and 1.6 RPM in Stage 3. The motors for each stage decrease in power from 5 hp to 3 hp and finally 2 hp for the last stage of mixing. The paddles rotate in clockwise (CW) or counterclockwise (CCW) in an alternating pattern to prevent short circuiting between stages. The mixing design parameters for the motor and paddles are included in Table 16 below.

Table 16. Mixing Design Parameters

Parameter	Stage 1 Mixer	Stage 2 Mixer	Stage 3 Mixer
Quantity	2	2	2
Motor Manufacturer	Sew-Eurodrive Inc.	Sew-Eurodrive Inc.	Sew-Eurodrive Inc.
Motor Model Number	DRE100LC4/FG/DH	K127R77DRE100L4/DH	DRE 90L4-DH
Power (hp)	5	3	2
Frequency (Hz)	60	60	60
Nominal Efficiency (%)		87.5	85.5
RPM	1750	743	1740
Paddle RPM	1.03 – 3.00	0.94 – 3.16	0.55 – 3.16
Paddle Quantity	4	2	2
Blades Per Paddle	3	3	2
Paddle Diameter (feet)	12.4	12.4	12.4
Mixing Direction	CCW	CW	CCW

The flocculation system can be bypassed, with approval from CPDHE, via a 30-inch line connected at the rapid mix chamber. Each basin is equipped with a 6-inch DIP drain line to remove settled residuals. The 6-inch line ties into a 4-inch drain line that connects to the 24-inch backwash line conveying solids to the north and south ponds. Currently, the north pond is primary and in service.

Condition and Performance Assessment

The flocculation basins were constructed in 2014 along with the rapid mix chamber. Corrosion can be seen on the paddle axels and the bolts of the outlet structure. White build-up has been observed along the aluminum angles at the edges of the wood baffle walls below the surface of the water. The concrete below the water surface has also seen degradation which could be related to pH levels.

The drainpipe from the flocculation basins is prone to clogging and is hydraulically restricted where the 6-inch drain line from the flocculators ties into the 4-inch line that conveys flows to the waste ponds. The drains from the flocculation basins are used infrequently, however the restriction is a maintenance issue. Replacement of the existing 4-inch line has been discussed with staff.

Both flocculation basins must be online to feed water to both clarifiers due to improper flow splitting. The system functions appropriately when the West Flocculator is dedicated to the West Clarifier, and the East Flocculator is dedicated to the East Clarifier. However, with one flocculation basin out of service the flow does not split evenly to both clarifiers. Hydraulic improvements can be considered, but due to the low frequency of the issue, staff does not want to dedicate capital to implementing a remedy.

The flocculation basins were designed for a 7 MGD capacity for each train. At the time of the original design, a 20 to 30-minute detention time range was required per the State design criteria, which has since be updated to a 30-minute standard. Including the volume of the splitter box, the detention time per train is 24.2 minutes at a depth of 14.1 feet at 14 MGD, which meets the CDPHE standard in 2014. Using the updated 2017 State regulation and a 2 ft freeboard, the maximum flow rate through both basins while still achieving a 30 min detention time is approximately 12.4 MGD. The flocculator detention time design criteria are shown in Table 17.

Table 17. Flocculator Detention Time Design Criteria

Parameter	Flow Rate	Detention Time	CDPHE Design Criteria
Ex. Peak Day	8.7 MGD Total 4.3 MGD per Train	40.2 min	> 30 min
Buildout	14 MGD Total 7 MGD per Train	25 min	
Maximum Capacity	12.4 MGD Total 6.2 MGD per Train	30 min	

The velocity of the flocculated water through the two 30-inch pipes leaving the process is approximately 2.21 ft/s each at 7 MGD for a combined total of 14 MGD. The CDPHE requirement is greater than 0.5 ft/s and less than 1.5 ft/s. The maximum velocity of 1.5 ft/s can be reached at 9.5 MGD. These calculations do not account for turbulences and bends. At higher flows, the flocs formed are more likely to destabilize and break apart before entering the clarifiers. Table 18 shows the effluent velocities based on flow through both 30-inch pipes leaving the flocculator

Table 18. Flocculator Exit Velocity Design Criteria

Parameter	Pipe Diameter = 30 inches	CDPHE Design Criteria
Effluent Velocity @ 14 MGD	2.21 ft/s	> 0.5 ft/s and < 1.5 ft/s
Effluent Velocity @ 8.7 MGD	1.37 ft/s	> 0.5 ft/s and < 1.5 ft/s
Capacity Based on Design Criteria		
Maximum Flow Rate	9.52 MGD Total 4.76 MGD Through Each Pipe	Based on a < 1.5 ft/s effluent velocity
Minimum Flow Rate	3.17 MGD Total 1.59 MGD Through Each Pipe	Based on a > 0.5 ft/s effluent velocity

Limiting Factors

Although the design capacity of each flocculation train is intended to be 14 MGD, or 7 MGD per train, the existing basins cannot achieve the required minimum contact time of 30 minutes per train. In addition, the velocity of the effluent pipe from the flocculators exceeds the velocity set forth in the CDPHE design criteria.

The velocity of the flocculated water in the 30-inch pipes is within acceptable ranges for the current and projected treatment flows. JVA recommends that for the next design improvements for the WTP, the City requests a variance for the flocculation time and velocity during the preparation of the Basis of Design Report (BDR) to retain the 14 MGD capacity rating.

CLARIFICATION

The WTP has two uncovered concrete clarifiers that are gravity fed from the flocculation basins. Each clarifier has a volume of approximately 1.32 MG. Water flows into each clarifier through a 30-inch DIP line that feeds into the center of the clarifiers. A flocculation curtain surrounding the inlet is used to prevent short circuiting and decrease disturbance of settled solids. The clarified water flows from the center of the clarifiers to the effluent troughs. The effluent troughs discharge to a 24-inch steel outlet pipe. The settled solids are collected at the bottom of the clarifier which is sloped for solids removal. The clarifier, flocculation curtain, and effluent trough design parameters are summarized in Table 19.

Table 19. Clarifier Design Parameters

Parameter	Clarification Basin
Quantity	2
Diameter (feet)	110
Outer Diameter Height (feet)	17.6
Inner Diameter Height (feet)	22.3
Slope	1"/12"
Operating Volume (gal)	1,326,000
Flocculation Curtain	1
Flocculation Curtain Diameter (feet)	18.5
Flocculation Curtain Height (feet)	12
Effluent Outer Diameter (feet)	106
Effluent Inner Diameter (feet)	104
Effluent Trough Height (feet)	1.5

The settled solids are collected via a rotating sludge recirculating well and rotating rake containing blades with squeegees that are distributed along the rake. The solids are gathered at the center of the clarifiers prior to draining to the residuals ponds via two dedicated 4-inch waste lines. The sludge recirculating well in each clarifier are no longer used. The motors used for rotating the rake in the clarifiers are 0.75 hp. The rake and motor design parameters are summarized in Table 20 below.

Table 20. Clarifier Rake & Motor Design Parameters

Parameter	Mixer
Quantity	2
Motor Manufacturer	Sew-Eurodrive Inc.
Power (hp)	0.75
Frequency (Hz)	60
Rake Arm	2
Rake Length (feet)	50
Blades Per Arm	10

The two 4-inch waste lines combine at a manhole which includes two wasting valves. The valves open for 10 minutes every 120 minutes for each clarifier. The east clarifier wastes approximately 13,100 gallons of sludge per day, while the west clarifier wastes approximately 13,500 gallons per day. The waste sludge flow is approximately 109 GPM for the east clarifier and 113 GPM for the west clarifier.

Condition and Performance Assessment

The clarifier rake mechanism, troughs, and concrete were rehabilitated in early 2019, and small improvements were made in late September, early October of 2019. Rehabilitation included sandblasting and recoating the rank arms, skim coating the floors and replacement of the launders and weirs. A noted deficiency is the lack of infrastructure to release groundwater pressure when draining the clarifiers.

The surfaces of the clarifiers are prone to freezing in the winter due to cold temperatures and low flows. Currently, this does not hinder operations and further action is not needed.

The drive on the east clarifier was replaced in 2010 and the drive on the west clarifier was replaced in 2014. Maintenance staff have maintained the drives and they are in good condition.

A summary of the clarifier CDPHE Design Criteria and operating values is provided in Table 21. All values are per clarifier unless otherwise noted. The depth of the clarifiers is determined by the average of the HWL at the outer diameter and inner diameter.

Table 21. Clarifier Design Parameters

Parameter	8.7 MGD	14.0 MGD	CDPHE Design Criteria
Detention Time	7.27 hours	4.52 hours	> 4 hours
Surface Loading Rate	0.32 gpm/ft^2	0.51 gpm/ft^2	< 0.7 gpm/ft^2
Horizontal Velocity	0.002 ft/s	0.003 ft/s	< 0.5 ft/s
Outlet Flow Rate	12,941 gpd/ft	20,824 gpd/ft	< 20,000 gpd/ft
Capacity Based on Design Criteria			
Maximum Flow Rate	15.82 MGD Total 7.91 MGD Per Clarifier		Based on a detention time of 4 hours
	19.16 MGD Total 9.58 MGD Per Clarifier		Based on weir overflow rate of 0.7 gpm/ft^2
	13.45 MGD Total 6.72 MGD Per Clarifier		Based on an outlet flow rate of 20,000 gpd/ft

LIMITING FACTORS

During the 2017 Sanitary Survey, CDPHE staff inspected the clarifiers and observed that the effluent weirs on both clarifiers were uneven as seen from varying rates of flow at different locations around each clarifier. Uneven weirs can cause short circuiting within the clarifiers. Short circuiting can cause higher flow and increased horizontal velocity in one part of the clarifier which can cause currents that carry solids through the clarifier. The department recommended that the clarifiers be evaluated to determine how severe the short-circuiting is and what solutions are available for rehabilitation the clarifiers.

POWDER ACTIVATED CARBON CONTACT CHAMBER

A powder activated carbon (PAC) contact chamber is located after clarification and before filtration. The PAC chamber is no longer used to feed PAC but is used to monitor chlorine residual entering the filters. If needed, the PAC chamber volume can be incorporated for additional CT for disinfection. The condition of the PAC chamber is unknown. Table 22 shows the dimensions and capacity of the PAC chamber.

Table 22. PAC Design Parameters

Parameter	PAC Chamber Basin
Quantity	1
Length (feet)	22
Width (feet)	20.5
Height (feet)	15
High Water Level Height (feet)	14
Volume (gallons)	47,235*

*Volume is based on the high-water level height

FILTRATION

Settled water from the clarifiers is distributed onto conventional tri-media filters via a 14-inch influent off the 24-inch settled water pipeline. Each filter has an operating volume of approximately 35,400 gallons based on the high-water level. The water level in the filters is set to be maintained at 42 inches above the media by a modulating valve located before rapid mix. The filter media has a depth of 30 inches and is comprised of anthracite, silica sand, and garnet sand. The media is replaced approximately every 10 years. The original gravel underdrain system has been replaced with the Leopold® IMS® (Integral Media Support) cap in 2001. The City has filter to waste installed, but the system is not automated so it is rarely used. The filter design parameters are summarized in Table 23 below.

Table 23. Filter Design Parameters

Parameter	Filter
Quantity	8
Length (feet)	26
Width (feet)	13
Height (feet)	14.7
High Water Level Height (feet)	14
Operating Volume (gal)	35,400
Media Depth (inches)	30
Mixed Media	Gravel, Sand, Anthracite Coal, Granular Activated Carbon
Underdrain	Leopold IMS Cap
Filter-to-Waste	Yes

The filters are maintained through filter backwashes and surface wash. Typically, backwashes occur under three conditions; whichever occurs first: (1) 70 hours of filter run; (2) filter media head loss reaches 7.0 feet; or (3) filter effluent turbidity reaches 0.07 NTU. A surface wash is activated upon the onset of each backwash cycle for the agitation of surface trapped particles. Backwash supply water for the backwash and surface wash comes from the clearwells. The low zone water pressure is used to provide water for backwashes, while the high zone pressure is used for surface wash. Surface wash and backwash supply is delivered to each filter via 4-inch line and common 18-inch header, respectively. Backwash waste flows into two parallel backwash troughs per filter and networked with 18-inch and 24-inch drain piping. Surface wash, backwash supply and backwash waste cycles are manually controlled by the original filter consoles. The backwash design parameters are summarized in Table 24.

Table 24. Backwash Design Parameters

Parameter	Value
Backwash Volume[1] (gal)	58,401
Backwash Frequency (days)	2.5
Backwash Duration[2] (minutes)	15

[1]Average of backwash volumes from 2013 to 2019
[2]Assumption from HDR Master Plan

CONDITION AND PERFORMANCE ASSESSMENT

The City replaces the filter media every 10 years. Typically, the media for two filters is replaced in a given year. The most recent media replacement occurred in 2019 for the final two filters, and media replacement for the other filters will not be needed until 2025.

The chlorine smell is strong within the filter building and a fan constantly runs in the main filter gallery to circulate the air. There is no air circulation in the filter control room adjacent to the filter gallery. A new ventilation system is recommended to provide better air circulation.

The filter backwash and surface wash are controlled using the original filter control consoles (FCC) consisting of old relays and controls that are no longer supported by the manufacturer. Operation staff are interested in filter automation by integrating a filter building PLC with user interface and SCADA to allow for automated filter sequences and trending.

From 2009 to 2019, the average backwash waste per filter backwash has decreased from approximately 100,000 gallons per backwash to just below 60,000 gallons per backwash. Figure 9 shows the average backwash volumes per month. Water reductions can be achieved through the addition of filter-to-waste automation and an air scour system. Adding air scour to the filters will reduce the backwash flowrates and duration, and improve removal of particles from the filter beds, increasing filter run times.

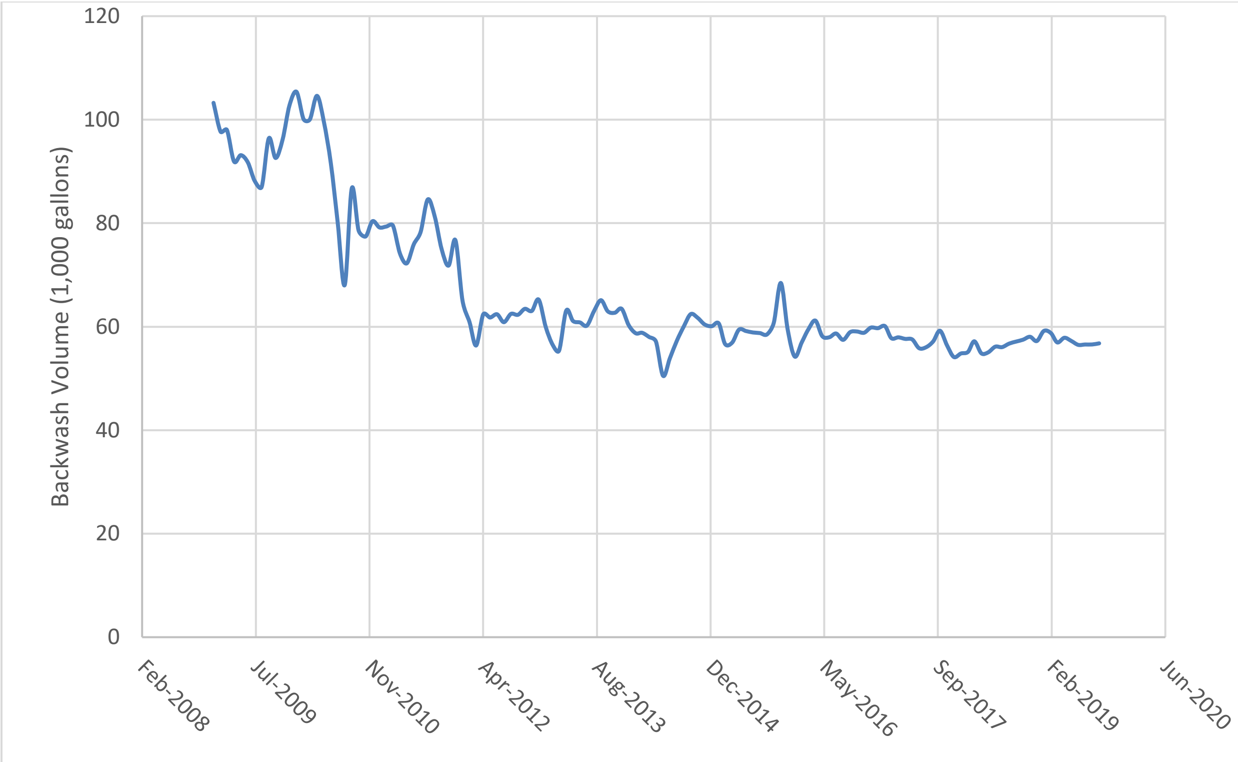

Figure 9. Monthly Average Volume Per Backwash

The butterfly valves and actuators in the filter gallery are approximately 40-years old and in need of replacement. The City made recent piping additions for filter to waste (FTW) and have budgeted funds this year to add valves / actuators to make final connections to include FTW in the filter backwash sequence. Most of the existing filter actuators are pneumatic and should be replaced with the best available technology and compatibility the City is most comfortable with. The two most common type of actuators are electric and pneumatic and both types have their distinct advantages. With exception to the recently installed FTW piping, there is significant corrosion to the remaining filter piping in the gallery, especially the piping closer to the ground level. All the pipes should be sandblasted and re-coated. Some corroded piping may require replacement upon further investigation.

The City measures filter bed expansion along the walls and have recorded approximately 13 to 14 percent during filter backwash. Due to the makeup of the filter, the bed expansion measuring device does not accurately measure the expansion in the middle of the filter. Leopold ®, the filter manufacturer, recommends an overall filter bed expansion of 20 to 30 percent. The City will continue profiling filter bed expansion in 2020.

The CDPHE design criteria and filter parameters are depicted in Table 25.

Table 25. Filter Design Criteria

Parameter	8.7 MGD	14.0 MGD	CDPHE Design Criteria
Hydraulic Loading Rate	2.23 gpm/ft^2	3.60 gpm/ft^2	< 5 gpm/ft^2
Influent Pipe Velocity	1.57 ft/s	2.53 ft/s	< 3.0 ft/s
L/d Ratio	1127		> 1000
Backwash Rate	11.5 gpm/ft^2		N/A
Capacity Based on Design Criteria			
Max Flow Rate	16.58 MGD Total 2.07 MGD Per Filter		Based on influent pipe velocity of 3.0 ft/s
Max Flow Rate	19.47 MGPD Total 2.43 MGPD Per Filter		Based on hydraulic loading rate of 5 gpm/ft2

LIMITING FACTORS

The hydraulic loading rate through each filter is 3.6 gpm/ft^2 at a 14 MGD capacity with all filters (8 filters) online. The hydraulic loading rate with 7 filters online is 4.11 gpm/ft^2. The influent pipe velocity for each filter is 2.53 ft/s at 14 MGD through 14-inch pipes. CDPHE sets a maximum velocity of 3 ft/s for the influent pipes. The recent addition of the FTW piping is not on-line as of current; however, the City has budgeted for this year to add the valves, actuators and integration into the filter backwash sequence. Filter improvement details are depicted in Figure 10

CDPHE requires the level of the backwash waste troughs to be located 12-inches above the maximum level of the top of expanded media during washing. The wash water troughs are located 14.4-inches above unexpanded media. If the media expands 20% during washing the elevation difference is only 8.4 ft.

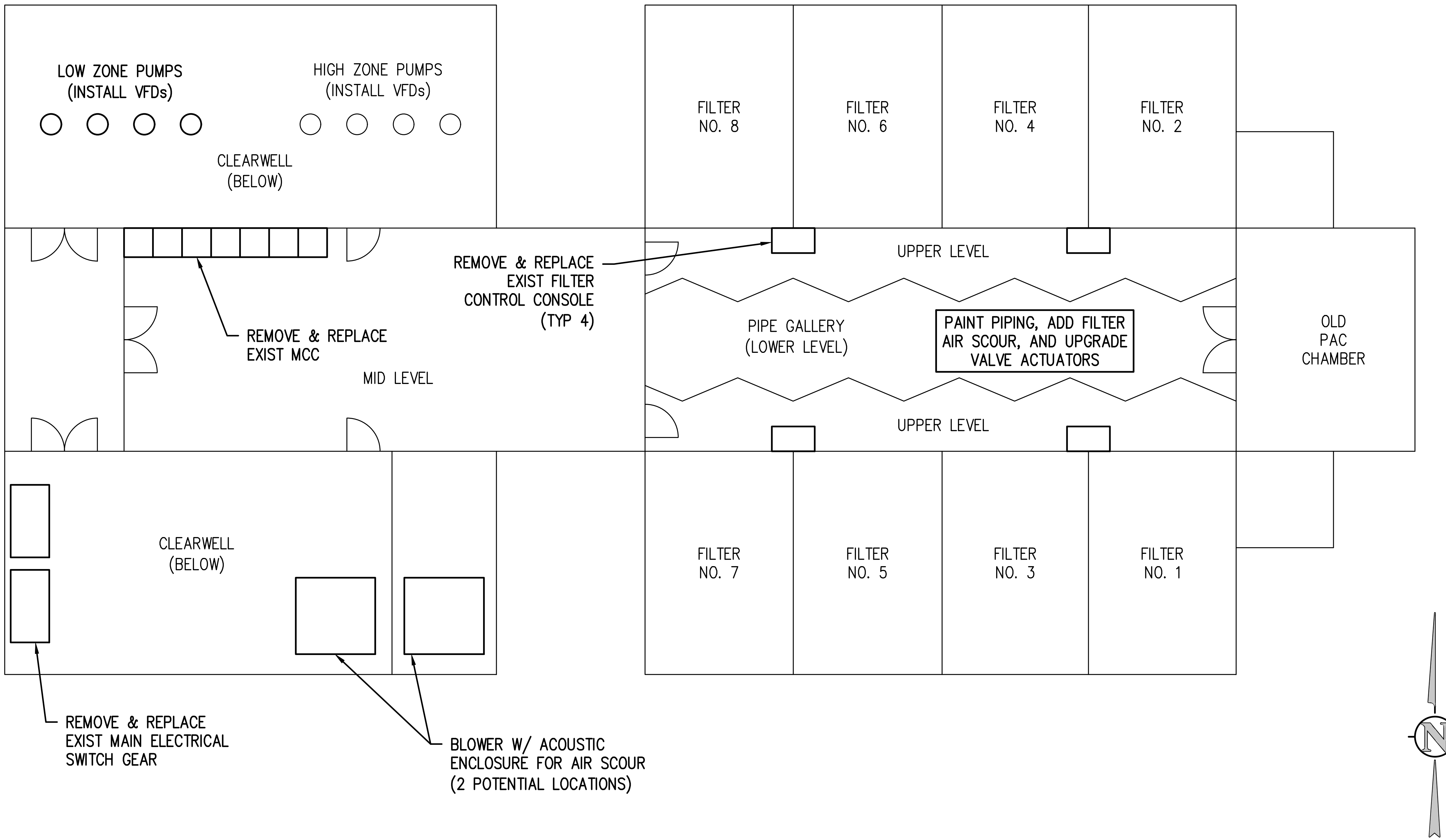

FIGURE 10 - FILTER BUILDING IMPROVEMENTS
CITY OF NORTHGLENN WATER MASTER PLAN UPDATE
AUGUST 2020

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Disinfection

Filtered water flows through two clearwells in series. The clearwells can operate independently, allowing for temporary shutdown of either clearwell during maintenance. Clearwell A and B are baffled and have respective volumes of 340,000 and 544,000 gallons at the high-water level for chlorine contact. Clearwell A was constructed in 2010 and clearwell B was constructed in 1979 and is original to the WTP. A summary of the clearwell design parameters are shown in Table 26.

Table 26. Clearwell Design Parameters

Parameter	Clearwell A	Clearwell B
Length (feet)	60	70
Width (feet)	56	77
Interior Height (feet)	14	14
High Water Level Height (feet)	13.5	13.5
Operating Volume (gal)	340,000	544,000
No. of Baffle Walls	4	4
Baffling Factor	0.5	0.5

Primary disinfection is provided by a sodium hypochlorite system. A 10 percent sodium hypochlorite solution is injected into the clarified water, prior to filtration, in a chlorine injection vault upstream of the PAC chamber. Chlorine feed rate pacing is based on the raw water flow rate and chlorine concentration from the analyzer information from the clearwell. The average chlorine feed rate set point averages 1.2 mg/L annually.

In the event that clearwell B is offline, water can be pumped to the low zone using the low zone pumps located in clearwell A. The clearwell A pumps have never been used and should be exercised.

Condition and Performance Assessment

The chlorine injection point is located in a manhole downstream of the clarifiers. At this location, the City can dose the combined clarifier flow or at the West Clarifier effluent line. Chlorine from the chemical building is pumped through 350 ft of 5/8-inch polyethylene line located in an 8-inch buried conduit to the injection point. The chlorine injection manhole has deteriorated and should be replaced.

Chlorine contact time (CT) was evaluated for both clearwells when operating independently and in series without the filter volume. The City targets a finished water pH of 8.2 which was used in the calculations. Winter temperatures were considered, representing a worst-case scenario, as required by the CDPHE design criteria. The winter water temperature was assumed to be 2.0 degrees Celsius and the free chlorine residual was assumed to be 1.2 mg/L. Using an 8.7 MGD production rate, the CT is summarized in Table 27. When used in series, the clearwells together can meet giardia and virus log removal requirements. However, at 8.7 MGD, clearwell A if operated independently, does not have enough capacity to meet giardia log removal requirements. Additional CT calcs are provided in Appendix A, which include the filter volume.

Table 27. Summary of Chlorine Contact Time at 8.7 MGD

	pH	Clearwells in Series	Clearwell A	Clearwell B	CDPHE Design Criteria
Giardia Inactivation (log)	8.2	3.4	2.83	3.03	3.0
Virus Inactivation (log)		33.3	12.7	20.4	4.0
Giardia Inactivation (log)	7.0	3.9	3.02	3.5	3.0
Virus Inactivation (log)		33.3	12.7	25.3	4.0

The maximum flow rate through both clearwells when operating independently and in series without the filter volume was assessed using a target pH of 8.2 and chlorine residual of 1.2 mg/L. Using the required giardia log removal rate as the control point and winter conditions, the maximum flow rate through the clearwells in series is 15 MGD, while the maximum flow rate through clearwells A and B separately is 5.8 and 9.2 MGD, respectively. With the filter volume include, the maximum flow rate through the clearwells in series is 21 MGD. A summary of this analysis is provided in Table 28.

Table 28. Summary of Maximum Flow Rate through Clearwells

Operation	Total Volume (gal)	Max Flow Rate (MGD)
Clearwells in Series and Filters	1,140,415	21.0
Clearwells in Series	892,615	15.0
Clearwell A	340,000	5.77
Clearwell B	544,000	9.24

LIMITING FACTORS

When operating in series, the clearwells have sufficient volume to meet giardia log removal requirements. The limitations of operating clearwell A independently can be overcome by reducing the water production rate, increasing the chlorine residual, and/or lowering the pH. There are two point for chlorine injection, with the valves for both located in a single manhole just north of the outdoor sedimentation basins. This represents a single point of failure which could result in loss of chlorine disinfection and potentially health and safety concerns. In addition, access to the chlorine injection is confined space and requires confined space permitting and trained personnel.

HIGH SERVICE PUMPS

The distribution system is divided into the low zone, intermediate zone, and high zone. At the WTP, finished water can be pumped either to the low zone and/or high zone. The typical operation scheme is to pump all treated water to the low zone storage tanks and utilize the system booster station to convey finished water to the high zone. Several intermediate zones are fed by both the high and low zone pumps. These intermediate zones are typically back fed through a PRV from the high zone.

The WTP has four low zone and four high zone high service pumps located after clearwell B, and three low zone pumps located after clearwell A. Clearwell B pumps are original to the Plant while the clearwell A pumps were installed in 2010. None of the pumps are operated with

VFDs. The clearwell B high service pump design criteria are provided in Table 29 and the clearwell A pumps are depicted in Table 30.

Table 29. Clearwell B Low Zone and High Zone High Service Pumps

Process	Low Zone Pumps	High Zone Pumps
Quantity	4	4
Manufacturer	Worthington Vertical Pump Co.	Worthington Vertical Pump Co.
Type	Vertical Turbine	Vertical Turbine
Design Capacity	2,310 gpm	1,390
Motor HP	150 hp	150 hp
TDH	162 feet	300 feet

Table 30. Clearwell A Low Zone High Service Pumps

Process	Number of Units
Quantity	3
Manufacturer	Mid-America Pump
Type	Vertical Turbine
Design Capacity	2,300 gpm
Motor HP	125 hp
TDH	167 feet

Condition and Performance Assessment

The clearwell B low zone and high zone high service pumps are original to the WTP and do not include VFDs. VFDs were purchased for the clearwell B low zone pumps in 2018 and are unusable because of harmonic issues. A pumping system evaluation should be conducted to assess the pump conditions and review options to remedy the vibration/harmonic issues. Assuming the vibration issue can be rectified, the VFDs should be subsequently installed.

The low zone pumps from Clearwell A have not been used since installation. The City plans on operating the pumps in 2020 to flush the system and test performance.

Limiting Factors

The WTP has adequate finished water, backwash supply and surface wash pumping capacity.

Chemical Feed and Storage Systems

The sodium permanganate, alum, polymer, sodium hydroxide, and sodium hypochlorite systems are located within a single chemical building. Sodium hypochlorite and sodium permanganate are each stored in separate rooms. Alum, polymer, and sodium hydroxide share a common room. A watering softening system is used to provide carrier water for polymer, sodium hypochlorite, and sodium hydroxide chemical feed systems. The water softening unit was recently replaced in 2019 and includes two resin tanks, a brine tank, and a controller.

During a site visit in September of 2019, JVA recorded chemical dosing rates. The chemical dosing rates on that day are shown in Table 31 and were used to assess the storage capacity for each chemical system.

Table 31. Chemical Dosing Rates, September 2019

Chemical	Dose (mg/L)
Sodium Permanganate	1.2
Alum	17
Polymer (coagulant aid)	2.75
Sodium Hydroxide	3.88
Sodium Hypochlorite	2.0

SODIUM PERMANGANATE

One peristaltic metering pump feeds the sodium permanganate solution through 5/8-inch polyethylene (PE) tubing for injection into the raw water. Design parameters for the existing sodium permanganate chemical feed and pumps are provided in Table 32. Operations staff manually set the sodium permanganate chemical feed pump to feed at a constant rate of 1.2 mg/L of solution. This dosing rate remains constant through the entire year with no adjustments based on seasonal or water quality changes.

Sodium permanganate is stored in a separate room within the chemical building. 20-percent concentrated sodium permanganate is delivered to the WTP in liquid form every 6 weeks. Concentrated sodium permanganate is pumped from a 325-gallon chemical storage tank to a 325-gallon day tank using a chemical transfer pump. Potable house water is added to dilute the concentrated chemical to create a 10-percent solution. The solution tank is equipped with an ultrasonic level detector. No mixer is present in the solution tank. A concrete curb in the sodium permanganate chemical feed room provides secondary containment. The containment curb is approximately 12 feet long by 10 feet wide and 1.5 feet tall. This is approximately 1,346 gallons of storage, sufficient for the 325-gallon tanks.

Table 32. Sodium Permanganate Chemical Feed and Storage Systems

Chemical Feed Pump	
Quantity	1
Type	Peristaltic
Manufacturer	Blue-White
Model Number	Flex Pro M-324-SNF
Maximum Flow	7.9 GPH
Dosing Rate	1.2 mg/L
Maximum Pressure	125 PSI
Maximum Strokes per Minute	125 RPM
Flow Control	4-20 mA
Turn Down Ratio	10,000:1
Head/ Fittings:	PVDF
Valve Balls	Ceramic

Valve/ Seat/ O-ring	TFE/P	
Connections	3/8 inch OD x ¼ inch ID	
Power Supply	115V/60HZ, US plug	
Chemical Transfer Pump		
Quantity	1	
Type	Horizontal end suction centrifugal	
Manufacturer	Finish Thompson	
Design Capacity	0.5 to 26 gpm	
Motor HP	1/3 hp	
Storage		
	Storage Tank	Day Tank
No. of Storage Tanks	1	1
Storage Tank Volume	325 gallons	325 gallons
Manufacturer	Polyprocessing	Polyprocessing
Material	XLHDPE	XLHDPE
Storage Tank Diameter	4 ft	4 ft
Storage Tank Height	4 ft, 7.5 in	4 ft, 7.5 in

ALUM

Alum is injected into the 24-inch raw water immediately upstream of the rapid mix chamber in the floc vault. The addition of Alum is to enhance coagulation. A single peristaltic metering pump feeds Alum through 5/8-inch PE tubing and injects it into the raw water pipe. Design criteria for the alum feed pumps is provided in Table 33. Operations staff flow pace and manually set the alum chemical feed pumps based on the raw water quality and the dosing rate ranges from 12 to 25 mg/L.

Alum is stored in the common area of the chemical building. Alum is delivered to the WTP in liquid form at an average of about every 5 weeks. Alum is stored in a 12,000-gallon chemical storage tank. The tank is refilled approximately every 4 weeks in the summer and every 8 weeks during the winter. Secondary containment is not provided for the alum chemical storage tank.

Table 33. Alum Chemical Feed and Storage Systems

Chemical Feed Pump	
Quantity	1
Type	Peristaltic
Manufacturer	Blue-White
Model	Flex Pro
Maximum Flow	7.9 GPH
Dosing Rate	17.0 mg/L
Maximum Pressure	125 PSI
Maximum Strokes per Minute	125 RPM
Flow Control	4-20 mA
Turn Down Ratio	10,000:1
Head/ Fittings:	PVDF
Valve Balls	Ceramic
Valve/ Seat/ O-ring	TFE/P
Power Supply	115V/60HZ, US plug
Storage	
No. of Storage Tanks	1
Storage Tank Volume	12,000 gallons
Manufacturer	Raven
Material	Fiberglass
Storage Tank Diameter	12 ft
Storage Tank Height	22 ft

POLYMER

A cationic liquid polymer called CAT-FLOC 8103 Plus (polymer) is injected into the 24-inch raw water with a static mixer immediately after the alum injection point and prior to rapid mix. The addition of polymer is to aid the coagulation process. A single peristaltic metering pumps feeds the polymer through 1-inch poly vinyl tubing with softened chase water prior to injection into the raw water pipe. A static mixer is located at the chase water injection point. Design criteria for the existing polymer feed system is provided in Table 34. Operations staff flow pace and manually set the polymer chemical feed pumps based on raw water quality. An average dosing rate of 2.75 mg/L of solution was assumed.

The polymer storage tank is located in the common area of the chemical building. Polymer is delivered to the WTP in liquid form every 8 weeks and is stored in a 300-gallon chemical storage tank. The tank is refilled every 6 weeks in the summer and every 3 months in the winter. Secondary containment is not provided for the polymer chemical storage tank.

Table 34. Polymer Chemical Feed and Storage Systems

Chemical Feed Pump	
Quantity	1
Type	Peristaltic
Manufacturer	Blue-White
Model Number	Flex Pro M-324-SNF
Maximum Flow	7.9 GPH
Dosing Rate	2.75 mg/L
Maximum Pressure	125 PSI
Maximum Strokes per Minute	125 RPM
Flow Control	4-20 mA
Turn Down Ratio	10,000:1
Head/ Fittings:	PVDF
Valve Balls	Ceramic
Valve/ Seat/ O-ring	TFE/P
Connections	3/8 inch OD x 1/4 inch ID
Power Supply	115V/60HZ, US plug
Storage	
No. of Storage Tanks	1
Storage Tank Volume	300 gallons
Manufacturer	Rotation Molding Inc. (RMI)
Storage Tank Diameter	4 ft
Storage Tank Height	3.8 ft

Sodium Hydroxide

Sodium hydroxide (caustic) is injected into the 36-inch filter effluent header to target a pH to 8.2. A single Blue-White peristaltic metering pump feeds the caustic solution through 5/8-inch PE tubing with softened chase water prior to injection into the raw water pipe. Design criteria for the existing chemical feed and transfer pumps is provided in Table 35. Operations staff manually set the sodium hydroxide chemical feed pump to feed at a constant rate of 3.12 mg/L of solution.

Caustic tanks are located in the common area of the chemical building. Caustic is delivered to the WTP in liquid form every 10 weeks and is stored in three 1,000-gallon chemical storage tanks. The tanks are refilled every 4 weeks in the summer and every 3 months in the winter. A containment curb is provided for the caustic storage tank area.

Table 35. Sodium Hydroxide Chemical Feed and Storage Systems

Chemical Feed Pump	
Quantity	1
Type	Peristaltic
Manufacturer	Blue-White
Model Number	Flex Pro M-324-SNF
Maximum Flow	7.9 GPH
Dosing Rate	3.12 mg/L
Maximum Pressure	125 PSI
Maximum Strokes per Minute	125 RPM
Flow Control	4-20 mA
Turn Down Ratio	10,000:1
Head/ Fittings:	PVDF
Valve Balls	Ceramic
Valve/ Seat/ O-ring	TFE/P
Connections	3/8 inch OD x 1/4 inch ID
Power Supply	115V/60HZ, US plug
Storage	
No. of Storage Tanks	3
Storage Tank Volume	1,200
Manufacturer	Chem-Tainer
Storage Tank Diameter	6 ft
Storage Tank Height	12-ft

SODIUM HYPOCHLORITE

The sodium hypochlorite system was installed in 2004 and improved in 2020. The chemical feed system includes two chemical storage tanks, two chemical feed pumps, a flow meter for softened chase water, and a control loop to flow pace the chlorine feed. Improvements were recently made to the system including the addition of a back-pressure valves, draw down columns, new valving, a pressure gauge and a rotometer for the carrier water. The chlorine feed rate set point averages 1.2 mg/L annually. Design criteria for the existing chemical feed and transfer pumps is provided in Table 36.

Sodium hypochlorite is stored in a separate room within the chemical building. Liquid, 10-percent sodium hypochlorite is delivered to the WTP every 7 weeks and is stored in two 3,850-gallon chemical storage tanks. The tanks are refilled approximately every 4 weeks in the summer and every 8 weeks in the winter. A containment trough is provided for the tank area. The east tank was relined in the spring of 2020 and the west tank is scheduled for relining in the fall of 2020.

Table 36. Disinfection System Design Parameters

Chemical Feed Pump	
Quantity	2
Type	Peristaltic
Manufacturer	Blue-White
Max Flow Rate	7.9 gpm
Max Dosing Rate	2.0 mg/L
Solution Concentration	10 Percent
Turn Down Ratio	10,000:1
Required Chlorine Residuals	1.2 mg/L
Storage	
No. of Storage Tanks	2
Storage Tank Volume	3,850
Manufacturer	MTE Design Tanks
Material	Hetron FRP
Storage Tank Diameter	7.5-ft
Storage Tank Height	12.3-ft

CONDITION AND PERFORMANCE ASSESSMENT

A general arrangement of the chemical storage building is provided in Figure 11. Sodium hypochlorite, alum, and caustic are corrosive chemicals. The International Building Code (IBC) lists hazardous (H) occupancy compliance when greater than 500-gallons of corrosive chemical is stored within a single room. H-occupancy requirements can include, but are not limited to, continuous ventilation, fire sprinklers, and fire rated construction. The local fire jurisdiction or building department interprets and enforces the IBC family of codes.

Operational staff have noted several safety concerns in regard to the chemical storage building. The emergency shower and eyewash located in the main chemical storage room is not heated. There is no emergency shower or eyewash located in the sodium permanganate storage room. The EPA recommends incompatible chemicals not be stored within the same area (EPA 816-F-09-002). The EPA classifies alum and polymers into Group III Salts and Polymers and sodium hydroxide into Group II Bases. Currently the caustic is stored within the same room as the alum and polymer. If any or all tanks spill, the chemicals will spill to a common pit which drains to the waste ponds. From the ponds, it will be pumped to the sanitary sewer collection system, where it could affect the treatment processes at the wastewater plant. Secondary containment for each chemical is needed to prevent this hazard.

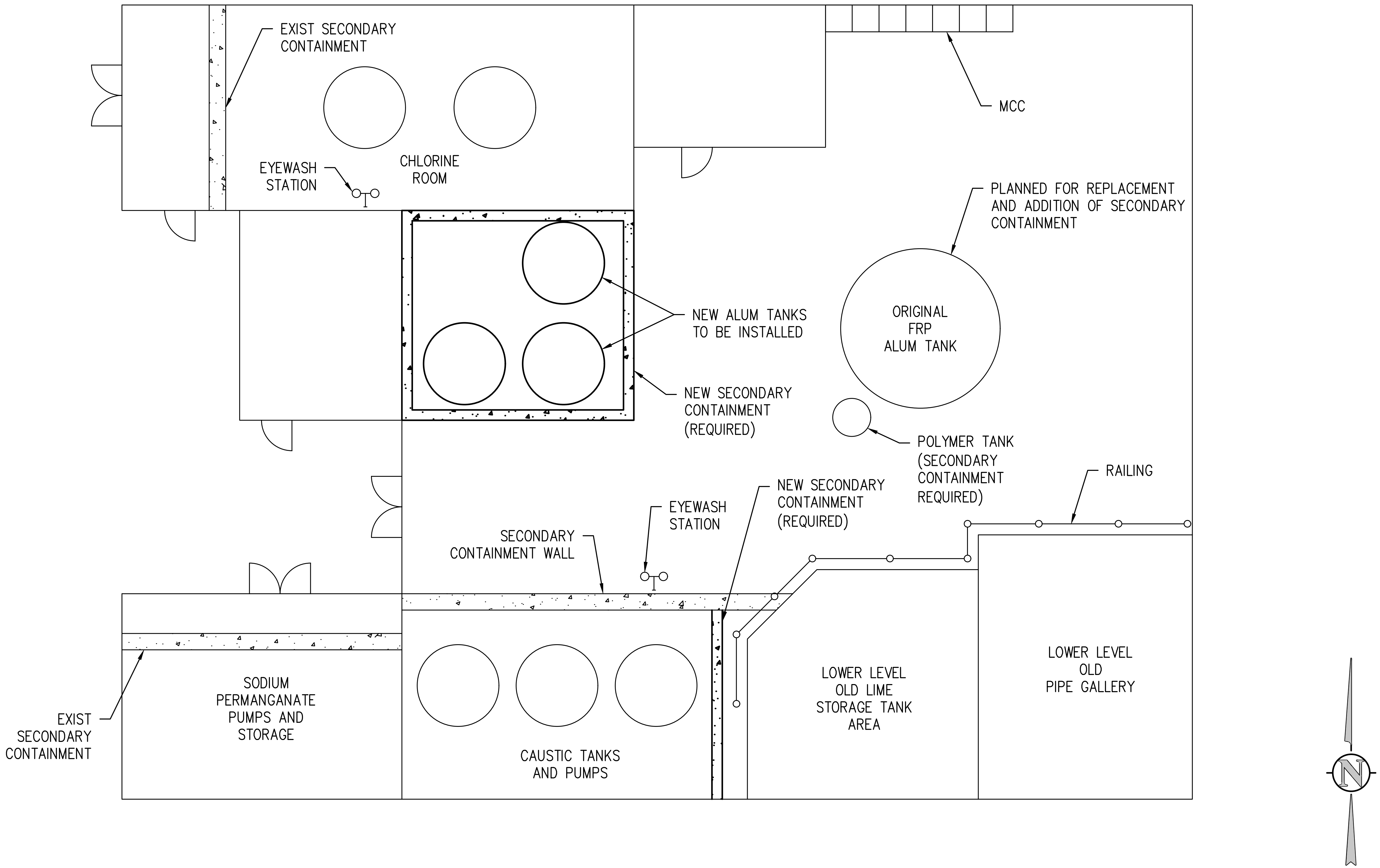

FIGURE 11 - CHEMICAL BUILDING IMPROVEMENTS
CITY OF NORTHGLENN WATER MASTER PLAN UPDATE
AUGUST 2020

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Sodium Permanganate

The sodium permanganate chemical storage and feed system was installed in 2014 and is in good condition. The sodium permanganate dosing rate, which is set at 1.2 mg/L, has not been changed since 2014. The City has a probe which can be used to monitor oxidation reduction potential (ORP), however, it is not currently in use. ORP readings can vary based on varying background water quality. An alternative would be to install a total chlorine monitor prior to chlorination. The output would be a residual permanganate concentration and a more reliable reference for dosing sodium permanganate. Based on the current chemical dosing rate of 1.2 mg/L, the peak day and future storage requirements are provided in Table 37.

Table 37. Sodium Permanganate Storage Requirements, 20% Solution

Flow Rate (MGD)	30-Days of Storage (gal)	15-Days of Storage (gal)
8.7	1,340	670
10	1,540	770
14	2,150	1,080

Alum

The Alum dosing rate is set manually. There is a streaming current monitor (SCM) in the floc vault which can be used for alum dosing adjustments. The existing SCM, which was recently installed, is functional but does not provide consistent information. The SCM, when properly calibrated, can be a useful tool to maximize coagulant efficiency. Typically, following rapid mix, alum needs approximately 30 seconds to a minute of contact time prior to measuring relative charge with the SCM. It is recommended that the City conduct bench or full-scale testing to determine the best location to sample for SCM.

The Alum chemical storage tank is original to the Plant and is being replaced with six 2,000-gallon storage tanks. This improvement will take place in the fall of 2020. No secondary containment is provided for the alum tank storage area but is needed. Based on the current chemical dosing rate of 17.0 mg/L, the peak day and future storage requirements are provided in Table 38.

Table 38. Alum Storage Requirements, 58% Solution

Flow Rate (MGD)	30-Days of Storage (gal)	15-Days of Storage (gal)
8.7	5,750	2,875
10	6,620	3,310
14	9,270	4,635

Polymer

The polymer feed system is in good condition; however, it does not have secondary containment. Based on the current chemical dosing rate of 2.75 mg/L, the peak day and future storage requirements are provided in Table 39.

Table 39. Polymer Storage Requirements, 20% Solution

Flow Rate (MGD)	30-Days of Storage (gal)	15-Days of Storage (gal)
8.7	3,500	1,750
10	4,030	2,015
14	5,645	2,823

SODIUM HYDROXIDE

The caustic storage tanks were installed in 2009 and are in need of replacement. The tank manufacturer, (Chem-Tainer) suggests a lifespan of 8 to 10 years and the existing tanks are now 11 years old. Removing and replacing the tanks will be difficult due to their size. Based on the current chemical dosing rate of 3.88 mg/L, the peak day and future storage requirements are provided Table 40.

Table 40. Sodium Hydroxide Storage Requirements, 32% Solution

Flow Rate (MGD)	30-Days of Storage (gal)	15-Days of Storage (gal)
8.7	3,120	1,560
10	3,590	1,795
14	5,030	2,515

SODIUM HYPOCHLORITE

The chlorine chemical storage tank is over 16 years old and is in need of replacement. One tank was relined in the Spring of 2020 and the other tank is scheduled to be relined in 2021. The City installed a new chlorine feed pump skid with two new chlorine feed pumps in February of 2020. Assuming a maximum dosing rate of 2.0 mg/L, the existing sodium hypochlorite storage system has 28 days of storage at 7 MGD and 14 days of storage at 14 MGD.

LIMITING FACTORS

The sodium permanganate, alum, polymer, chlorine and caustic feed systems all use the same model of chemical feed pump with interchangeable parts which allows for flexibility and redundancy. However, the WTP only has a single shelf spare to serve as a redundant pump for all systems. CDPHE Design Criteria requires that standby units for chemical feeders must be provided.

There is no secondary containment for the alum and polymer which will need to be added for meeting CDPHE design criteria and regulations.

A single shelf spare pump is available to serve as a backup pump for all chemical feed systems. Chlorine addition is the only system with a fully redundant feed pump. This is problematic, as chemical systems for pH control must be fully redundant, per CDPHE Design Criteria.

RESIDUAL SOLIDS HANDLING

The solids handling system consists of two recycle ponds (north and south) and a pump station. Filter backwash water, solids from the clarifiers and plant drains all flow by gravity to the north pond, which is hydraulically connected to the south pond. Based on the as-built drawings, at a depth of 8 ft the volume of each pond is approximately 300,000 gallons. Originally decanted water in the south pond could be recycled to the TR via a single 30-hp recycle pump, however, the WTP recently capped this line and is no longer active to recycle back to the TR. The water/solids mixture from the north pond is currently pumped to the sanitary sewer collection system via two 25-hp sludge pumps. The design criteria for the solids and recycle pumps are listed in Tables 41 and 42, respectively.

Table 41. Solids Pump Design Criteria

Process	Number of Units
Quantity	2
Manufacturer	Gorman Rupp
Design Capacity	340 gpm
Motor HP	25

Table 42. Recycle Pump Design Criteria (not currently used)

Process	Number of Units
Quantity	1
Manufacturer	Gorman Rupp
Design Capacity	425 gpm
Motor HP	30

CONDITION AND PERFORMANCE ASSESSMENT

The clarifiers produce approximately 27,000 gallons of solids per day. This production rate is relatively constant and does not fluctuate throughout the year based on the current plant operations. Backwash waste volumes, however, fluctuate seasonally as raw water flow rates increase and decrease throughout the year. Average summer backwash volumes range from 75,000 to 170,000 gallons per day and winter backwash volumes range from 42,000 to 82,000 gallons per day. The backwash waste stream is far more dilute than the clarifier waste stream which is estimated between 0.1 and 0.2 percent solids.

Residual production rates were estimated based on turbidity removal and chemical dosing rates. Assuming an average raw water turbidity value 3.0 NTU, estimated residual solids production rates are depicted in Table 43. An industry standard conversion factor of 1.25 mg TSS per NTU was used for the estimate. In addition, to account for the addition of alum, sodium permanganate, and polymer, it was assumed that 0.45 kilogram (kg) of dry sludge is produced for every kg of chemical added. Supporting calculations for the residual solids estimates are provided in Appendix A.

Table 43. Residual Solids Production Estimates – Current Condition – Peak Day 8.7 MGD

Winter/Summer	Average Production Rate (MGD)	Residuals Production (lbs/day)
Winter	2.5	400
Summer	6.2	1,000

LIMITING FACTORS

The ponds do not provide for any consistent settling of particles withdrawn from the treatment processes, so decant water is not high quality. Wastewater disposal is currently managed by pumping all the water to the sanitary sewer collection system. This practice wastes water that might otherwise be recycled. Water, however, is not currently recycled to the TR due to a history of poor settling and water quality in the south pond. All the solids are pumped to the forcemain from Lift Station D which was constructed to serve an adjacent subdivision. The combination of the two sources of flow have caused capacity restrictions for Lift Station D, forcemain and connecting gravity sanitary sewer. In addition, the weed barriers on both ponds are visibly damaged and are in need of repair. In general, the ponds are in very poor condition and need to be rehabilitated to effectively serve as backwash settling and recycle basins. Improvements to the residuals handling system will be required to reduce the volume of water and residuals sent to the sewer system.

INSTRUMENTATION AND CONTROLS

WTP CONTROL SYSTEM

The Northglenn WTP control system is comprised of Allen-Bradley PLCs and an Intellution iFix 6.0 SCADA HMI software with stand-alone Historian server. This includes a redundant iFix system with active fail-over. They currently have a combination of Micologix, Compact Logix, and Control logix PLC's.

There are two (2) auto dialers for the water treatment process. These are located at the High Zone Tank and Booster pump station. These are the only systems that have an active call out function. The plant alarming is on the operator's screens. The current water plant configuration allows for alarm notification on SCADA and an audible alarm located at the operator's station, however, operators are single coverage and alarms do not go to the operator's phone.

The existing filter control consoles in the WTP have had numerous modifications over the years. They are mostly a manual control panel for the individual filters.

BOOSTER PUMP STATION CONTROL SYSTEM

The booster pump station has a local Allen Bradley MicroLogix 1400 PLC that controls and monitors the station and gathers up all the signals to be transmitted to the WTP. The station communicates to the WTP over private broadband radios on a 5.8GHz spectrum.

CONDITION AND PERFORMANCE ASSESSMENT

The filter control consoles in the plant should be completely replaced. When replaced each control console should be arranged to have a PLC in the console to control the filters associated with that console. This way, there is redundancy built into the filter controls. The automation of the backwash system would be added at this time.

LIMITING FACTORS

The filter controls limit the operation of the filter and backwashes. With new controls, new programs may be developed for filter runs and backwashes that allow the operators to optimize the process. Redundancy can also be added to the controls.

ELECTRICAL POWER

WTP ELECTRICAL

The WTP has an existing 480 volt, 2000A electric service entrance Main Switchgear. This Main Switchgear is connected to a switchboard which in turn feeds the other power distribution equipment in the plant. All the service entrance equipment, except for the newer automatic transfer switch (ATS), appear to be original plant equipment.

BOOSTER PUMP STATION ELECTRICAL

The booster pump station is fed by EXCEL Energy with 480 volts, 400A, three phase power. This power connects to a main fused disconnect in the existing MCC located inside the station. At some time after the initial installation a generator and automatic transfer switch (ATS) were added to the station. This installation modified the MCC by taking power off the main MCC fused disconnect and routed cables outside the station to the ATS and then back from the ATS to the bussing in the MCC. This is how the MCC is today.

Currently this MCC only has breakers (no starters) feeding the various loads within the station.

CONDITION AND PERFORMANCE ASSESSMENT

WTP ELECTRICAL

The electrical service entrance equipment at the WTP is well past its safe life expectancy. It would be important to replace this equipment under a controlled planned scenario. This equipment creates a single point of failure that would completely shutdown the WTP. Due to the critical nature of this equipment it is important that all the electrical service entrance equipment be upgraded while it is still operational. Then it can be replaced in an orderly manner and not in an emergency. The existing breakers are past their intended life cycle. Due to the age and lack of maintenance on the equipment, it would not be possible to guarantee that the breakers in the equipment would trip as they are intended to trip.

Even in a planned scenario, the replacement of the electrical equipment will be a severe disruption to the plant. This will need to be well planned and well executed. The new gear would have to be procured, tested, and on site before the switchover could even start. Then several portable generators could be connected to the necessary loads at the plant, in order to keep the plant operational. It will take several days to remove the existing gear and install the new gear.

The motor control center (MCC) with across-the-line starters shares similar concerns with service entrance equipment. Due to the critical function of the High Zone and Low Zone pumps, it is essential this equipment be upgraded while operational just as the service entrance electrical gear. It is recommended that the across-the-line starters be replace with either reduced voltage solid state soft starters (RVSS) or variable frequency drives (VFD), as currently stored. Both of these types of motor controllers will reduce the electrical hammer on the electrical gear.

The replacement of this MCC will need proper coordination to transition to a new MCC.

Booster Pump Station Electrical

The booster pump station MCC is old and has been modified too many times to be safe. It is also not supported by any manufacturer. This MCC should be replaced with a Panelboard. The MCC no longer has any starters in the MCC and a panelboard would be less expensive and is all that is needed at this site.

Limiting Factors

The electrical service entrance equipment and MCCs are old and need to be replaced. The electrical service entrance equipment and MCCs are the main electrical components of the WTP. If either fails, the WTP would not be able to operate. It is necessary to update this equipment before a failure occurs.

SECTION 5 – CAPITAL IMPROVEMENTS PLAN

Presented in this Section is the Capital Improvements Plan (CIP) for the Northglenn WTP. These improvements are projected to occur over the next 10 years and have been prioritized with staff. Projects projected for the 5 to 10-year horizon have been shown with a completion year of 2030. The CIP includes recommended projects based on staff input and the performance limiting factors identified in Section 4. The list of CIP projects described in this section are organized by flow through the WTP followed by residuals handling, and electrical and controls. At the request of staff, the preparation of the American Water Infrastructure Act (AWIA) Risk Resilience Assessment and Emergency Response Plan has been added to the CIP list as Project No. 1.

CAPITAL IMPROVEMENT PROJECTS

In working with staff, the following projects where identified for inclusion in the capital improvements plan. Costs are shown as 2020 dollars. Future project costs must be inflated using industry standard indexes. Smaller projects at budgets less than $150,000 are estimated based on industry experience and standards. Larger project costs are detailed in the opinion of cost matrixes provided in Appendix B. Recommended improvement areas are shown in the Figure 12 site plan.

Project No. 1 – AWIA – Risk Resilience Assessment & Emergency Response Plan

Per the AWIA of 2018, utilities are required to prepare a Risk Resilience Assessment and Emergency Response Plan. At a population of less than 50,000, the City of Northglenn must submit the Risk Resilience Assessment by June 30, 2021, followed by the Emergency Reponse Plan by December 30, 2021. Key elements of the plans are listed below.

Risk Resilience Assessment

- Assess hazards
- Resilience of water infrastructure
- Monitoring practices
- Financial systems
- Chemical storage practices

Emergency Response Plan

- Strategies to improve resiliency (physical security & cybersecurity)
- Plans for responding to malevolent acts and threats
- Actions to lessen the impact of malevolent acts & natural hazards
- Strategies to detect malevolent acts & hazards

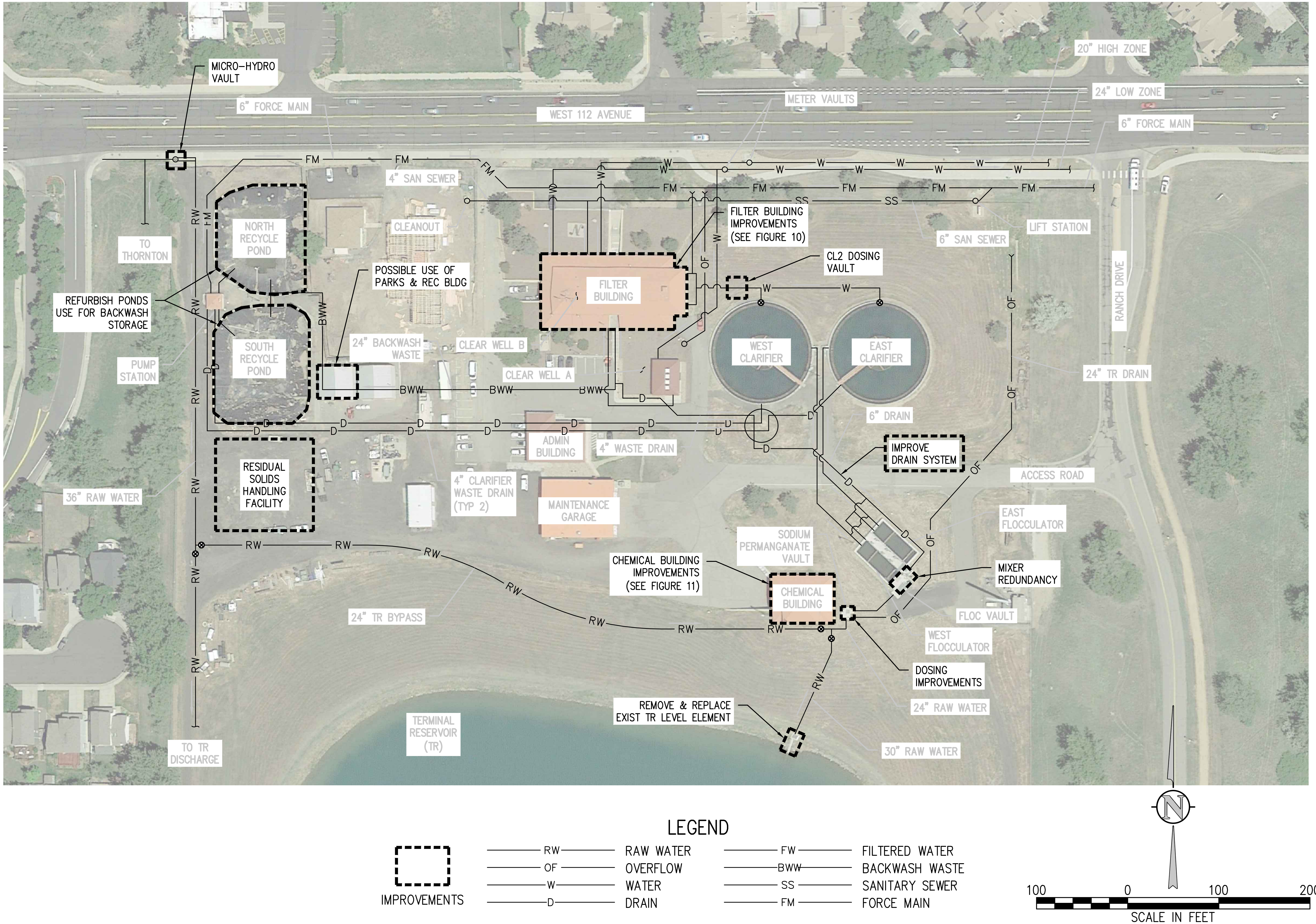

FIGURE 12 - WATER TREATMENT FACILITY IMPROVEMENT PLAN

CITY OF NORTHGLENN WATER MASTER PLAN UPDATE

AUGUST 2020

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Risk Resilience Assessment

Year to Complete: June 2021
Anticipated Cost: $50,000

Emergency Response Plan

Year to Complete: December 2021
Anticipated Cost: $50,000

Project No. 2 – Hydropower Generation

The raw water PRV reduces water pressure from approximately 100 to 30 PSI. Rather than reducing the pressure via headloss through a PRV, an in-line hydro-turbine could reduce the pressure while simultaneously generating electricity for the WTP. Based on existing raw water flows and conservative assumptions for energy generation, an in-line hydro-turbine could generate enough power to save the City between $400 and $1,000 per month in energy costs. In addition, the City would be relying on clean, renewable energy to offset monthly electrical costs. A hydropower feasibility study would need to be performed, as well as an interconnect study with Xcel Energy, and permitting through the Federal Energy Regulatory Commission (FERC).

Year to Complete: 2030
Feasibility & Interconnect Study: $50,000
Anticipated Cost: $1,000,000

Project No. 3 – TR Level Indicator

A new level monitoring system for the TR is recommended to replace the current system, which can be inaccurate due to the location. A system at the reservoir outlet is suggested consisting of a hydrostatic pressure transducer or bubbler

Year to Complete:2021
Anticipated Cost: $40,000

Project No. 4 – CL17 and Streaming Current Monitor (SCM) – Pretreatment Improvements

As discussed in Section 4, the City expressed interest in optimizing chemical usage for manganese removal and coagulation. The recommended instruments for these functions are a Hach CL-17 and an SCM (Milton Roy, Hach, Chemtrac). The CL-17 typically measures free chlorine residual, however, can also measure permanganate concentration using a conversion factor. The SCM uses an electronic sensor to determine whether neutralization has occurred after the addition of coagulant to the raw water. SCMs have a good track record for use as a device to optimize coagulant, particularly on waters with low turbidity and TDS

Year to Complete: 2022
Anticipated Cost: $30,000

Project No. 5 – Chemical Feed and Storage Improvements

The higher priority improvements as identified in Section 4 include secondary containment and the addition of back up chemical pumping. Sodium permanganate, alum, polymer, sodium hydroxide and sodium hypochlorite are all feed by a single chemical pump, and shelf ready replacements should be purchased. The alum storage is in flux and the current in process improvement of installing six new tanks should be completed along with the construction of secondary containment. In addition, secondary containment improvements are needed for the sodium hydroxide and polymer systems.

The chemical building is original to the WTP and has served its useful life of over 40 years. Building HVAC and electrical components have aged and access to tanks and equipment is inadequate. It is recommended that a more thorough evaluation be conducted for the entire chemical feed and storage system and building. This evaluation should include occupancy classifications, chemical storage requirements, fail/safe controls, electrical/control elements, ventilation, fire suppression, safety showers and a review of applicable CDPHE, NFPA and building codes.

Higher Priority Improvements (Chemical Pumps & Secondary Containment)
Year to Complete: 2021
Anticipated Cost: $320,000

Chemical Systems Comprehensive Evaluation
Year to Complete: 2022
Anticipated Cost: $60,000

Project No. 6 – Rapid Mix Improvements

To help ensure redundancy for the pretreatment system, a shelf spare rapid mixer was suggested by staff and is recommended for installation. The current rapid mix system is a single point of failure, that can substantially reduce treatment performance in an emergency. The WTP is currently set up with a hoist system for removing the on-line rapid mixer, which can be used for installing the shelf spare.

Year to Complete: 2021
Anticipated Cost: $50,000

Project No. 7 – Flocculator Basin Improvements

The flocculation basins were added in 2014 and have performed well. Each flocculation basin is equipped with a 6-inch drain and isolation valve. However, the drain connects to a 4-inch bottleneck before entering the 24-inch backwash waste line feeding the residual ponds. The reduction in drain size, inhibits the time it takes to drop down a basin for maintenance or emergency situations. Replacement of the restricted 4-inch drain is recommended. The length of replacement is estimated at 200 LF and would require a deep trench and asphalt replacement.

Based on the evaluation, the 14 MGD original capacity designation has been downgraded due to detention time and velocity restraints. 1n 2014, the system was compliant with CDPHE design criteria, which were modified in 2017. It is recommended that the City request a variance for the flocculation time and velocity during the next construction plan submittal (possibly with the Residual Solids Project) and retain the 14 MGD designation.

Year to Complete:2022
Anticipated Cost: $80,000

Project No. 8 – Chlorine Vault Improvements

Due to age and accessibility, replacement of the existing chlorine addition vault is recommended. Another option is to daylight the chlorine injection system at the existing PAC chamber and abandon the current vault or utilize the vault as back-up. The opinion of cost for replacing the chlorine vault with better access and improved operations is provided below.

Year to Complete: 2024
Anticipated Cost: $100,000

Project No. 9 – Filter Improvements

The filter building was originally constructed in 1979 and a number of improvements have been identified. In working with staff, the following are recommended:

- Automate filter-to-waste system (valves and piping have already been roughed out)
- Automate filter backwash
- Addition of air scour to backwash system
- Improve ventilation in filter control room
- Update filter controls and SCADA (see Project No. 12)
- Sand blast and paint filter gallery piping
- Replace original valve actuators

Staff manually operate the existing filter-to-waste system. Automating the filter-to-waste would improve operational flexibility. The existing filters rely on a surface wash system to aid in removing surface material from the filter beds but do not capture the filter box corners. Addition of air scour to the filters will increase filter backwash efficiency with improved filter coverage and bed expansion thereby reducing the volume of treated water required for backwash and backwash waste and increasing filter rum times.

Year to Complete: 2023 to 2026
Anticipated Cost: $ 1.1 million

Project No. 10 – High Service Pump Improvements

The high service pumps in Clearwell B are original and now 40 years old. Over the operational period several motors have been replaced and the City purchased four VFDs for the Low Zone 150 hp pumps. The VFDs have yet to be installed and were purchased to reduce power costs and provide better flow control. The VFDs have not been installed due to discovered harmonic and vibration issues. A pumping system evaluation is recommended to inspect the aging pump casings and bowels and remedy the vibration problem. The high zone pumps can also be evaluated along with the Clearwell A, 125 pumps, which have never been used. A plan to exercise and periodically operate the Clearwell A pumps is needed. Assuming the harmonics issue can be overcome, the four VFDs should be installed. The pump evaluation and VFD install budgetary costs are provided.

Pumping System Evaluation
Year to Complete: 2021
Anticipated Cost: $40,000

Low Zone Pumps – VFD Installation (4)
Year to Complete: 2022
Anticipated Cost: $100,000

No. 11 – Residuals Handling Improvements

The existing residuals handling system is outdated and poorly functioning. Sedimentation solids (blow down) are currently mixed with the backwash solids and sent to the sewer. The residuals handling system needs to be improved to allow for recycling of the backwash water and reduce waste volumes sent to the sanitary sewer collection system. This can be achieved though the addition of settling basins or a gravity thickener providing high quality decant water and the implementation of a residuals dewatering process using mechanical equipment such as a screw press or rotary fan press. The residual handling system will require a new building, settling basins and associated infrastructure. Opportunities for beneficial reuse of the residual solids should also be explored.

Year to Complete:2021-2022
Anticipated Cost: $3.6 million

Project No. 12 – Electrical and Controls Improvements

Due to age of the equipment, electrical and control improvements are a high priority. Because of the critical nature of the electrical gear and devices it is very important to complete these improvements prior to failure. The budgetary costs for the WTP switchgear, filter building MCC, booster pump station MCC and filter control console replacement are presented below. Potential future electrical and control improvements at the chemical building are separate and should be part of a more detailed holistic evaluation of the facility.

Year to Complete: 2025
Anticipated Cost

WTP Main Service Entrance Switchgear - $400,000
Filter Building MCC - $350,000
Booster Pump Station MCC - $150,000
Filter Control Console ($60,000 per x 4) = $240,000

SECTION 6 – FUNDING OPTIONS

GRANTS AND LOANS

The following grants and loans are available to the City as funding options for CIP projects.

COLORADO WATER CONSERVATION BOARD (CWCB)

The CWCB offers numerous loans and grants for water-related projects, studies, planning documents, awareness campaigns and other activities. Some relevant grants that the City may be eligible for include the following:

- Water Project Loan Program: Provides low-interest loans for the design and construction of raw water projects. Interest rates range from 2.15% for low income municipalities to 2.75% for high income municipalities.
- Colorado's Water Plan (CWP) Grants: Provides financial assistance to make progress on the CWP's Measurable Objectives or critical actions
- Water Efficiency Grants: Provides financial assistance for water conservation-related projects
- Water Supply Reserve Account Grants: Provides grants and loans to assist with water supply issues
- Severance Tax Trust Fund Operational Account Grants: Provides grants for regional water resource planning studies and associated demonstration projects

For more detailed information including application deadlines and procedures, please consult the CWCB website.

ENERGY AND MINERAL IMPACT ASSISTANCE FUND (EIAF)

The purpose of the Energy and Mineral Impact Assistance Program is to assist political subdivisions that are socially and/or economically impacted by the development, processing, or energy conversion of minerals and mineral fuels. Funds come from the state severance tax on energy and mineral production and from a portion of the state's share of royalties paid to the federal government for mining and drilling of minerals and mineral fuels on federally owned land.

The kinds of projects that are funded include, but are not limited to, water and sewer improvements, road improvements, construction/improvements to recreation centers, senior centers and other public facilities, fire protection buildings and equipment, and local government planning. The EIAF grants are categorized into Administrative Grants, Tier I, Tier II, and Tier III. Application deadlines for each category are on April 1, August 1, and December 1 of each year.

Administrative Grants

Administrative Grants are available for planning, preliminary engineering, and architectural design projects. The application process requires the local government to submit a detailed letter to the appropriate DOLA (Department of Local Affairs) Regional Manager, and signed by the Chief Elected Official. The letter should include information such as the project description, budget, financial need, why the project is necessary, urgency of the project, how soon the project can begin, and how soon it can be completed. The maximum award for an Administrative Grant is $25,000, and the total project cost should not exceed $100,000. A dollar-for-dollar match is required for this grant.

Tier I Grants

Tier I grant funds can be used for a variety of public purposes including planning, engineering and design studies, and capital projects requiring a limited level of financial assistance. A Tier I grant awards up to $200,000. Applications for grant consideration will be expected to include a minimum match of 25%. Larger matching amounts are generally more competitive. Applications will be reviewed and recommended for funding by DOLA staff. The Executive Director will make funding decisions three times per year.

Tier II Grants

The Tier II grant program is intended to support a wide variety of community development projects to improve quality of life in communities. A Tier II grant awards greater than $200,000 up to $2.0 million. Applications for grant consideration will be expected to include a minimum match of 25%. Larger matching amounts are generally more competitive. Applications will be reviewed and recommended for funding by DOLA staff. The Executive Director will make funding decisions three times per year.

Tier III Grants

To be competitive for a Tier III grant, applications require regional or multi-jurisdictional collaboration assistance to solve a multi-jurisdictional problem. A Tier III grant awards greater than $2.0 million. Applications will be reviewed and recommended for funding by DOLA staff. The Executive Director will make funding decisions dependent on revenue availability. Local governments that receive a Tier III grant may be asked to withdraw from future funding application cycles.

REVOLVING FUND (SRF) – LOW INTEREST LOANS

The Drinking Water Revolving Fund (DWRF) provides low interest loans to governmental entities for the construction of water projects for public health and compliance purposes. The DWRF can support the following types of projects:

- New Regional Water Treatment Facilities
- Improvement / Expansion of Water Treatment Plant
- Consolidation of Water Treatment Facilities

- Connect to Existing Facility Eliminate Individual Private Wells
- Distribution / Transmission Lines Construction / Rehabilitation
- Water Storage Facilities
- Water Supply Facilities (excluding reservoirs, dams and water rights)

Available loan types include:

- Direct Loans: up to $3.0 million, current APR of 2.5 percent for 20 years.
- Leveraged Loans: generally provided to investment grade borrowers with larger projects greater than $3.0 million, bond market interest rate for 20 years.

The CDPHE Water Quality Control Division (WQCD), DOLA, and the Colorado Water Resources and Power Development Authority (Authority) jointly administer the SRF program. If the City is considered a DAC, then they could qualify for either Category 1 or 2 DAC interest rates. Category 1 communities are qualified to receive a low interest rate established at 50 percent of the direct loan rate on loans up to $3.0 million with a term up to 30 years. Category 2 communities are qualified to receive a zero percent interest rate on loans up to $3.0 million with a loan term up to 30 years. The City does not satisfy the eligibility requirements for Category 1 but may be eligible for Category 2 community loans. Although the City does not meet the defined eligibility requirements, the City could qualify as a DAC pending a prequalification application and review by the Authority. The WQCD administers the environmental reviews; engineering and design approval; and overall project management. The Authority manages the finances and loan approvals. DOLA staff works with applicants on credit reviews and reports.

There are several milestones that need to be met in order for a project to be eligible for the DWRF.

- The entity must be included on the most current Drinking Water Intended Use Plan – (Complete)
- A Prequalification Application must be submitted to the Grants and Loans Unit
- A Preapplication meeting with the WQCD, DOLA, and the Authority must be held
- Eligibility for a $10,000 Planning Grant is determined at the Preapplication meeting
- A Project Needs Assessment (PNA) and Environmental Determination for the project must be submitted to the WQCD Engineering Section for review
- WQCD will provide an Environmental Determination (Categorical Exclusion or Environmental Assessment)
- If necessary, an Environmental Assessment shall be submitted and reviewed. If a Finding of No Significant Impact (FNSI) is determined it shall be published with a 30-day comment period
- PNA and Environmental Approval must be obtained.
- Eligibility for a Design and Engineering Grant is determined after approval of the PNA
- A Technical, Managerial, and Financial (TMF) Capacity review must be completed and submitted to the WQCD a minimum of 30 days prior to the loan application.
- A public meeting must be held with a 30-day notice period, notifying the public of the project.
- The loan application shall then be submitted.
- The Authority will then approve the loan.

HYDROELECTRIC POWER GENERATION FUNDING OPPORTUNITIES

Multiple funding opportunities exist including the Colorado Water Resource and Power Development Authority (CWRPDA), which have a Small Hydropower Loan program that currently offers 30-year loans for projects up to $5-million dollars for governmental agencies. Since the proposed microturbine would produce less than 10MW per year, this project would be eligible for FERC exemption, thus reducing the cost for permitting efforts. Additional funding is also available through the Colorado Water Conservation Board (CWCB), as well as the Colorado Department of Agriculture.

APPENDIX A – CALCULATIONS

NORTHGLENN WTP MASTER PLAN UPDATE
CHLORINE CONTACT TIME CALCULATION

Reference:
CDPHE Log Inactivation Brochure (2009)
https://www.colorado.gov/pacific/sites/default/files/WQ-ENG-AppendixA%20Log%20Inactivation%20Brochure%202009.pdf

Instructions

User Input

Contact Time Calculations - 0.5 BF, Winter Temp, both CWs and Filters

Flow	
8.70	MGD
6,042	gpm

	Section	Volume	Baffle Factor	Effective Volume	Flow	Detention Time	Free Chlorine Residual	CT_{CALC}	pH	Temp	Giardia $CT_{99.9}$	Giardia Inactivation	Virus $CT_{99.9}$	Virus Inactivation
		(gal)		(gal)	(gpm)	(min)	(mg/L)	(min*mg/L)		(deg C)	(min*mg/L)	(Log)	(min*mg/L)	(Log)
1	Filter Volume	247,800	0.70	173,460	6,042	28.71	1.2	34.45	8.2	2.0	305.3	0.339	10.60	13.001
2	Clearwell A	340,000	0.50	170,000	6,042	28.14	1.2	33.77	8.2	2.0	305.3	0.332	10.60	12.742
3	36" DIP Between CW A and B	8,615	0.27	2,340	6,042	0.39	1.2	0.46	8.2	2.0	305.3	0.005	10.60	0.175
4	Clearwell B	544,000	0.50	272,000	6,042	45.02	1.2	54.02	8.2	2.0	305.3	0.531	11.60	18.629
											Subtotal	1.206	Subtotal	44.547
											Credit	2.5	Credit	0
											Total	**3.71**	**Total**	**44.55**
											Required	3.0	Required	4.0

Contact Time Calculations - 0.5 BF, Winter Temp, both CWs

Flow	
8.70	MGD
6,042	gpm

	Section	Volume	Baffle Factor	Effective Volume	Flow	Detention Time	Free Chlorine Residual	CT_{CALC}	pH	Temp	Giardia $CT_{99.9}$	Giardia Inactivation	Virus $CT_{99.9}$	Virus Inactivation
		(gal)		(gal)	(gpm)	(min)	(mg/L)	(min*mg/L)		(deg C)	(min*mg/L)	(Log)	(min*mg/L)	(Log)
1	Clearwell A	340,000	0.50	170,000	6,042	28.14	1.2	33.77	8.2	2.0	305.3	0.332	10.60	12.742
2	36" DIP Between CW A and B	8,615	0.27	2,340	6,042	0.39	1.2	0.46	8.2	2.0	305.3	0.005	10.60	0.175
3	Clearwell B	544,000	0.50	272,000	6,042	45.02	1.2	54.02	8.2	2.0	305.3	0.531	10.60	20.387
											Subtotal	0.867	Subtotal	33.304
											Credit	2.5	Credit	0
											Total	**3.37**	**Total**	**33.30**
											Required	3.0	Required	4.0

Contact Time Calculations - 0.5 BF, Winter Temp, both CWs, Max Flow Rate

Flow	
15.00	MGD
10,417	gpm

	Section	Volume	Baffle Factor	Effective Volume	Flow	Detention Time	Free Chlorine Residual	CT_{CALC}	pH	Temp	Giardia $CT_{99.9}$	Giardia Inactivation	Virus $CT_{99.9}$	Virus Inactivation
		(gal)		(gal)	(gpm)	(min)	(mg/L)	(min*mg/L)		(deg C)	(min*mg/L)	(Log)	(min*mg/L)	(Log)
1	Clearwell A	340,000	0.50	170,000	10,417	16.32	1.2	19.58	8.2	2.0	305.3	0.192	10.60	7.390
2	36" DIP Between CW A and B	8,615	0.27	2,340	10,417	0.22	1.2	0.27	8.2	2.0	305.3	0.003	10.60	0.102
3	Clearwell B	544,000	0.50	272,000	10,417	26.11	1.2	31.33	8.2	2.0	305.3	0.308	10.60	11.824
											Subtotal	0.503	Subtotal	19.316
											Credit	2.5	Credit	0
											Total	**3.00**	**Total**	**19.32**
											Required	3.0	Required	4.0

Contact Time Calculations - 0.5 BF, Winter Temp, CW A

Flow	
8.70	MGD
6,042	gpm

	Section	Volume	Baffle Factor	Effective Volume	Flow	Detention Time	Free Chlorine Residual	CT_{CALC}	pH	Temp	Giardia $CT_{99.9}$	Giardia Inactivation	Virus $CT_{99.9}$	Virus Inactivation
		(gal)		(gal)	(gpm)	(min)	(mg/L)	(min*mg/L)		(deg C)	(min*mg/L)	(Log)	(min*mg/L)	(Log)
1	Clearwell A	340,000	0.5	170,000	6,042	28.14	1.8	50.65	8.2	2.0	305.3	0.498	10.60	19.113
											Subtotal	0.498	Subtotal	19.113
											Credit	2.5	Credit	0
											Total	**3.00**	**Total**	**19.11**
											Required	3.0	Required	4.0

Contact Time Calculations - 0.5 BF, Winter Temp, CW B

Flow	
8.70	MGD
6,042	gpm

	Section	Volume	Baffle Factor	Effective Volume	Flow	Detention Time	Free Chlorine Residual	CT_{CALC}	pH	Temp	Giardia $CT_{99.9}$	Giardia Inactivation	Virus $CT_{99.9}$	Virus Inactivation
		(gal)		(gal)	(gpm)	(min)	(mg/L)	(min*mg/L)		(deg C)	(min*mg/L)	(Log)	(min*mg/L)	(Log)
1	Clearwell B	544,000	0.5	272,000	6,042	45.02	1.2	54.02	8.2	2.0	305.3	0.531	10.60	20.387
											Subtotal	0.531	Subtotal	20.387
											Credit	2.5	Credit	0
											Total	**3.03**	**Total**	**20.39**
											Required	3.0	Required	4.0

Assumptions

Clearwell A		
Length	60	ft
Width	56	ft
Interior Height	14	ft
Height to Overflow	13.5	ft
Water Height	13.5	ft
Water Volume	340,000	gal

Notes
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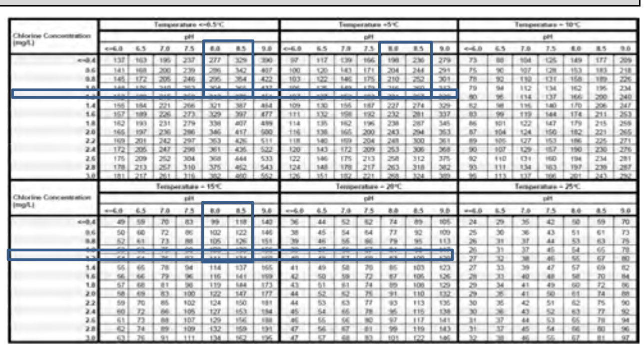
36" DIP Between CW A and B		
Pipe Diameter	3	ft
Pipe Length	163	ft
Baffling Factor	0.27	ft
Pipe Volume	8,615	gal

Clearwell B		
Length	70	ft
Width	77	ft
Interior Height	14	ft
Height to Overflow	13.47	ft
Water Height	13.47	ft
Water Volume	544,000	gal

Filter Volume		
No. of Filters Running	7	
Single Filter Volume	35,400	gal
Total Filter Volume	247,800	gal



Temperature °C	pH 6-9	pH 10
0.5	12	90
5	8	60
10	6	45
15	4	30
20	3	22
25	2	15

NORTHGLENN WTP MASTER PLAN UPDATE
CHLORINE CONTACT TIME CALCULATION, MAX FLOWS

Reference:
CDPHE Log Inactivation Brochure (2009)
https://www.colorado.gov/pacific/sites/default/files/WQ-ENG-AppendixA%20Log%20Inactivation%20Brochure%202009.pdf

Instructions

User Input

Contact Time Calculations - 0.5 BF, Winter Temp, both CWs and Filters

Flow 21.00 MGD
14,583 gpm

	Section	Volume	Baffle Factor	Effective Volume	Flow	Detention Time	Free Chlorine Residual	CT_{CALC}	pH	Temp	Giardia $CT_{99.9}$	Giardia Inactivation	Virus $CT_{99.9}$	Virus Inactivation
		(gal)		(gal)	(gpm)	(min)	(mg/L)	(min*mg/L)		(deg C)	(min*mg/L)	(Log)	(min*mg/L)	(Log)
1	Filter Volume	247,800	0.70	173,460	14,583	11.89	1.2	14.27	8.2	2.0	305.3	0.140	10.60	5.386
2	Clearwell A	340,000	0.50	170,000	14,583	11.66	1.2	13.99	8.2	2.0	305.3	0.137	10.60	5.279
3	36" DIP Between CW A and B	8,615	0.27	2,340	14,583	0.16	1.2	0.19	8.2	2.0	305.3	0.002	10.60	0.073
4	Clearwell B	544,000	0.50	272,000	14,583	18.65	1.2	22.38	8.2	2.0	305.3	0.220	11.60	7.718
		1,140,415									Subtotal	0.500	Subtotal	18.455
											Credit	2.5	Credit	0
											Total	**3.00**	**Total**	**18.46**
											Required	3.0	Required	4.0

Contact Time Calculations - 0.5 BF, Winter Temp, both CWs, Max Flow Rate

Flow 15.00 MGD
10,417 gpm

	Section	Volume	Baffle Factor	Effective Volume	Flow	Detention Time	Free Chlorine Residual	CT_{CALC}	pH	Temp	Giardia $CT_{99.9}$	Giardia Inactivation	Virus $CT_{99.9}$	Virus Inactivation
		(gal)		(gal)	(gpm)	(min)	(mg/L)	(min*mg/L)		(deg C)	(min*mg/L)	(Log)	(min*mg/L)	(Log)
1	Clearwell A	340,000	0.50	170,000	10,417	16.32	1.2	19.58	8.2	2.0	305.3	0.192	10.60	7.390
2	36" DIP Between CW A and B	8,615	0.27	2,340	10,417	0.22	1.2	0.27	8.2	2.0	305.3	0.003	10.60	0.102
3	Clearwell B	544,000	0.50	272,000	10,417	26.11	1.2	31.33	8.2	2.0	305.3	0.308	10.60	11.824
		892,615									Subtotal	0.503	Subtotal	19.316
											Credit	2.5	Credit	0
											Total	**3.00**	**Total**	**19.32**
											Required	3.0	Required	4.0

Contact Time Calculations - 0.5 BF, Winter Temp, CW A

Flow 5.77 MGD
4,009 gpm

	Section	Volume	Baffle Factor	Effective Volume	Flow	Detention Time	Free Chlorine Residual	CT_{CALC}	pH	Temp	Giardia $CT_{99.9}$	Giardia Inactivation	Virus $CT_{99.9}$	Virus Inactivation
		(gal)		(gal)	(gpm)	(min)	(mg/L)	(min*mg/L)		(deg C)	(min*mg/L)	(Log)	(min*mg/L)	(Log)
1	Clearwell A	340,000	0.5	170,000	4,009	42.40	1.2	50.88	8.2	2.0	305.3	0.500	10.60	19.201
											Subtotal	0.500	Subtotal	19.201
											Credit	2.5	Credit	0
											Total	**3.00**	**Total**	**19.20**
											Required	3.0	Required	4.0

Contact Time Calculations - 0.5 BF, Winter Temp, CW B

Flow 9.24 MGD
6,415 gpm

	Section	Volume	Baffle Factor	Effective Volume	Flow	Detention Time	Free Chlorine Residual	CT_{CALC}	pH	Temp	Giardia $CT_{99.9}$	Giardia Inactivation	Virus $CT_{99.9}$	Virus Inactivation
		(gal)		(gal)	(gpm)	(min)	(mg/L)	(min*mg/L)		(deg C)	(min*mg/L)	(Log)	(min*mg/L)	(Log)
1	Clearwell B	544,000	0.5	272,000	6,415	42.40	1.2	50.88	8.2	2.0	305.3	0.500	10.60	19.201
											Subtotal	0.500	Subtotal	19.201
											Credit	2.5	Credit	0
											Total	**3.00**	**Total**	**19.20**
											Required	3.0	Required	4.0

Assumptions

Clearwell A		
Length	60 ft	
Width	56 ft	
Interior Height	14 ft	
Height to Overflow	13.5 ft	
Water Height	13.5 ft	
Water Volume	340,000	gal

Notes
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36" DIP Between CW A and B		
Pipe Diameter	3 ft	
Pipe Length	163 ft	
Baffling Factor	0.27 ft	
Pipe Volume	8,615	gal

Clearwell B		
Length	70 ft	
Width	77 ft	
Interior Height	14 ft	
Height to Overflow	13.47 ft	
Water Height	13.47 ft	
Water Volume	544,000	gal

Filter Volume		
No. of Filters Running	7	
Single Filter Volume	35,400	gal
Total Filter Volume	247,800	gal

Chlorine Concentration (mg/L)	Temperature <=0.5°C pH <=6.0	6.5	7.0	7.5	8.0	8.5	9.0	Temperature =5°C pH <=6.0	6.5	7.0	7.5	8.0	8.5	9.0	Temperature = 10°C pH <=6.0	6.5	7.0	7.5	8.0	8.5	9.0
<=0.4	137	163	195	237	277	329	390	97	117	139	166	198	236	279	73	88	104	125	149	177	209
0.6	141	168	200	239	286	342	407	100	120	143	171	204	244	291	75	90	107	128	153	183	218
0.8	145	172	205	246	295	354	422	103	122	146	175	210	252	301	78	92	110	131	158	189	226
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	79	94	112	134	162	195	234
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	80	95	114	137	166	200	240
1.4	155	184	221	266	321	387	464	109	130	155	187	227	274	329	82	98	116	140	170	206	247
1.6	157	189	226	273	329	397	477	111	132	158	192	232	281	337	83	99	119	144	174	211	253
1.8	162	193	231	279	338	407	489	114	136	162	196	238	287	345	86	101	122	147	179	215	259
2.0	165	197	236	286	346	417	500	116	138	165	200	243	294	353	87	104	124	150	182	221	265
2.2	169	201	242	297	353	426	511	118	140	169	204	248	300	361	89	105	127	153	186	225	271
2.4	172	205	247	298	361	435	522	120	143	172	209	253	306	368	90	107	129	157	190	230	276
2.6	175	209	252	304	368	444	533	122	146	175	213	258	312	375	92	110	131	160	194	234	281
2.8	178	213	257	310	375	452	543	124	148	178	217	263	318	382	93	111	134	163	197	239	287
3.0	181	217	261	316	382	460	552	126	151	182	221	268	324	389	95	113	137	166	201	243	292

Chlorine Concentration (mg/L)	Temperature = 15°C pH <=6.0	6.5	7.0	7.5	8.0	8.5	9.0	Temperature = 20°C pH <=6.0	6.5	7.0	7.5	8.0	8.5	9.0	Temperature = 25°C pH <=6.0	6.5	7.0	7.5	8.0	8.5	9.0
<=0.4	49	59	70	83	99	118	140	36	44	52	62	74	89	105	24	29	35	42	50	59	70
0.6	50	60	72	86	102	122	146	38	45	54	64	77	92	109	25	30	36	43	51	61	73
0.8	52	61	73	88	105	126	151	39	46	55	66	79	95	113	26	31	37	44	53	63	75
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	26	31	37	45	54	65	78
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	27	32	38	46	55	67	80
1.4	55	65	78	94	114	137	165	41	49	58	70	85	103	123	27	33	39	47	57	69	82
1.6	56	66	79	96	116	141	169	42	50	59	72	87	105	126	28	33	40	48	58	70	84
1.8	57	68	81	98	119	144	173	43	51	61	74	89	108	129	29	34	41	49	60	72	86
2.0	58	69	83	100	122	147	177	44	52	62	75	91	110	132	29	35	41	50	61	74	88
2.2	59	70	85	102	124	150	181	44	53	63	77	93	113	135	30	35	42	51	62	75	90
2.4	60	72	86	105	127	153	184	45	54	65	78	95	115	138	30	36	43	52	63	77	92
2.6	61	73	88	107	129	156	188	46	55	66	80	97	117	141	31	37	44	53	65	78	94
2.8	62	74	89	109	132	159	191	47	56	67	81	99	119	143	31	37	45	54	66	80	96
3.0	63	76	91	111	134	162	195	47	57	68	83	101	122	146	32	38	46	55	67	81	97

Temperature °C	pH 6-9	10
0.5	12	90
5	8	60
10	6	45
15	4	30
20	3	22
25	2	15

NORTHGLENN WTP MASTER PLAN UPDATE
CHLORINE CONTACT TIME CALCULATION

Reference:
CDPHE Log Inactivation Brochure (2009)
https://www.colorado.gov/pacific/sites/default/files/WQ-ENG-AppendixA%20Log%20Inactivation%20Brochure%202009.pdf

Instructions

User Input

Contact Time Calculations - 0.5 BF, Winter Temp, both CWs

Flow	8.70	MGD
	6,042	gpm

	Section	Volume	Baffle Factor	Effective Volume	Flow	Detention Time	Free Chlorine Residual	CT_{CALC}	pH	Temp	Giardia $CT_{99.9}$	Giardia Inactivation	Virus $CT_{99.9}$	Virus Inactivation
		(gal)		(gal)	(gpm)	(min)	(mg/L)	(min*mg/L)		(deg C)	(min*mg/L)	(Log)	(min*mg/L)	(Log)
1	Clearwell A	340,000	0.5	170,000	6,042	28.14	1.2	33.77	7.0	2.0	194	0.522	10.60	12.742
2	36" DIP Between CW A and B	8,615	0.27	2,340	6,042	0.39	1.2	0.46	7.0	2.0	194	0.007	10.60	0.175
3	Clearwell B	544,000	0.5	272,000	6,042	45.02	1.2	54.02	7.0	2.0	194	0.835	10.60	20.387
											Subtotal	1.365	Subtotal	33.304
											Credit	2.5	Credit	0
											Total	**3.86**	**Total**	**33.30**
											Required	3.0	Required	4.0

Contact Time Calculations - 0.5 BF, Winter Temp, CW A

Flow	8.70	MGD
	6,042	gpm

	Section	Volume	Baffle Factor	Effective Volume	Flow	Detention Time	Free Chlorine Residual	CT_{CALC}	pH	Temp	Giardia $CT_{99.9}$	Giardia Inactivation	Virus $CT_{99.9}$	Virus Inactivation
		(gal)		(gal)	(gpm)	(min)	(mg/L)	(min*mg/L)		(deg C)	(min*mg/L)	(Log)	(min*mg/L)	(Log)
1	#REF!	340,000	0.5	170,000	6,042	28.14	1.2	33.77	7.0	2.0	194	0.522	10.60	12.742
											Subtotal	0.522	Subtotal	12.742
											Credit	2.5	Credit	0
											Total	**3.02**	**Total**	**12.74**
											Required	3.0	Required	4.0

Contact Time Calculations - 0.5 BF, Winter Temp, CW B

Flow	8.70	MGD
	6,042	gpm

	Section	Volume	Baffle Factor	Effective Volume	Flow	Detention Time	Free Chlorine Residual	CT_{CALC}	pH	Temp	Giardia $CT_{99.9}$	Giardia Inactivation	Virus $CT_{99.9}$	Virus Inactivation
		(gal)		(gal)	(gpm)	(min)	(mg/L)	(min*mg/L)		(deg C)	(min*mg/L)	(Log)	(min*mg/L)	(Log)
1	Clearwell B	544,000	0.5	272,000	6,042	45.02	1.2	54.02	7.0	2.0	194	0.835	10.60	20.387
											Subtotal	0.835	Subtotal	20.387
											Credit	2.5	Credit	0
											Total	**3.34**	**Total**	**20.39**
											Required	3.0	Required	4.0

Assumptions

1	**Clearwell A**		
	Length	60	ft
	Width	56	ft
	Interior Height	14	ft
	Height to Overflow	13.5	ft
	Water Height	13.5	ft
	Water Volume	340,000	gal

Notes
--

2	**36" DIP Between CW A and B**		
	Pipe Diameter	3	ft
	Pipe Length	163	ft
	Baffling Factor	0.27	ft
	Pipe Volume	8,615	gal

3	**Clearwell B**		
	Length	70	ft
	Width	77	ft
	Interior Height	14	ft
	Height to Overflow	13.47	ft
	Water Height	13.47	ft
	Water Volume	544,000	gal

Chlorine Concentration (mg/L)	Temperature <=0.5°C pH <=6.0	6.5	7.0	7.5	8.0	8.5	9.0	Temperature =5°C pH <=6.0	6.5	7.0	7.5	8.0	8.5	9.0	Temperature = 10°C pH <=6.0	6.5	7.0	7.5	8.0	8.5	9.0
<=0.4	137	163	196	237	277	329	390	97	117	139	166	198	236	279	73	88	104	125	149	177	209
0.6	141	168	200	239	286	342	407	100	120	143	171	204	244	291	75	90	107	128	153	183	218
0.8	145	172	205	246	295	354	422	103	122	146	175	210	252	301	78	92	110	131	158	189	226
1.0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	79	94	112	134	162	195	234
1.2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	80	95	114	137	166	200	240
1.4	155	184	221	266	321	387	464	109	130	155	187	227	274	329	82	98	116	140	170	206	247
1.6	157	189	226	273	329	397	477	111	132	158	192	232	281	337	83	99	119	144	174	211	253
1.8	162	193	231	279	338	407	489	114	136	162	196	238	287	345	86	101	122	147	179	215	259
2.0	165	197	236	286	346	417	500	116	138	165	200	243	294	353	87	104	124	150	182	221	265
2.2	169	201	242	297	353	426	511	118	140	169	204	248	300	361	89	105	127	153	186	225	271
2.4	172	205	247	298	361	435	522	120	143	172	209	253	306	368	90	107	129	157	190	230	276
2.6	175	209	252	304	368	444	533	122	146	175	213	258	312	375	92	110	131	160	194	234	281
2.8	178	213	257	310	375	452	543	124	148	178	217	263	318	382	93	111	134	163	197	239	287
3.0	181	217	261	316	382	460	552	126	151	182	221	268	324	389	95	113	137	166	201	243	292

Chlorine Concentration (mg/L)	Temperature = 15°C pH <=6.0	6.5	7.0	7.5	8.0	8.5	9.0	Temperature = 20°C pH <=6.0	6.5	7.0	7.5	8.0	8.5	9.0	Temperature = 25°C pH <=6.0	6.5	7.0	7.5	8.0	8.5	9.0
<=0.4	49	59	70	83	99	118	140	36	44	52	62	74	89	105	24	29	35	42	50	59	70
0.6	50	60	72	86	102	122	146	38	45	54	64	77	92	109	25	30	36	43	51	61	73
0.8	52	61	73	88	105	126	151	39	46	55	66	79	95	113	26	31	37	44	53	63	75
1.0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	26	31	37	45	54	65	78
1.2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	27	32	38	46	55	67	80
1.4	55	65	78	94	114	137	165	41	49	58	70	85	103	123	27	33	39	47	57	69	82
1.6	56	66	79	96	116	141	169	42	50	59	72	87	105	126	28	33	40	48	58	70	84
1.8	57	68	81	98	119	144	173	43	51	61	74	89	108	129	29	34	41	49	60	72	86
2.0	58	69	83	100	122	147	177	44	52	62	75	91	110	132	29	35	41	50	61	74	88
2.2	59	70	85	102	124	150	181	44	53	63	77	93	113	135	30	35	42	51	62	75	90
2.4	60	72	86	105	127	153	184	45	54	65	78	95	116	138	30	36	43	52	63	77	92
2.6	61	73	88	107	129	156	188	46	55	66	80	97	117	141	31	37	44	53	65	78	94
2.8	62	74	89	109	132	159	191	47	56	67	81	99	119	143	31	37	45	54	66	80	96
3.0	63	76	91	111	134	162	195	47	57	68	83	101	122	146	32	38	46	55	67	81	97

Temperature °C	pH 6-9	10
0.5	12	90
5	8	60
10	6	45
15	4	30
20	3	22
25	2	15

Job Name: Northglenn WTP
Job Number: 1061e
Date: 2020.07
By: LCO

CONSULTING ENGINEERS

Northglenn Water Treatment Plant
Example Calculations for Sludge Production

Raw Flow Rate			
Qavg	2,500,000	GPD	
Qpeak	6,200,000	GPD	
Qfuture	0	GPD	
Amount of Dry Sludge Produced from the Removal of Turbidity			
Traw,avg	3.0	NTU	
ϕ_{raw}	1.25	g/m^3 *NTU	mg TSS/NTU Removed, Typical Value (Range is 1.0-2.0)
Sludge, Qavg from Turbidity	35	kg/d	Sludge, min from Turbidity [kg/d] = Q_{in} [m^3/s] * $T_{raw\ influent,min}$ [NTU] * ϕ_{raw} [g / NTU *m^3] * [(86.4 kg/d)/(1m^3/s)]
	78	lbs/d	Sludge, min from Turbidity [lbs/d] = Sludge, min from Turbidity [kg/d] * 2.20462 lb / 1 kg
Sludge, Qpeak from Turbidity	88	kg/d	Sludge, max from Turbidity [kg/d] = Q_{in} [m^3/s] * $T_{raw\ influent,max}$ [NTU] * ϕ_{raw} [g /NTU* m^3] * [(86.4 kg/d)/(1m^3/s)]
	194	lbs/d	Sludge, max from Turbidity [lbs/d] = Sludge, min from Turbidity [kg/d] * 2.20462 lb / 1 kg
Sludge, Qfuture from Turbidity	0	kg/d	Sludge, Future from Turbidity [kg/d] = Q_{in} [m^3/s] * $T_{raw\ influent,avg}$ [NTU] * ϕ_{raw} [g / NTU*m^3] * [(86.4 kg/d)/(1m^3/s)]
	0	lbs/d	Sludge, future from Turbidity [lbs/d] = Sludge, avg from Turbidity [kg/d] * 2.20462 lb / 1 kg
Amount of Dry Sludge Produced from Alum Addition to Raw Water			
Alum, avg	25.0	mg/L	Use upper feed rate
Alum Rate	0.45		kg dry sludge/kg of Alum, typical value from MWH
Sludge, Qavg from Alum	106	kg/d	Sludge, min from Alum [kg/d] = Q_{in} [m^3/s] * Alum [mg/L] * 1 kg dry sludge/kg Alum* [(86.4 kg/g)/(s/d)]
	235	lbs/day	Sludge, min from Alum RAW [lbs/d] = Sludge, min from Alum [kg/d] * 2.20462 lb / 1 kg
Sludge, Qpeak from Alum	264	kg/d	Sludge, max from Alum [kg/d] = Q_{in} [m^3/s] * Alum [mg/L] * 1 kg dry sludge/kg Alum* [(86.4 kg/g)/(s/d)]
	582	lbs/day	Sludge, max from Alum RAW [lbs/d] = Sludge, min from Alum [kg/d] * 2.20462 lb / 1 kg
Sludge, Qfuture from Alum	0	kg/d	Sludge, future from Alum [kg/d] = Q_{in} [m^3/s] * Alum [mg/L] * 1 kg dry sludge/kg * [(86.4 kg/g)/(s/d)]
	0	lbs/day	Sludge, future from Alum [lbs/d] = Sludge, min from Alum [kg/d] * 2.20462 lb / 1 kg
Amount of Dry Sludge Produced from Sodium Permanganate Addition to Raw Water			
Permanganate, avg	1.2	mg/L	
Safety Factor	1.0	Assumption	
Sludge, Qavg from NaMnO4	11	kg/d	Sludge, min from NaMnO4 [kg/d] = Q_{in} [m^3/s] * NaMnO4 [mg/L] * 1 kg dry sludge/kg NaMnO4* [(86.4 kg/g)/(s/d)]* SF
	25.04	lbs/day	Sludge, min from NaMnO4 [lbs/d] = Sludge, min from NaMnO4 [kg/d] * 2.20462 lb / 1 kg
Sludge, Qpeak from NaMnO4	28	kg/d	Sludge, max from NaMnO4 [kg/d] = Q_{in} [m^3/s] * NaMnO4 [mg/L] * 1 kg dry sludge/kg NaMnO4* [(86.4 kg/g)/(s/d)]* SF
	62	lbs/day	Sludge, max from NaMnO4 [lbs/d] = Sludge, min from NaMnO4 [kg/d] * 2.20462 lb / 1 kg
Sludge, Qfuture from NaMnO4	0	kg/d	Sludge, future from NaMnO4 [kg/d] = Q_{in} [m^3/s] * NaMnO4 [mg/L] * 1 kg dry sludge/kg NaMnO4* [(86.4 kg/g)/(s/d)]* SF
	0	lbs/day	Sludge, future from NaMnO4 [lbs/d] = Sludge, min from NaMnO4 [kg/d] * 2.20462 lb / 1 kg
Amount of Dry Sludge Produced from Polymer Addition to Raw Water			
Polymer Dose, avg	2.75	mg/L	
Sludge, Qavg from Polymer	26.02	kg/d	Sludge, min from Polymer [kg/d] = Q_{in} [m^3/s] * Polymer [mg/L] * 1 kg dry sludge/kg Polymer* [(86.4 kg/g)/(s/d)]
	57.37	lbs/day	Sludge, min from Polymer [lbs/d] = Sludge, min from Polymer [kg/d] * 2.20462 lb / 1 kg
Sludge, Qpeak from Polymer	65	kg/d	Sludge, max from Polymer [kg/d] = Q_{in} [m^3/s] * Polymer [mg/L] * 1 kg dry sludge/kg Polymer* [(86.4 kg/g)/(s/d)]
	142	lbs/day	Sludge, max from Polymer [lbs/d] = Sludge, min from Polymer [kg/d] * 2.20462 lb / 1 kg
Sludge, Qfuture from Polymer	0	kg/d	Sludge, future from Polymer [kg/d] = Q_{in} [m^3/s] * Polymer [mg/L] * 1 kg dry sludge/kg Polymer* [(86.4 kg/g)/(s/d)]
	0	lbs/day	Sludge, avg from Polymer [lbs/d] = Sludge, min from Polymer [kg/d] * 2.20462 lb / 1 kg
Total Amount of Sludge Produced			
Specific weight of water	8.34	lbs/gal	
% Solids in Pretreatment Sludge	0.10%	%	
Specific gravity of sludge	1.2	Assumption	
Sludge Producted at Qavg	**395**	**lbs/day**	
	158	**lbs/sludge generated per MG of water produced**	
Sludge Produced at Qpeak	**980**	**lbs/day**	
	158	**lbs/sludge generated per MG of water produced**	

Legend

Input
Calculation
Linked Cell

APPENDIX B– OPINION OF PROBABLE COSTS

OPINION OF PROBABLE COST
FOR
Project No. 2, Hydropower Generation

Description	Quantity	Units	Unit Cost	Total Cost
Mobilization/Demobilization	1	LS	$40,000	$40,000
Demo existing piping	1	EA	$5,000	$5,000
Micro Hydro Turbine	1	LS	$225,000	$225,000
Pipinng Modifications	1	LS	$20,000	$20,000
New Vault for Turbine	1	EA	$100,000	$100,000
Power Interconect with WTP MCC	600	LF	$200	$120,000
Misc Appurtenances	1	LS	$30,000	$30,000
Electrical	1	LS	$80,000	$80,000

Subtotal	**$620,000**
Contingency (15%)	$93,000
Contractor's OH&P and General Conditions (20%)	$143,000
Engineering, Permitting and Design (12%)	$103,000
Bidding and Construction Administration (7%)	$59,920
Administrative and Legal (2.5%)	$21,000
Project Total	**$1,039,920**

OPINION OF PROBABLE COST
FOR
Project No. 5, Chemical Feed & Storage Improvements

Description	Quantity	Units	Unit Cost	Total Cost
Mobilization/Demobilization	1	LS	$20,000	$20,000
Demo existing Alum Tank	1	LS	$10,000	$10,000
Finish install of three Alum Tanks	1	LS	$30,000	$30,000
Install thee more Alum Tanks (tanks on site)	1	LS	$50,000	$50,000
Secondary Containment - Alum Tanks - 2 sets	2	EA	$10,000	$20,000
Secondary Containment - Polymer Tank	1	LS	$5,000	$5,000
Finish Secondary Containment for Hyroxide	1	LS	$3,000	$3,000
Chemical Feed Pumps - for shelf	5	EA	$5,000	$25,000
Misc Fittings and Appurtenances	1	LS	$10,000	$10,000
Electrical	1	LS	$20,000	$20,000

Subtotal	**$193,000**
Contingency (15%)	$29,000
Contractor's OH&P and General Conditions (20%)	$44,000
Engineering, Permitting and Design (12%)	$32,000
Bidding and Construction Administration (7%)	$18,620
Administrative and Legal (2.5%)	$7,000
Project Total	**$323,620**

Job Name: City of Northglenn WTP MP
Job Number: 1061e
Date: 6/17/2020
By: RJA

OPINION OF PROBABLE COST
FOR
Project No. 9, Filter Improvements

Description	Quantity	Units	Unit Cost	Total Cost
Mobilization/Demobilization	1	LS	$60,000	$60,000
Demo existing surface wash	8	EA	$500	$4,000
Air Scour Blower - 50 hp w/ enclosure	1	LS	$60,000	$60,000
Air Scour piping to filters	1	LS	$20,000	$20,000
Air Scour 4-inch BFV & Actuator	8	EA	$3,500	$28,000
Air Scour Interconnect to Filter	8	EA	$3,000	$24,000
Replace 24" Backwash Waste BFV & Actuator	8	EA	$15,000	$120,000
Replace 18" Backwash Supply BFV & Actuator	8	EA	$10,000	$80,000
Replace 14" Filter Influent BFV & Actuator	8	EA	$8,000	$64,000
Replace 12" Filter Effluent BFV & Actuator	8	EA	$6,000	$48,000
Add 12" Filter to Waste Actuator	8	EA	$4,500	$36,000
Improve Ventilation in Filter Room	1	LS	$20,000	$20,000
Sandblast & Paint Filter Gallery Piping	1	LS	$25,000	$25,000
Misc Fittings and Appurtenances	1	LS	$10,000	$10,000
Electrical	1	LS	$80,000	$80,000

Subtotal	**$679,000**
Contingency (15%)	$102,000
Contractor's OH&P and General Conditions (20%)	$156,000
Engineering, Permitting and Design (10%)	$94,000
Bidding and Construction Administration (5%)	$46,850
Administrative and Legal (2.5%)	$23,000
Project Total	**$1,100,850**

Job Name: City of Northglenn WTP MP
Job Number: 1061e
Date: 6/17/2020
By: RJA

OPINION OF PROBABLE COST
FOR
Project No. 11, Residual Solids Handling

Description	Quantity	Units	Unit Cost	Total Cost
Mobilization/Demobilization	1	LS	$100,000	$100,000
Demo and Rehabilitate Existing BW Ponds	2	EA	$150,000	$300,000
Yard Piping & Earthwork	1	LS	$120,000	$120,000
Site Improvements (Paving, etc.)	1	LS	$75,000	$75,000
Gravity Thickener Tank	1	LS	$350,000	$350,000
BW Recovery Pumping and Solids Removal	2	EA	$100,000	$200,000
Polymer Mixing and Feed System	1	LS	$40,000	$40,000
Thickening Equipment & Pumps	1	LS	$120,000	$120,000
Mechanical Dewatering & Conveyors	1	LS	$200,000	$200,000
Solids Handling Metal Building	1	LS	$300,000	$300,000
Internal Piping - Plumbing	1	LS	$120,000	$120,000
HVAC	1	LS	$50,000	$50,000
Electrical	1	LS	$150,000	$150,000
Instrumentation & Control	1	LS	$100,000	$100,000

Subtotal	**$2,225,000**
Contingency (15%)	$334,000
Contractor's OH&P and General Conditions (20%)	$512,000
Engineering, Permitting and Design (10%)	$307,000
Bidding and Construction Administration (6%)	$184,260
Administrative and Legal (2.5%)	$77,000
Project Total	**$3,639,260**

Attachment C

CITY OF NORTHGLENN

FINAL

2020 Wastewater Treatment Plant Master Plan Update

July 2021

CDM Smith

In partnership with:

INDIGO WATER GROUP

Table of Contents

List of Figures

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Appendices

Acronyms

AAD	annual average day
AAF	annual average flow
ABNR	Advanced Biological Nutrient Recovery
ATAD	aerobic thermophilic digestion
Authority	Colorado Water Resources and Power Development Authority
BDCWA	Big Dry Creek Watershed Association
BOD	Biological Oxygen Demand
BNR	biological nutrient removal
CDBG	Community Development Block Grant
CDPHE	Colorado Department of Public Health and Environment
CDPS	Colorado Discharge Permit System
CEBs	chemically enhanced backwashes
cfm	cubic feet per minute
cfs	cubic feet per second
cfu	colony forming units
CIP	Capital Improvements Plan
cip	clean-in-place
CMMS	Computerized Maintenance Management System
CRS	Colorado Revised Statutes
Commission	Water Quality Control Commission
D or Dis	Dissolved
DAF	dissolved air flotation
Division	Water Quality Control Division
DM	Daily Maximum
DMR	Discharge Monitoring Reports
DOLA	Department of Local Government Affairs
DRCOG	Denver Regional Council of Governments
E. coli	Escherichia coli
EBPR	enhanced biological phosphorus removal
EPA	US Environmental Protection Agency
FRICO	Farmers Reservoir and Irrigation Company
ft^2	square feet
GBT	gravity belt thickeners
gdp	gallons per day
gpcd	generation per capita per day
GMA	Growth Management Area
GWT	Gross-Wen Technologies
H_2S	hydrogen sulfide
HP	horsepower
HRT	hydraulic retention time
HUD	Housing and Urban Development
HVAC	heating, ventilation, and air conditioning
IGA	Intergovernmental Agreement
I/I	Inflow and infiltration
IFAS	Integrated Fixed Film Activated Sludge
IFC	International Fire Code
IWC	instream waste concentration
kPa	kilopascals
lbs/yr	pounds per year

LCOE	levelized cost of electricity
M&E	Monitoring and Evaluation
MBR	membrane bioreactor
MCL	maximum contaminant level
MDL	method detection limit
MDPF	maximum day divided by annual average flow
Metro	Metro Wastewater Reclamation District
min	minimum
MF	microfiltration
mgd	million gallons per day
mg/L	milligrams per liter
mg/m^2	milligrams per square meter
ml	milliliter
mL/g	milliliters per gram
MLSS	mixed liquor suspended solids
MM	maximum month
MMPF	maximum month peaking factor
MP	master plans
MWAT	Maximum Weekly Average Temperature
N	Nitrogen
N_2	Nitrogen Gas
NEC	National Electric Code
NFPA	National Fire Protection Association
NFRWQPA	North Front Range Water Quality Planning Association
NH_3	ammonia
NO_3	Nitrate
NO_2	Nitrite
Northglenn	City of Northglenn
NREL	National Renewable Energy Laboratory
NSA	North Service Area
O&M	Operations and Maintenance
OPCC	opinion of probable construction cost
P	phosphorus
PAX	polyaluminum chloride
PD	potentially dissolved
PEL	permissible exposure limit
PELs	preliminary effluent limits
PFAS	per- and polyfluoroalkyl substances
Plan	Wastewater Treatment Plant Master Plan Update
ppcd	pounds per capita per day
ppd	pounds per day
ppm	parts per million
PS	primary sludge
psi	pounds per square inch
PV	photovoltaic
RAB	revolving algal biofilm
RAS	Return Activated Sludge
rDON	refractory dissolved organic nitrogen
RDT	rotary drum thickeners
RO	reverse osmosis
SBR	sequencing batch reactor

scfm	standard cubic feet per minute
SDWA	Safe Drinking Water Act
Segment 1	COSPBD01 of Big Dry Creek
sNRP	soluble and non-reactive phosphorus
SPA	State Point Analysis
SRT	solids residence time
SSA	South Service Area
STAG	State and Tribal Assistance Grants
su	standard units
SVI	sludge volume index
T or Tot	total
TDS	Total Dissolved Solids
TIN	Total Inorganic Nitrogen
TMDL	Total Maximum Daily Load
TMP	transmembrane pressure
TN	Total Nitrogen
TP	Total Phosphorus
TRC	Total Residual Chlorine
TR	Total Recoverable
TSS	Total Suspended Solids
TVS	Table Value Standard
UDO	Unified Development Ordinance
UF	ultrafiltration
µg/L	micrograms per liter
µm	micrometer
USGS	US Geological Survey
UV	Ultraviolet
WAD	Weak acid dissociable
WAS	waste activated sludge
WET	Whole Effluent Toxicity
WLAs	Waste Load Allocations
WPCRF	Water Pollution Control Revolving Funds
WQA	Water Quality Assessment
WQBELs	Water Quality Based Effluent Limits
WRAP	watershed rapid assessment program
WS	Water Supply
WUSA	Wastewater Utility Service Area
WWTP	Wastewater Treatment Plant

Section 1

Executive Summary

The City of Northglenn (Northglenn) Wastewater Treatment Plant (WWTP) has a permitted capacity of 4.2 million gallons per day (mgd) and 7,916 pounds per day (ppd) Biological Oxygen Demand (BOD_5). The liquid stream of the Northglenn WWTP includes headworks with screening, degritting, and flow measurement followed by a 3-stage biological nutrient removal (BNR) activated sludge process, secondary clarification, and ultraviolet (UV) disinfection. Treated effluent may be discharged directly to either the Thompson Ditch or Big Dry Creek or be stored in Bull Reservoir prior to discharge. Bull Reservoir discharges to either Bull Canal, Thompson Ditch, or Big Dry Creek. Screenings and grit are sent to a landfill. Waste activated sludge (WAS) is transferred to one of two solids handling ponds located adjacent to Bull Reservoir and north of the mechanical treatment.

The WWTP was originally constructed in 1982 and consisted of four large, aerated ponds and Bull Reservoir. In 2007, an activated sludge process was constructed alongside the ponds to meet new effluent ammonia limitations. Northglenn intended to operate the pond system and the new mechanical facility side-by-side. However, budget constraints prevented construction of the headworks (screening and grit removal), primary clarifiers, and solids handling facilities. Two of the ponds were repurposed to serve as headworks and primary clarifiers, providing some settling of the influent solids before entering the secondary treatment system. Two other ponds were repurposed as sludge handling ponds.

In 2017, Northglenn added a headworks building with screening and grit removal to the mechanical treatment facility along with a third secondary clarifier. Operating for years without a true headworks has had a negative impact on pumps and other mechanical equipment. Additionally, the two smaller ponds that had been serving as the facility headworks were decommissioned in 2017. The ponds had reached the end of their useful life after 38 years of continuous service. Without the ponds, all of the organic load must be treated by the mechanical facility. Loss of the ponds forced a facility downgrade. The loss of capacity could prevent additional development and population growth within the city unless it can be mitigated. The decreased hydraulic and treatment capacity at the existing plant also limits the amount of flow that can be discharged into Big Dry Creek and Thompson Ditch, which may force more discharges to Bull Canal, potentially impacting Northglenn's water rights and augmentation requirements.

1.1 Purpose

The purpose of this WWTP Master Plan Update (Plan) is to build on previous Master Plan (MP) efforts to support Northglenn's commitment to providing the community with sustainable levels of service through proper operation and maintenance of its treatment facility and by planning for future rehabilitation and replacement of aging infrastructure. Northglenn is also proactively looking ahead to future regulatory requirements.

This Plan prioritizes projects that focus capital spending on the most critical assets and processes to maximize the benefits of Northglenn's reinvestment in infrastructure. This Plan should be viewed as a working document and reviewed regularly in order to adjust planning and implementation when/if regulatory updates are adopted and conditions at the treatment plant and in the City's service area change. The capital improvement plan (CIP) is intended to guide Northglenn in prioritizing projects and developing annual budgets. A number of recommendations identified in this Plan are considered essential to expand capacity and minimize odors at the plant while other recommendations should be further investigated and analyzed in the preliminary design engineering phase of each project. This Plan focuses on WWTP upgrades only, therefore collection system and nonpoint source changes and/or upgrades are not discussed in this document.

1.2 Scope

The principal issues examined as part of this Plan include the following:

- Explore alternatives to restore lost treatment capacity.
- Explore alternatives to meet anticipated nutrient limits.
- Explore alternatives for solids handling and biosolids reuse or disposal.
- Determine buildout treatment facility capacity, footprint, and setback requirements.
- Prioritize future capital reinvestments based on collaborative asset condition assessment.

Recommended improvements to maintain compliance at the WWTP are based on a review of existing treatment system performance and a conditions assessment which included onsite inspection of equipment, documentation review and discussions with operations and maintenance (O&M) personnel. Future needs for the Northglenn WWTP are estimated based on an analysis of the service areas and historic flows and loads. Treatment alternatives for meeting future regulations are also documented.

1.3 Planning Period

A 20-year planning period was selected for this Plan as Northglenn will likely achieve near full-development within that time frame. The adoption of stringent instream nutrient criteria is anticipated within the 20-year planning period, however, compliance with corresponding effluent limits at the WWTP is likely to occur beyond the planning horizon.

1.4 Flow and Load Projections

Table 1-1 summarizes future flows and loads for the Northglenn WWTP. The table presents future flows and loads for the south service area (SSA), the north service area (NSA), and a combined total for both. Northglenn is expected to reach its buildout population by the year 2040. Buildout flows and loads were estimated for the SSA based on the future anticipated population, existing per capita generation rates, and existing flow and loads peaking factors.

Planning documents provided by Northglenn indicate that development within Section 36 of the NSA will consist entirely of commercial and industrial uses. It was estimated that 367.5 acres are available within Section 36 for possible future development. A maximum month (MM) flow of 1750 gallons per day (gpd) per acre was selected for planning purposes based on guidance in Colorado's Design Criteria for Domestic Wastewater Treatment Works. This results in a potential future flow of 0.64 mgd from Section 36. The design criteria are silent on the question of how much BOD_5, total suspended solids (TSS), ammonia (NH_3), and total phosphorus (TP) commercial and industrial uses are expected to generate. While wastewater from light commercial facilities is expected to be similar in strength and composition to domestic wastewater, industrial wastewater is highly variable especially for manufacturing. Northglenn has an Environmental Protection Agency (EPA)-recognized pretreatment program (Appendix G). Industrial users would be managed under the program, which should prevent excessively high concentrations of any constituent from being discharged to the WWTP. For planning purposes, it was assumed that wastewater generated in Section 36 will be twice the strength of the domestic wastewater currently received.

Table 1-1: Estimated Future Flows and Loads

Parameter	2040 SSA	2040 Section 36	2040 Total
Annual Average Flow, mgd	4.04	-	-
Maximum Month Flow, mgd	5.13	0.64	5.78
Peak Hour Flow, mgd	10.10	-	-
BOD_5, average load, ppd	7,086	-	-
BOD_5, max month load, ppd	8,625	3,169	11,794
TSS, average load, ppd	8,847		
TSS, max month load, ppd	12,590	3,680	16,269
NH_3-N, average load, ppd	1,083	-	-
NH_3-N, max month load, ppd	1,301	478	1,778
TP, average load, ppd	255	-	-
TP, max month load, ppd	311	110	421

Note: Because peaking factors for commercial and industrial users are unknown, average daily flows and loads were not estimated for Section 36.

Development of the NSA outside of Section 36 is not anticipated to occur within the 20-year planning horizon. There are five additional sections that could be developed for residential, commercial, or industrial uses should potable water become available. Conversion of agricultural water rights to domestic water rights could make potable water available in the future. Complete development of the NSA could generate an estimated 4.5 mgd of flow in addition to the projected flows shown in Table 1-1, but this flow is not considered in the alternatives analysis conducted in this Plan.

The existing 3-stage BNR process is rated for 4.2 mgd and 7,916 ppd BOD_5 on a MM basis. The permitted capacity of the BNR process was lowered after the ponds being used for primary treatment were decommissioned in 2017. The estimated total 2040 MM flow of 5.78 mgd and BOD_5 load of 11,794 ppd both exceed the current permitted capacity. The estimated 2040 MM flow of 5.13 mgd and BOD_5 load of 8,625 ppd for the SSA also both exceed the current permitted capacity.

1.5 Project Recommendations

This Plan includes recommendations for compliance with water quality regulations, maintaining and improving the performance of the existing WWTP, and a recommended treatment plan to improve capacity while maintaining compliance with nutrient targets outlined in the Voluntary Incentives Program for Early Nutrient Reduction.

1.5.1 Regulatory Recommendations

Planning recommendations based on Northglenn's permitting cycle and the current schedule of water quality regulatory changes outlined by the Division of Water Quality (Division) in their 10-year water quality roadmap include but are not limited to:

- Compliance with Water Supply Use parameter effluent limits by 2030 based on permit incorporation in 2025 and compliance schedules as needed and appropriate.
- Compliance with *E. coli* and temperature limits by 2030 based on permit incorporation in 2025 and compliance schedules as needed and appropriate.
- Continued and maximized participation in the Voluntary Incentives Program for Early Nutrient Reduction.
- Continued participation in the Big Dry Creek Watershed Association (BDCWA), review of the BDCWA annual reports, and review of the future update to the watershed plan.
- Participation in the Division's 10-year water quality roadmap working group. Participation in the working group will keep Northglenn up to date on draft criteria, standard adoption, and anticipated timing for implementation into permits.
- Monitor developments in Per- and Polyfluoroalkyl Substances (PFAS) regulations.

The recommendations listed above do not require capital investment beyond permit monitoring and reporting requirements and labor associated with the suggested participatory activities.

1.5.2 Existing Treatment System Recommendations

An assessment team visited the WWTP during the development of this Plan. The team inspected the major equipment, reviewed documentation, and interviewed O&M personnel regarding the O&M history of the major facilities and assets.

Working with facility staff, the following pieces of equipment were identified as needing urgent replacement. Items needing urgent replacement were identified based on whether they meet one of the following characteristics: Asset failed or failure is imminent; Excessive maintenance is required; No further service life expectancy; or Significant health and safety hazard. The equipment was broken up by location within the WWTP.

- Aeration Basins:
  - Replace Mixers
  - Replace Gates
  - Replace Valves
- Process Basement - Secondary Treatment:
  - Replace Valves
- Secondary Clarifiers:
  - Replace Clarifier Mechanism

- UV Building:
  - Replace Slide Gates

Improvements to the existing WWTP in addition to equipment replacement were identified based on specific concerns raised by Northglenn. The items listed below will help maintain and potentially improve overall performance of the WWTP:

- Demolish the heat exchange equipment located in the main process building and the UV Disinfection Building. Northglenn staff is already removing the heat exchange coils from the aeration basins.
- Relocate the clarifier launders to the outside wall of the clarifiers and install new skimmer arms that extend all the way to the launders. Relocation of the launders will minimize the amount of regular maintenance required.
- Install one new dissolved oxygen (DO) probe and one nitrate (NO_3) analyzer in each aeration basin to assist with process control.
- Install two additional hydrogen sulfide (H_2S) sensors and transmitters on the main level for the headworks building for operator safety.

Additional items were identified by Northglenn, but they require additional evaluation before a recommended plan of action can be developed. Northglenn should consider in more detail the items below to determine the best course of action:

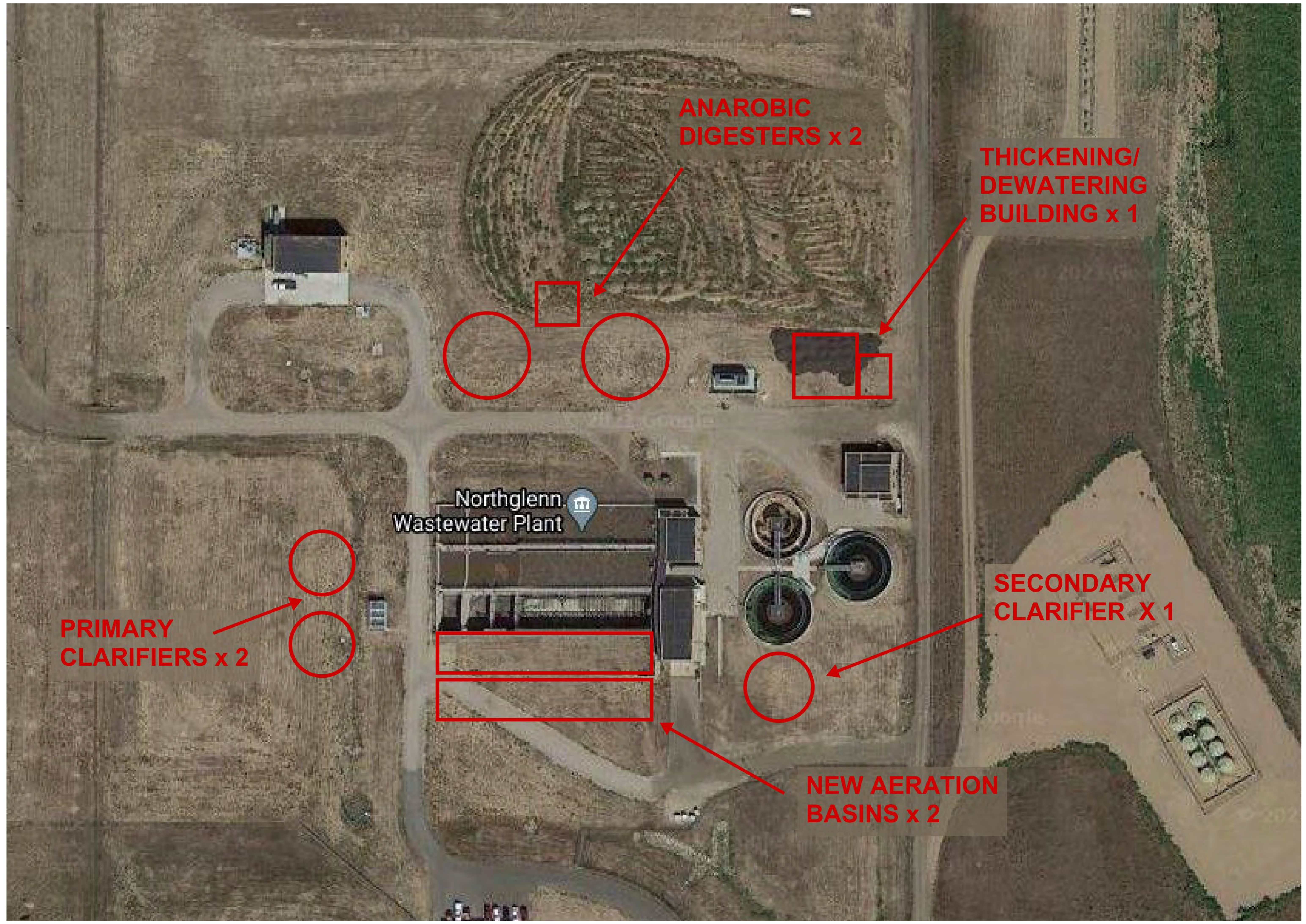
- Coordinate potential changes to the air to oil cooler with the blower manufacturer. Adequate cooling of the lubrication oil is critical to performance of the blower, and the controls for the air-cooled system are integrated into the blower local control panels. Potential relocation of the fans outdoors should consider temperature, snow and noise impacts.
- Monitor the impact of the proposed improvements for upgrading residuals handling at the water treatment plant (WTP). Since the practice of sending alum residuals to the WWTP is going to change, effluent phosphorus concentrations are likely to increase. A new chemical feed system may be needed at the WWTP to earn nutrient credits.
- Address the safety concerns associated with the existing sulfuric acid storage and feed system, including performing a detailed code review of the chemical storage area. Consider alternative acids to lower pH prior to discharge into Bull Canal. Northglenn has had difficulty procuring sulfuric acid in a timely manner, if at all, due to the chemical supply market conditions. Switching to an alternate chemical, such as citric acid, would alleviate the supply issues but increase the required size of equipment on site.

1.5.3 Recommended Treatment Upgrades

The recommended treatment process to address capacity limitations at the WWTP and to maintain compliance with current and near future regulatory requirements for nutrients includes the following improvements:

- Two 60-ft diameter primary clarifiers, a clarifier splitter box and primary sludge pumping station. The pumping station would house both primary sludge and scum pumps.
- Two additional 3-Stage BNR trains, for a total of five, configured the same as the existing trains and with the same dimensions. The new trains include all associated mechanical equipment such as anoxic mixers, fine bubble diffusers, IMLR pumps and all required process piping. No additional blower capacity is required.
- One additional 65-ft diameter secondary clarifier, for a total of four, three new return activated sludge (RAS) pumps and modifications to the RAS pump discharge header. No additional WAS pumps are required.
- Two 80-ft diameter anaerobic digesters (or equivalent volume) and all accompanying equipment such as boiler, heat exchanger, recirculation pumps, gas handling system and flare. The associated equipment would be located in a building adjacent to the digesters.
- One equalization tank so dewatering side streams can be returned over a 24-hour cycle or be used to even out the total NH_3–N load to the activated sludge process by returning more of it during periods of low flow and load.
- Two dissolved air flotation (DAF) thickeners to thicken WAS upstream of the anaerobic digesters. The DAFs would be located within a new Solids Handling Building.
- Two (1 duty/1 standby) rotary fan press dewatering units installed within the new Solids Handling Building. The rotary fan presses would dewater the thickened sludge.
- A Solids Handling Building that would house the DAFs and rotary fan presses and all associated equipment such as sludge transfer pumps, polymer feed systems, mix tanks, cake pumps and truck loading bays.

A site layout for this treatment plan is shown on **Figure 1-1**. This treatment plan is recommended based on improved energy efficiency, reduced maintenance time and expense, and the additional flexibility it offers the operators in both the liquid and solids trains. Primary treatment reduces the overall loading to the BNR process and provides another tool to manage the variability inherent in wastewater flow and loading to the plant. Primary treatment reduces the number of activated sludge basins required and the associated maintenance, such as diffuser replacement. It also removes grease, scum and heavy solids which can protect and lessen maintenance on downstream equipment. Load reduction into the secondary process decreases energy consumption of the aeration process. Anerobic digestion can be upgraded to meet class A requirements and also has the ability to produce energy rather than consume it.



ALT1 SITE

PLAN

SCALE: NTS

CDM Smith

Figure 1-1
NORTHGLENN WWTP
Site Layout for Treatment Plan

1.6 Project Financial Summary

Northglenn uses a council-manager form of government, where a publicly elected council handles the legislative duties, but hires a city manager to enforce the council's decisions. The following Capital Improvement Plan (CIP) has been developed with Northglenn WWTP staff. Two projects are recommended for implementation through the CIP over the 20-year planning cycle in this Plan. Project 1 includes replacement of existing equipment in poor condition and improvements to the existing facility addressing specific concerns from staff related to plant O&M and process performance. **Table 1-2** presents the Opinion of Probable Construction Costs (OPCC) for the replacement of equipment. The OPCC includes construction and implementation, direct and indirect costs, a construction contingency and escalation to the assumed mid-point of construction. **Table 1-3** presents the program costs for additional recommended improvements to the existing WWTP. The program costs include the OPCC as well as project contingency to account for scope development, and engineering and implementation. The combined total program cost for Project 1 is **$5,500,000**.

Table 1-2: Opinion of Probable Construction Cost to Replace Equipment – Project 1

Description	Quantity	Total Amount
Aeration Basins		
Water Treatment Equipment		
9.3 HP Submersible Mixers	3	$260,000
	Water Treatment Equipment	$260,000
Gates		
18" Sluice Gate	2	$62,000
30" Sluice Gate	3	$130,000
36" Sluice Gate	3	$150,000
42" Sluice Gate	5	$300,000
54" Sluice Gate	4	$330,000
	Gates	$970,000
Valves		
12" Butterfly Valves	3	$15,000
14" Butterfly Valves	6	$45,000
	Valves	$60,000
	Aeration Basins Subtotal	**$1,300,000**
Process Basement - Secondary Treatment		
16" MLR-BFV	6	52,000
14" MLR-PV	4	100,000
12" MLR-PV	4	72,000
12" MLR-SCV	4	47,000
16" RAS-BFV	2	17,000
14" RAS-BFV	2	15,000
10" RAS-PV	2	32,000
12" RAS-PV	6	110,000
12" RAS-SCV	3	35,000
6" WAS-PV	6	74,000
4" WAS-DCV	2	3,800
6" WAS-DCV	2	6,100

Description	Quantity	Total Amount
8" EFF-BFV	3	8,000
10" EFF-BFV	3	14,000
8" EFF-SCV	3	15,000
Process Basement - Secondary Treatment Subtotal		**$600,000**
Secondary Clarifiers		
Clarifier Mechanism	3	$2,000,000
Secondary Clarifiers Subtotal		**$2,000,000**
UV Building		
36" Slide Gate	3	$150,000
UV Building Subtotal		**$150,000**
Total Opinion of Probable Construction Cost		**$4,100,000**

Table 1-3: Costs for Recommended Improvements to Existing Treatment System – Project 1

Plant Improvements		Cost
Demolition of Heat Exchange Equipment		$50,000
Relocation of Existing Secondary Clarifier Launders		$750,000
Installation of Additional Process Control Analyzers		$90,000
H_2S Sensors		$8,300
Subtotal of Process Improvements		$848,300
Indirect Costs (Permits, Bonding and Insurance)		$42,415
Subtotal		$890,715
Contractor's Field General Conditions, Overhead and Profit	10%	$89,072
Subtotal with OH&P		$979,787
Construction Contingencies	30%	$293,936
Total Construction Costs		$1,273,722
Construction Escalation to Mid-Point of Construction	5%	$63,686
Total Opinion of Probable Construction Cost (Rounded)		$1,000,000
Project Contingency	20%	$200,000
Subtotal		$1,200,000
Engineering and Implementation	20%	$240,000
Total Program Cost		$1,440,000
TOTAL PROGRAM COST (Rounded)		**$1,400,000**

Project 2 includes facility upgrades to increase capacity to accommodate current and future flows and loads. Project 2 was broken down into two phases for implementation. Phase 1 consists of installing primary clarifiers and new solids handling processes to increase treatment capacity of the WWTP. Construction of Phase 1 upgrades is recommended as soon as possible to accommodate current flows and loads since the plant is already operating near capacity. Phase 1 upgrades would also account for any additional loads associated with development in the SSA but not necessarily the full flow increase. **Table 1-4** presents the program costs for Project 2, Phase 1 upgrades.

Table 1-4: Costs for Phase 1 of WWTP Capacity Upgrades - Project 2

Process Improvements		Costs
Primary Clarifiers and Splitter Box		$5,120,000
Digesters		$5,100,000
Solids Handling Building		$7,300,000
Equalization Tank		$400,000
Yard Piping		$2,000,000
Electrical and Controls		$3,200,000
Subtotal of Process Improvements		$23,130,000
Indirect Costs (Permits, Bonding and Insurance)	5%	$1,156,500
Subtotal		$24,286,500
Contractor's Field General Conditions, Overhead and Profit	10%	$2,428,650
Subtotal with OH&P		$26,715,150
Construction Contingencies	30%	$8,014,545
Total Construction Costs		$34,729,695
Construction Escalation to Mid-Point of Construction	11.25%	$3,907,090
Total Opinion of Probable Construction Cost (Rounded)		$38,636,785
Project Contingency	20%	$7,727,357
Subtotal		$46,364,142
Engineering and Implementation	20%	$9,272,828
Total Program Cost		$55,636,971
Total Program Cost (Rounded)		**$56,000,000**

Project 2, Phase 2 upgrades are required to meet the projected flows and loads associated with the development of Section 36 within the NSA in addition to the development that will occur in the SSA. If it is determined that Section 36 will not be developed within the next 20 years, a more detailed flow study should be conducted to determine if the Phase 2 upgrades are required. The capacity increase associated with the Project 2, Phase 1 upgrades is greater than the projected 2040 influent loads for the SSA but not the projected 2040 flows for the SSA. Assuming that Section 36 will be developed, **Table 1-5** presents the program costs for the Project 2, Phase 2 upgrades. The combined total program cost for Project 2 is **$93,000,000**. This is approximately 6% higher than if both phases were constructed at the same time. However, the recommended phasing provides Northglenn the opportunity to reassess development plans in the service areas, evaluate the performance of the upgraded WWTP after Project 2, Phase 1, and spread the capital cost burden over more time.

It is recommended that a rate study be conducted as soon as possible to plan for the two projects included in the CIP. Funding opportunities from federal and local grants and loans are presented in the document.

Table 1-5: Costs for Phase 2 of WWTP Capacity Upgrades - Project 2

Process Improvements		Costs
Aeration Basins		$6,200,000
Secondary Clarifiers		$2,900,000
WAS and RAS Pumps		$310,000
Yard Piping		$1,765,000
Electrical and Controls		$2,490,000
Subtotal of Process Improvements		$13,665,000
Indirect Costs (Permits, Bonding and Insurance)	5%	$683,250
Subtotal		$14,348,250
Contractor's Field General Conditions, Overhead and Profit	10%	$1,434,825
Subtotal with OH&P		$15,783,075
Construction Contingencies	30%	$4,734,923
Total Construction Costs		$20,517,998
Construction Escalation to Mid-Point of Construction	26.25%	$5,385,974
Total Opinion of Probable Construction Cost (Rounded)		$26,000,000
Project Contingency	20%	$5,200,000
Subtotal		$31,200,000
Engineering and Implementation	20%	$6,240,000
Total Program Cost		$37,440,000
Total Program Cost (Rounded)		**$37,000,000**

The projected rates for solids disposal for 2040 were used to estimate the costs for onsite and offsite disposal for the recommended treatment plan. Disposal costs are shown for both 2020 rates and projected 2040 rates in **Table 1-6**.

Table 1-6: Projected Annual Biosolids Disposal Costs

Price Year	Onsite Rates $/Dry Ton	Offsite Rates $/Dry Ton	Dry Ton/yr Produced	Onsite Disposal Cost $/yr	Offsite Disposal Cost $/yr
2020	$342.0	$410.0	1400	$480,000	$570,000
2040	$445.0	$534.0	1400	$620,000	$750,000

1.7 Implementation Schedule

Table 1-7 presents a summary of the CIP, including an implementation schedule. Project 1 addresses immediate needs at the existing WWTP and should commence as soon as possible. It is recommended that this project should be completed within the next permit cycle to avoid future permit violations.

There have been no permit violations caused by capacity or BOD loading issues prior to 2020. However, when reviewing discharge monitoring reports (DMRs) from the last 10 years and comparing the data to the most recently issued permit limits, the WWTP would have exceeded 80% of the current permitted capacity of 4.2 mgd for 22 months and would have exceeded 95% of the current permitted capacity for 6 additional months. According to Northglenn's discharge permit, the city is required to initiate financial planning for the expansion of the WWTP when the 30-day average flows exceed 80% of the treatment capacity. When the 30-day average flows reach 95% of the capacity, construction There have been no permit violations caused by capacity or BOD loading issues prior to 2020. However, when reviewing discharge monitoring reports (DMRs) from the last 10 years and comparing the data to the most recently issued permit limits, the WWTP would have exceeded 80% of the current permitted capacity of 4.2 mgd for 22 months and would have exceeded 95% of the current permitted capacity for 6 additional months. According to Northglenn's discharge permit, the city is required to initiate financial planning for the expansion of the WWTP when the 30-day average flows exceed 80% of the treatment capacity. When the 30-day average flows reach 95% of the capacity, construction of facility upgrades must commence or issuance of building permits within the municipality which will contribute to the increase of flow to the facility must cease until construction of upgrades has commenced (CDPHE, 2019). In addition, historical data show that the WWTP exceeded 80% of influent BOD_5 loading capacity 32 months over the last 10 years and exceeded 95% during November of 2019 when compared to current permit limits. of facility upgrades must commence or issuance of building permits within the municipality which will contribute to the increase of flow to the facility must cease until construction of upgrades has commenced (CDPHE, 2019). In addition, historical data show that the WWTP exceeded 80% of influent BOD_5 loading capacity 32 months over the last 10 years and exceeded 95% during November of 2019 when compared to current permit limits. Therefore, it is recommended that the construction of Phase 1 of the Project 2 upgrades begin as soon as possible also to allow development within the service area to continue. It is recommended that Project 2, Phase 1 be completed early within the next permit cycle. Phase 2 of the Project 2 upgrades are required to accommodate full buildout of the SSA and development of Section 36 within the NSA. The target completion data for Project 2, Phase 2 is December 2031 to allow for reassessment of the development plans in the service areas. If it is determined that Section 36 will not be developed within the next 20 years, a more detailed flow study should be conducted to determine if the Phase 2 upgrades are even required.

Table 1-7: Capital Improvements Summary

Project Description	Estimated Cost	Year										
		2021	2022	2023	2024	2025	2026	2027	2028	2029	2030	2031
Project 1												
Existing Facility Equipment Replacements	$4,100,000	Start			Completion							
Existing Treatment Facility Improvements	$1,450,000	Start			Completion							
Project 2												
Facility Capacity Upgrades Phase 1	$56,000,000		Begin Design	Finish Design	Begin Construction		Completion					
Facility Capacity Upgrades Phase 2	$37,000,000								Begin Design	Begin Construction		Completion

Section 2

Introduction

The City of Northglenn (Northglenn) has invested hundreds of millions of dollars in its water, wastewater, and stormwater infrastructure. Northglenn is committed to providing the community with sustainable levels of service through proper operation and maintenance of these valuable resources and by planning for future rehabilitation and replacement of aging infrastructure. Northglenn must also plan to meet future regulatory requirements. By prioritizing projects that focus capital spending on the most critical assets and processes first, this Wastewater Treatment Plant Master Plan Update (Plan) will maximize the benefits of Northglenn's reinvestment in infrastructure.

This Plan builds upon the efforts of previous master plans (MPs): Wastewater Utility Plan (Integra, 2003) and subsequent updates (HDR, 2011 and 2012). These documents can be found in **Appendices A and B**, respectively. Some information required for planning agency review of this document has not changed from the previous documents. This information has been noted in the text or references have been made to text or figures in the appendices.

2.1 General Background

General background information for Northglenn is unchanged from previous MPs and is provided in this section. Population information has been updated.

Northglenn is located in Adams County, Colorado. The community of Northglenn was established in 1959 at the intersection of Interstate Highway I-25 and 104th Avenue. The Northglenn subdivision became the City of Northglenn on April 18, 1969. Northglenn encompasses an area of approximately 7 square miles and has an estimated population of 38,973 (U.S. Census Bureau, 2019). The city boundaries extend from 95th Avenue north to 124th Avenue, and from Madison Way west to Zuni Street. Interstate Highway I-25 runs through the geographic center of the city in a north to south direction.

Northglenn uses a council-manager form of government, where a publicly elected council handles the legislative duties, but hires a city manager to enforce the council's decisions.

Northglenn's wastewater utility service area (WUSA) includes two separate geographical areas. The South Service Area (SSA) consists of the area within Northglenn corporate boundaries and three Enclaves within the City of Thornton adjacent to the southern boundary of Northglenn. The North Service Area (NSA) consists of Section 36 (also incorporated) and adjacent land located in Weld County east of the Northglenn wastewater treatment plant (WWTP). The WUSA boundaries are shown on **Figure 2-1**.

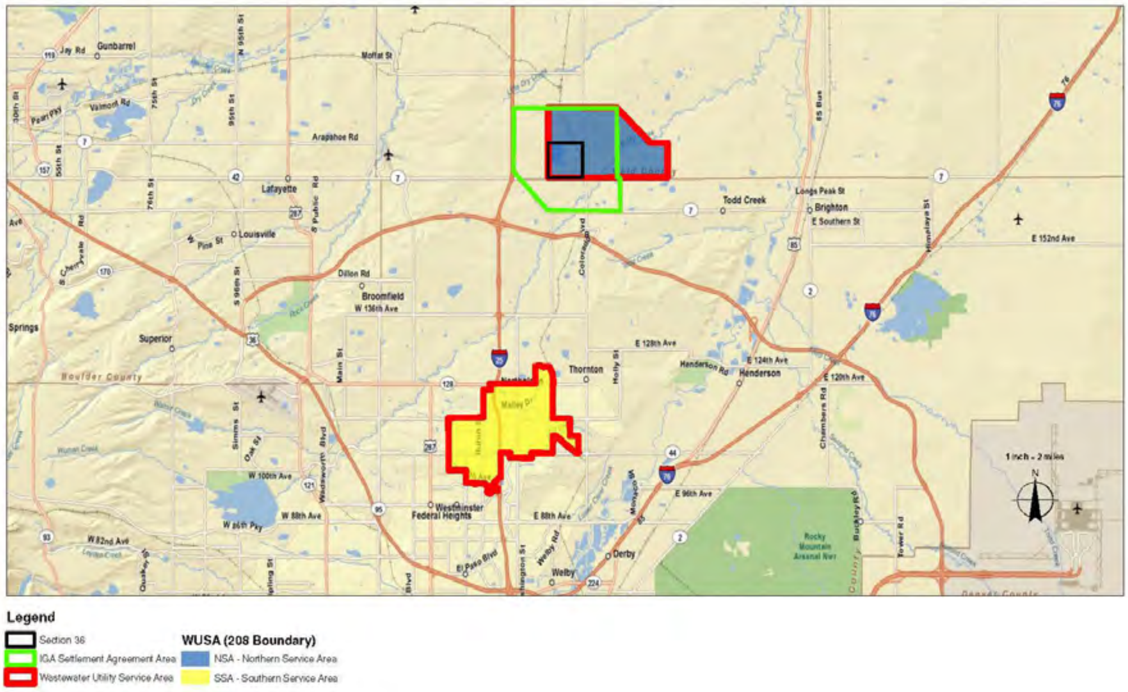


Figure 2-1: Wastewater Utility Service Area (Source: HDR, 2012)

The Northglenn WWTP was originally planned as a regional facility to serve the city and new developments in the area north of 100th Avenue and east of Interstate Highway I-25. The original planning documents anticipated future flows of approximately 12 mgd (million gallons per day). Intergovernmental agreements (IGAs) were developed that would allow wastewater from portions of Broomfield and Thornton to be treated at the Northglenn WWTP. However, Broomfield and Thornton have gradually extended their wastewater service area boundaries to cover these IGA areas. The Northglenn WWTP could still serve developing parts of Broomfield and Thornton if the cities choose to implement the IGAs.

2.2 Facility Plan Objectives

Two key drivers for this facility plan update are related to facility capacity and regulatory planning. These are described below.

2.2.1 Capacity

The Northglenn WWTP was originally constructed in 1982 and consisted of four large, aerated ponds and Bull Reservoir. The pond system was designed for an average annual flow of 4.6 mgd but was subsequently re-rated for 6.5 mgd and 11,384 pounds per day (ppd) of biochemical oxygen demand (BOD_5). In 2007, an activated sludge process was constructed alongside the ponds to meet new effluent ammonia limitations. With an eye to the future, Northglenn wisely included provisions for both denitrification and biological phosphorus removal.

Northglenn intended to operate the pond system and the new mechanical facility side-by-side, thus providing a combined treatment capacity of 11.3 mgd. However, budget constraints prevented construction of the headworks (screening and grit removal), primary clarifiers, and solids handling facilities. Two of the ponds were repurposed to serve as headworks and primary clarifiers, providing some settling of the influent solids before entering the secondary treatment system. Two other ponds were repurposed as sludge handling ponds. The pond-mechanical hybrid treatment system retained its rating of 6.5 mgd and increased loading capacity slightly to 12,650 ppd of BOD_5.

In 2017, Northglenn added a headworks building with screening and grit removal to the mechanical treatment facility along with a third, 65-foot diameter, secondary clarifier. Operating for years without a true headworks has had a negative impact on pumps and other mechanical equipment. A condition assessment was completed as part of this Plan to determine which pieces of equipment may need to be replaced as a result.

The two smaller ponds that had been serving as the facility headworks were decommissioned in 2017. The ponds had reached the end of their useful life after 38 years of continuous service. The ponds had been removing and treating about 30 – 35% of the total BOD_5 load entering the facility. Without them, all of the organic load must be treated by the mechanical facility. Loss of the ponds forced a facility downgrade from 6.5 mgd and 12,650 ppd of BOD_5 to 4.2 mgd and 7,916 ppd of BOD_5. This is a derating of the plant capacity by 37%. The loss of capacity could prevent additional development and population growth within the city unless it can be mitigated. In recent months, the plant has exceeded 80% of capacity and is under construction of a new large development (Karl's Farm).

The decreased hydraulic and treatment capacity at the existing plant limits the amount of flow that can be discharged into Big Dry Creek and Thompson Ditch, which may force more discharges to Bull Canal, potentially impacting Northglenn's water rights and augmentation requirements.

2.2.2 Future Regulations

In addition to regaining lost capacity, Northglenn is aware that water quality regulations are trending toward more stringent effluent limits for trace metals and nutrients. Specific areas of concern include total inorganic nitrogen (TIN), total phosphorus (TP), new temperature regulations, whole effluent toxicity (WET) testing changes, trace metals, per- and polyfluoroalkyl substances (PFAS), disinfection byproducts, and total dissolved solids (TDS).

2.2.3 Objectives Summary

Given the history provided above, the principal issues examined as part of this Plan include the following:

- Explore alternatives to restore lost treatment capacity.
- Explore alternatives to meet anticipated nutrient limits.
- Explore alternatives for solids handling and biosolids reuse or disposal.
- Determine buildout treatment facility capacity, footprint, and setback requirements.
- Prioritize future capital reinvestments based on collaborative asset condition assessment.

A 20-year planning period was selected for this Plan as Northglenn will likely achieve near full-development within that time frame. This Plan focuses on WWTP upgrades only, therefore collection system and nonpoint source changes and/or upgrades are not discussed in this document.

2.3 General Format of Plan

This Plan follows the general format provided by the North Front Range Water Quality Planning Association (NFRWQPA) 2019 Utility Plan Guidance Document (NFRWQPA, 2019) with some adjustments in section order and contents. **Table 2-1** contains the checklist provided by the NFRWQPA and provides information on where to find checklist items throughout this Plan. Table 2-1 also notes items that are incorporated through reference as well as items not included. This Plan focuses on WWTP upgrades, includes watershed information in the context of permitting and regulations only, and does not include detailed discussion of the collection system and other nonpoint sources within Northglenn's watershed.

Table 2-1: NFRWQPA Utility Plan Outline Checklist

NFRWQPA Outline	Page Number	Notes
Section 1: Executive Summary	Section 1: Executive Summary	
1. Purpose	1-1	
2. Scope	1-2	
3. Planning Period	1-2	
4. Project Recommendations	1-3	
5. Project Financial Summary	1-8	
6. Implementation Schedule	1-11	
Section 2: Introduction	Section 2: Introduction	
1. General Background of Entity	2-1	
2. Facility Planning Summary	2-2	
3. General Format of Utility Plan	2-3	
Section 3: Existing Conditions	Section 4: Existing Conditions	
1. Current Planning Service Area	4-1	
2. Current Wastewater Flows and Loads	4-5	
Influent Flows	4-5	
Historical Wastewater Loads	4-8	
Current Effluent Limitations	3-1	Included in Regulatory Summary
TMDL Loads	3-8	Included in Regulatory Summary
3. Existing Wastewater Treatment System	4-16	
Description of the Existing System	4-16	
Performance of Existing System	4-30	
Existing Air Quality Permit	-	Not Included in Plan
Existing Stormwater Management Plan	-	Not Included in Plan
Existing Site Characterization	-	Not Included in Plan
Existing Emergency Response Protocols	-	Not Included in Plan
Existing Biosolids Management Program	4-40	
Condition of Existing Treatment System	4-46	

NFRWQPA Outline		Page Number	Notes
	Recommendations for Treatment System and Biosolids Program Improvements	4-50	
4.	Existing Collection System	4-19	
	Existing Layout	Figure 4-5	
	Existing Lift Stations	-	Not Included in Plan
	Existing Condition Assessment of Collection System	-	Not Included in Plan
	Pretreatment Program		Appendix G
	Recommendations for Collection System Improvements	-	Not Included in Plan
5.	Existing Service Area Nonpoint Source Contributions	-	Not Included in Plan
	Existing Nonpoint Sources and Stormwater Sewer Collection Systems	-	Not Included in Plan
	Existing Nonpoint Source Loads	-	Not Included in Plan
	Recommendations for Existing Nonpoint Source Improvements	-	Not Included in Plan
Section 4: Future Conditions		Section 5: Future Conditions	
1.	Population and Land Use Projections	5-1	
2.	Flow and Load Forecasts	5-2	
3.	Projected Wastewater Flow Characterization	Table 5-1	
	Wastewater Flow Projections	5-2	
	Projected I&I Analysis	5-2	
	Typical Wastewater Flow Contributions for Planning Projections	5-2	
	Future Design Loadings for Constituents of Concern	5-4	
4.	Future Interceptor of Lift Station Collection System Alignments	-	Not Included in Plan
5.	Future Service Area Nonpoint Source Contributions	-	Not Included in Plan
Section 5: Receiving Stream Water Quality		Section 3: Regulatory Summary	
1.	Watershed Identification	3-9	
	Ambient Water Quality	3-13	
	Watershed Issues	3-14	Table 3-10
	Map of Watershed Basin	3-9	Figure 3-2
2.	Total Maximum Daily Loads	3-15	
3.	Future Level of Treatment Required	3-16	
	Preliminary Effluent Limits (PELs)	NA	Page 3-17 includes discussion of "Potential Effluent Limits"
	Notice of Authorization	NA	
	Water Quality Planning Targets	3-22	
4.	Point and Nonpoint Contributions of the River Basin	-	Not Included in Plan

NFRWQPA Outline	Page Number	Notes
WWTF Point Source Contributions (lbs/yr for 3 years)	-	Not Included in Plan
Service Area Nonpoint Contributions (lbs/yr for 1-3 years)	-	Not Included in Plan
MS4 Permit	-	Not Included in Plan
5. Consideration for Modification of Standards	-	Not Included in Plan
Section 6: Wastewater Treatment and Collection System Improvements	Section 6: Alternatives Analysis	Does not include Collection System Improvements
1. Development and Screening of Treatment ~~and Collection System Improvement~~ Alternatives	6-5	
Optimization of Existing Facility	4-51	Included at end of Existing Conditions
Regional Consolidation as an Alternative	-	Not Included in Plan
Alternatives for Wastewater Reuse Opportunities	-	Not Included in Plan
Treatment ~~or Collection System~~ Alternatives	6-9	
2. Treatment ~~or Collection System~~ Evaluation Matrix	6-34	
3. Treatment ~~or Collection~~ Improvement Alternative Selection	6-34	
Alternative Plan Selection Matrix or Other Process		Plan includes cost and non-cost considerations
Selected Treatment ~~or Collection System Improvements~~ Description	6-38	
Emergency Standby Power System	-	Not Included in Plan
Odor Control Considerations	-	Odor considerations discussed throughout Plan
Air Quality Requirements	-	Not Included in Plan
Site Stormwater Management Plan	-	Not Included in Plan
Site Map	Figures 6-9 and 10	
Site Characteristics	Figures 6-9 and 10	
NEPA Components	NA	
Record of Public Participation in the Plan Selection Process	-	Not Included in Plan
Section 7: Service Area Nonpoint Source Improvements	-	Not Included in Plan
Section 8: System Management and Financial Plan	Section 7: Capital Improvements Plan	
1. Wastewater Management Plan	7-1	
Management Structure	-	Not Included in Plan
Provisions for Operation and Maintenance	-	Not Included in Plan
Implementation Schedule for Projects	7-9	Tables 7-3 and 7-4
2. Arrangements for Implementation	-	Not Included in Plan
Control of Site-Ownership Documentation	NA	

NFRWQPA Outline	Page Number	Notes
IGAs	2-2	IGAs discussed in Section 2.1
3. Financial Management Plan	7-10	
Financing for Proposed Project	7-10	
User Charge Rate Studies	-	Rate Study Recommended
Sewer Tap (PIFs) Rate Studies	-	Not Included in Plan
State Revolving Loan Fund	7-10	
20-year Financial Graph	-	Not Included in Plan
Section 9: NFRWQPA Regional 208 Data Summary		
1. Agency Point Source Inventory Datasheet	-	Not Included in Plan
2. eRAMS CLEAN Dashboard Report	-	Not Included in Plan
3. eRAMS Watershed Rapid Assessment Program (WRAP)	-	Not Included in Plan
Appendices		
A. Utility Plan Check List	2-4	Included at end of Section 2
B. Reports and Special Studies	Appendix A/B	Previous MP and MP Update
C. Legal Description of Site and Deed	Appendix E	
D. Copies of Agency Contact Letters		NA
E. Special Surveys (Environmental or Endangered Species)		NA
F. Site Characterization: Wetlands, Flood Plain, Soil Reports, Geology		NA
G. Copy of PELs or NOA Report		NA
H. Copy of Current Effluent Permit Requirements	Appendix C	Permit and Fact Sheet
I. Planning and Zoning Information	Appendix D	
J. Copies of IGAs	-	See previous MPs
K. User Charge Studies	-	See previous MPs
L. Air Quality Permit	-	See previous MPs
M. Odor Control Studies or Plans		Incorporated through reference
N. Site Stormwater Management Plan – Permit		NA
O. Minutes of Public Hearing or Record of Public Meetings		NA
P. I&I Studies	-	Not Included in Plan
Q. Copy of Pretreatment Program	Appendix G	
R. CLEAN Report	-	Not Included in Plan
S. WRAP Report	-	Not Included in Plan
T. Three (3) year history of all Division Notice of Violation(s)/Cease and Desist Orders	-	Not Included in Plan

Section 3

Regulatory Summary

Regulatory requirements for water quality dictate the levels to which wastewater is treated and drive the selection process for treatment options. Effluent from the Northglenn WWTP is permitted under Colorado Discharge Permit System (CDPS) Permit Number CO0036757 (Appendix C). The permit covers effluent discharges to Big Dry Creek, Bull Canal, and Thompson Ditch within the South Platte River Basin. Colorado regulations applicable to the discharge are:

- Regulation 31: The Basic Standards and Methodologies for Surface Water
- Regulation 38: Classifications and Numeric Standards for South Platte River Basin, Laramie River Basin, Republican River Basin, Smoky Hill River Basin
- Regulation 62: Regulations for Effluent Limitations
- Regulation 85: Nutrients Management Control Regulation

The discharge permit was most recently renewed by the Water Quality Control Division (Division) in November 2019, went into effect on January 1, 2020, and is set to expire December 31, 2024. The permit was amended on March 31, 2021 to allow for the addition of ferric chloride and went into effect May 1, 2021. Recent regulatory updates to the classifications of COSPBD01 (Segment 1) of Big Dry Creek and the resulting changes to a number of applicable water quality standards will likely affect monitoring requirements and/or effluent limits in the next permit renewal. The current effluent limitations, existing total maximum daily loads (TMDLs), receiving water quality and applicable standards, and anticipated future regulatory updates are all discussed in this section.

3.1 Current Effluent Limitations

Discharges from the following outfalls are covered under Permit CO0036757 (**Figure 3-1**):

- Outfall 001A following disinfection and prior to mixing with Bull Canal at Latitude 40.014643 °N, Longitude 104.959527 °W
- Outfall 002A combined flow calculation to Big Dry Creek
- Outfall 003A combined flow calculation to Bull Canal and Thompson Ditch
- Outfall 004A following disinfection and prior to mixing with Big Dry Creek with flow to the South Platte River at Latitude 40.014643 °N, Longitude 104.959527 °W
- Outfall 005A following disinfection and prior to mixing with Thompson Ditch at Latitude 40.014643 °N, Longitude 104.959527 °W
- Outfall 006A following disinfection and prior to mixing with Thompson Ditch at Latitude 40.002728 °N, Longitude 104.952180 °W

- Outfall 007A following disinfection and prior to mixing with Big Dry Creek with flow to the South Platte River at Latitude 40.002728 °N, Longitude 104.952180 °W

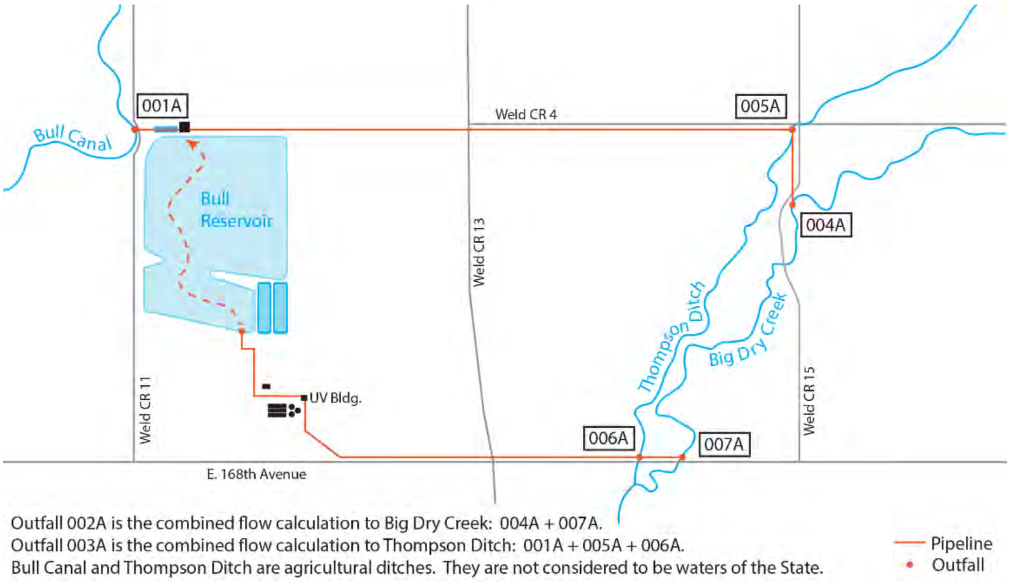


Figure 3-1: Outfall Locations

The locations provided above serve as the points of compliance for the permit and are located after all treatment has occurred and prior to discharge to the receiving waters. An additional monitoring station is included in the permit for monitoring ambient upstream temperature in Big Dry Creek:

- UST1A is located upstream from the facility discharge at Latitude 40.000055°N, Longitude 104.930429°W.

The WWTP is permitted to discharge 4.2 mgd to Big Dry Creek. Bull Canal and the Thompson Ditch are used to transfer water for agricultural use under an agreement between Northglenn and Farmers Reservoir and Irrigation Company (FRICO). This agreement will end in 2023. Discharges occur when water levels in Bull Reservoir reach a predetermined level or when there is a call for water through the Division of Water Resources/Office of the State Engineer. Discharges may also be used to satisfy Northglenn's augmentation plan.

Discharges to Bull Canal and the Thompson Ditch (001A, 005A, and 006A) are monitored for flow, pH, Escherichia coli (*E. coli)*, BOD_5, total suspended solids (TSS), and Oil and Grease. A maximum of 43.2 mgd average daily flow (30-day average) may be discharged through these outfalls. Monitoring for other parameters is not required as Bull Canal and Thompson Ditch are not considered to be waters of the state.

Discharges to Big Dry Creek (004A and 007A) eventually reach the South Platte River. They have the potential to impact aquatic life, drinking water supplies, and recreation. Discharges to Big Dry Creek are monitored for total residual chlorine (TRC), metals, sulfide, sulfate, nonylphenol, and nutrients. Chronic WET testing is conducted quarterly. A maximum of 4.2 mgd (30-day average) may be discharged through these outfalls. Bull Reservoir provides storage of final effluent and is not part of the treatment process.

Tables 3-1 through **3-5** contain the effluent limits and monitoring requirements found in Permit CO0036757. Note that effluent limits were calculated in the permit based on a combined modeling of discharges from the Broomfield, Westminster, and Northglenn WWTPs which all discharge to the same receiving water. The Broomfield and Westminster WWTPs discharge approximately 12 and 9 miles upstream of the Northglenn WWTP, respectively.

Table 3-1: Effluent Flow Limit and Monitoring Requirement for Outfall 002A (Calculated from a combination of Outfall 004A and 007A – Big Dry Creek)

ICIS Code	Effluent Parameter	Effluent Limitations Maximum Concentrations			Monitoring Requirements	
		30-Day Average	7-Day Average	Daily Maximum	Frequency	Sample Type
50050	Effluent Flow (mgd)	4.2		Report	Daily	Calculated

Table 3-2: Effluent Limits and Monitoring Requirements for Outfalls 004A and 007A (Big Dry Creek)

ICIS Code	Effluent Parameter	Effluent Limitations Maximum Concentrations			Monitoring Requirements	
		30-Day Average	7-Day Average	Daily Maximum	Frequency	Sample Type
50050	Effluent Flow (mgd)	Report		Report	Continuous	Recorder
00010	Temperature Daily Maximum (DM) (°C) March-November			Report	Continuous	Recorder
00010	Temperature DM (°C) December-February			Report	Continuous	Recorder
00010	Temperature Maximum Weekly Average Temperature (MWAT) (°C) March-November		Report		Continuous	Recorder
00010	Temperature MWAT (°C) December-February		Report		Continuous	Recorder
400	pH (su)			6.5-9.0	Daily	Grab
51040	*E. coli* (#/100 mL)	205	410		Weekly	Grab
50060	Total Residual Chlorine (mg/L)	0.030		0.021	3 days/week	Grab
00640	Total Inorganic Nitrogen as N (mg/L), until June 30, 2024			15	3 days/week	Composite
00640	Total Inorganic Nitrogen as N (mg/L), beginning July 1, 2024			14	3 days/week	Composite
00610	Total Ammonia as N (mg/L)					
	January	16		19	2 days/week	Composite
	February	17		20	2 days/week	Composite
	March	13		16	2 days/week	Composite
	April	3.6		8.8	2 days/week	Composite
	May	10		28	2 days/week	Composite
	June	2.8		8.0	2 days/week	Composite
	July	3.9		12	2 days/week	Composite
	August	4.1		12	2 days/week	Composite
	September	7.8		25	2 days/week	Composite

ICIS Code	Effluent Parameter	Effluent Limitations Maximum Concentrations			Monitoring Requirements	
		30-Day Average	7-Day Average	Daily Maximum	Frequency	Sample Type
	October	5.0		10	2 days/week	Composite
	November	13		18	2 days/week	Composite
	December	14		20	2 days/week	Composite
310	BOD_5, effluent (mg/L)	30	45		Weekly	Composite
81010	BOD_5 (% removal)	85 (min)			Weekly	Calculated
530	TSS, effluent (mg/L)	30	45		3 Days/Week	Composite
81011	TSS (% removal)	85 (min)			3 Days/Week	Calculated
84066	Oil and Grease (visual)			Report	Daily	Visual
3582	Oil and Grease (mg/L)			10	Contingent	Grab
00978	Arsenic, TR (µg/L)	Report			Quarterly	Composite
00998	Beryllium, TR (µg/L)	Report			Quarterly	Composite
01220	Chromium+6, Dis (µg/L)	Report		Report	Quarterly	Grab
00718	Cyanide, WAD (µg/L)			5.0	2 times/ month	Grab
00980	Iron, TR (µg/L)	1,013			2 times/ month	Composite
01046	Iron, Dis (µg/L)	865			2 times/ month	Composite
01056	Manganese, Dis (µg/L)	384			2 times/ month	Composite
01319	Manganese, PD (µg/L)	Report		Report	Quarterly	Composite
01129	Molybdenum, TR (µg/L)	Report			Quarterly	Composite
50286	Mercury, Tot (low level) (µg/L)	Report			Quarterly	Composite
82057	Boron, Tot (mg/L)	Report			2 times/ month	Composite
51202	Sulfide as H_2S (mg/L)	Report			2 times/ month	Composite
00940	Chloride (mg/L)	477			2 times/ month	Composite
81020	Sulfate (mg/L)	4002			2 times/ month	Composite
51568	Nonylphenol (µg/L)	Report		Report	Monthly	Grab
	WET, chronic until December 31, 2021					
TKP6C	Static Renewal 7 Day Chronic *Pimephales promelas*			Report	Quarterly	3 Composites /Test
TKP3B	Static Renewal 7 Day Chronic *Ceriodaphnia dubia*			Report	Quarterly	3 Composites /Test
	WET, chronic beginning January 1, 2022					

ICIS Code	Effluent Parameter	Effluent Limitations Maximum Concentrations			Monitoring Requirements	
		30-Day Average	7-Day Average	Daily Maximum	Frequency	Sample Type
TKP6C	Static Renewal 7 Day Chronic *Pimephales promelas*			NOEC or IC25 ≥ IWC*	Quarterly	3 Composites /Test
TKP3B	Static Renewal 7 Day Chronic *Ceriodaphnia dubia*			NOEC or IC25 ≥ IWC*	Quarterly	3 Composites /Test

TR = Total Recoverable WAD = Weak Acid Dissociable Dis = Dissolved PD = Potentially Dissolved Tot = Total *IWC = 35% min=minimum

Table 3-3: Nutrient Effluent Limits and Monitoring Requirements for Outfalls 004A and 007A (Big Dry Creek)

ICIS Code	Effluent Parameter	Effluent Limitations Maximum Concentrations[1]		Monitoring Requirements	
		Running Annual Median*	95th Percentile**	Frequency	Sample Type
00640	Total Inorganic Nitrogen as N (mg/L) until 12/31/2020	Report	Report	Monthly	Composite
00640	Total Inorganic Nitrogen as N (mg/L) beginning 1/1/2021	15	20	Monthly	Composite
00665	Total Phosphorus (mg/L) until 12/31/2020	Report	Report	Monthly	Composite
00665	Total Phosphorus (mg/L) beginning 1/1/2021	1.0	2.5	Monthly	Composite

*Reported as a running annual median, which is a median of all samples collected in the most recent 12 calendar months including samples collected in accordance with Regulation 85.

**Reported as the 95th percentile of all samples taken in the most recent 12 calendar months including samples collected in accordance with Regulation 85.

[1]Note that 12 months of data collection after the effective date is needed prior to the reporting with a delay in the numeric limitation until 12 months of data has been collected.

Table 3-4: Effluent Limits and Monitoring Requirements for Outfall 003A (Calculated from a combination of Outfall 001A, 005A, and 006A – Bull Canal and Thompson Ditch)003A (Calculated from a combination of Outfall 001A, 005A, and 006A – Bull Canal and Thompson Ditch)

ICIS Code	Effluent Parameter	Effluent Limitations Maximum Concentrations			Monitoring Requirements	
		30-Day Average	7-Day Average	Daily Maximum	Frequency	Sample Type
50050	Effluent Flow (mgd)	43.2		Report	Daily	Calculated

Table 3-5: Effluent Limits and Monitoring Requirements for Outfalls 001A, 005A, and 006A (Bull Canal and Thompson Ditch)

ICIS Code	Effluent Parameter	Effluent Limitations Maximum Concentrations			Monitoring Requirements	
		30-Day Average	7-Day Average	Daily Maximum	Frequency	Sample Type
50050	Effluent Flow (mgd)	Report		Report	Continuous	Recorder
400	pH (su)			6.0-9.0	Daily	Grab
51040	*E. coli* (#/100 mL)	2,000	4,000		Weekly	Grab

ICIS Code	Effluent Parameter	Effluent Limitations Maximum Concentrations			Monitoring Requirements	
		30-Day Average	7-Day Average	Daily Maximum	Frequency	Sample Type
310	BOD5, effluent (mg/L)	30	45		Weekly	Composite
81010	BOD5 (% removal)	85 (min)			Weekly	Calculated
530	TSS, effluent (mg/L)	30	45		3 Days/Week	Composite
81011	TSS (% removal)	85 (min)			3 Days/Week	Calculated
84066	Oil and Grease (visual)			Report	Daily	Visual
3582	Oil and Grease (mg/L)			10	Contingent	Grab

In addition to the monitoring and reporting requirements listed above, requirements for continuous ambient temperature monitoring were added effective March 31, 2020. Ambient temperature data may be used to establish ammonia (NH_3) and/or temperature limits in future permit renewals. Ambient temperature data is reported under outfall UST1A in the permit.

Changes to effluent limits and monitoring requirements from the previous permit include:

- NH_3 is now required at a twice per week monitoring frequency.
- TIN now has a permit limitation, based on the reasonable potential analysis, with a compliance schedule (discussed further in following subsection).
- Dissolved manganese and cyanide now have permit limitations, based on the reasonable potential analysis.
- A new procedure for chronic WET testing was implemented with a delayed effective date.
- Total recoverable arsenic, total recoverable beryllium, dissolved hexavalent chromium, total recoverable and dissolved iron, potentially dissolved manganese, total recoverable molybdenum, total mercury, boron, sulfide, sulfate, and nonylphenol were added to the permit with a report only requirement to monitor for future reasonable potential.
- The monitor requirements for iron (total recoverable and dissolved), sulfate, and chloride were replaced with effluent limits during the 2021 permit modification for outfalls 004A and 007A.

During the review of permit limits, minor errors were noted in the Division's calculations as follows:

- An incorrect formula was used by the Division when calculating the applicable acute and chronic cadmium standards. The error caused more stringent water quality standard values to be used in calculating Water Quality Based Effluent Limits (WQBELs) for the permit and resulted in more stringent than necessary acute and chronic effluent limits (current acute permit limit is 8.5 ug/L versus 9.1 ug/L and current chronic permit limit is 1.1 ug/L versus 2 ug/L using the correct water quality standard equations).
- The Division also used an incorrect dissolved manganese value for upstream ambient water quality. The error resulted in a slightly less stringent effluent limit in the permit.

Neither of these minor errors translate into significant issues with regard to treatment or meeting effluent limits but are noted so that Northglenn is aware to check these values during future permit renewals.

3.1.1 Compliance Schedules/Other Studies

The most recent permit also includes a compliance schedule for the following parameter (Part 1.B.6 of CO0036757):

- **TIN** - The Division has included a compliance schedule for TIN at outfalls 004A and 007A. Effluent limits for TIN are being applied for the first time and available data were used to establish an interim limit for TIN (refer to Table 3-2). The interim limit is below those established in Regulation 85 because it is based on the Water Supply Use standard for nitrate that currently applies downstream on the South Platte River.

  Monitoring while meeting the interim limits for the duration of the permit cycle (until June 30, 2024) will allow time to collect the necessary data to determine whether the limitation can be met and to meet the final effluent limit.

  The compliance schedule contains annual progress submittals (due to the Division by June 30 each year) that include reports on funding for design and construction, notification that funding has been obtained and notification of final plans, approval of final design, progress report on construction, and documentation of meeting final limitations. Northglenn currently meets the final effluent limit and should proceed by submitting data with a cover letter to the Division showing that compliance with the final effluent limit will not require construction or changes to operations.

  Note that the TIN limits in this compliance schedule are based on protecting downstream water supply standards for nitrate and nitrite. Potential future limits for nutrients based on the implementation of stream standards in Regulation 31 are further discussed in Section 3.3.

In addition, chronic WET testing was determined to be applicable for the WWTP based on the instream waste concentrations calculated in the Water Quality Assessment (WQA) performed for the permit (Appendix C). A delayed effective date is included in the permit for outfalls 004A and 007A (monitor and report through 2021 – refer to Table 3-2) to give the facility time to evaluate the discharge and ensure compliance with the limit (effective January 1, 2022).

Northglenn is also required to conduct the remaining threshold tests for exclusion from further analysis under Mixing Zone Regulations. Northglenn contracted the initial collection of site-specific data for the Application of the Mixing Zone Exclusion Tables in 2020 (Brown and Caldwell, 2020). Width and depth measurements were taken at outfalls 007A and 004A during a low flow event (flows must be in the lower 15th percentile for the receiving water during data collection). Field data are then compared to Division-established mixing zone exclusion tables (CDPHE, 2002) to determine if additional steps are required. The initial findings in the technical memorandum are that outfall 007A is excluded from further study while outfall 004A may require additional data collection and further study. The memorandum notes that the same Big Dry Creek low flows were applied to each outfall for the study (17.7 cubic feet per second (cfs)) and that there are two options available for outfall 004A for next steps. These include:

- Measure Big Dry Creek flows at outfall 004A and reassess the exclusion table measurements when flows are below 17.7 cfs at that site; or
- Calculate the regulatory and physical mixing zones.

Reevaluating low flows at outfall 004A and conducting the width and depth measurements for use with the Division-established exclusion tables is the less complex threshold test. Northglenn has until January 31, 2024 to work through the options presented above (as needed) and submit study results to the Division. If outfall 004A is not ultimately excluded from further mixing zone analysis based on additional data collection, it could potentially reduce the assimilative capacity at the point of discharge, which could result in more stringent limits in future permits.

3.1.2 TMDLs

A TMDL for *E. coli* was established for Big Dry Creek in 2016. The Waste Load Allocations (WLAs) for *E. coli* have been implemented in the permit. The limitation for *E. coli* is from the Regulations for Effluent Limitations (Regulation 62) and is the same as that contained in the previous permit.

Additionally, Big Dry Creek has been assigned a Load Allocation (non-point source) in a downstream TMDL for Barr Lake and Milton Reservoir and has been assigned a load reduction target for total phosphorus to meet TMDL targets for pH and dissolved oxygen for the waterbodies.

3.2 Receiving Stream Water Quality

Assimilative capacity is the amount of any given constituent that can mix into a receiving water without exceeding water quality standards. WQBELs are derived based on the assimilative capacity of a receiving water at regulatory low flows and are calculated using the following mass-balance equation where assimilative capacity is available:

$$M_2 = \frac{M_3 Q_3 - M_1 Q_1}{Q_2}$$

Where,

Q_1 = Regulatory low flow of receiving water upstream of discharge (1E3 or 30E3)

Q_2 = Design Capacity

Q_3 = Downstream flow ($Q_1 + Q_2$)

M_1 = In-stream ambient concentrations at the existing quality

M_2 = Maximum allowable effluent pollutant concentration

M_3 = Maximum allowable in-stream pollutant concentration (water quality standards)

The variables in the mass-balance equation are further detailed below as they apply to the Northglenn WWTP.

3.2.1 Watershed Identification

When not discharging to Bull Canal or Thompson Ditch for agricultural use, the Northglenn WWTP is permitted to discharge to Segment 1 of Big Dry Creek through outfalls 004A and 007A (refer to outfall descriptions in Section 3.1). The segmentation of the receiving water was updated in 2020 and is now defined in Regulation 38 as the "Mainstem of Big Dry Creek, including all tributaries and wetlands, from the outlet of Standley Lake to the confluence with the South Platte River, Walnut Creek, including tributaries and wetlands, from the outlet of Great Western Reservoir to the confluence with Big Dry Creek."

The previous MP (HDR, 2013) described the Big Dry Creek watershed as follows: "The watershed originates at the mouth of Coal Creek Canyon. The watershed drains easterly from the source across the Rocky Flats site to Standley Lake. Leaving Standley Lake, flows in Big Dry Creek are heavily regulated by releases for agricultural irrigation. Below the reservoir, Big Dry Creek flows in a northeasterly direction approximately 33 miles to its confluence with the South Platte River near Fort Lupton in Weld County. The total drainage area at the confluence is approximately 110 square miles with a 42-mile length. The first 8 miles below Standley Lake consist of a transitional foothill-plains stream with alternating zones of cobbles and sandy bottom. The lower 25 miles below the City of Westminster WWTP discharge is a plains type stream characterized by shifting channels, eroding banks, and migrating sand bars. The streambed consists primarily of sand and silt." The Big Dry Creek watershed and Segment 1 are shown on **Figure 3-2**.

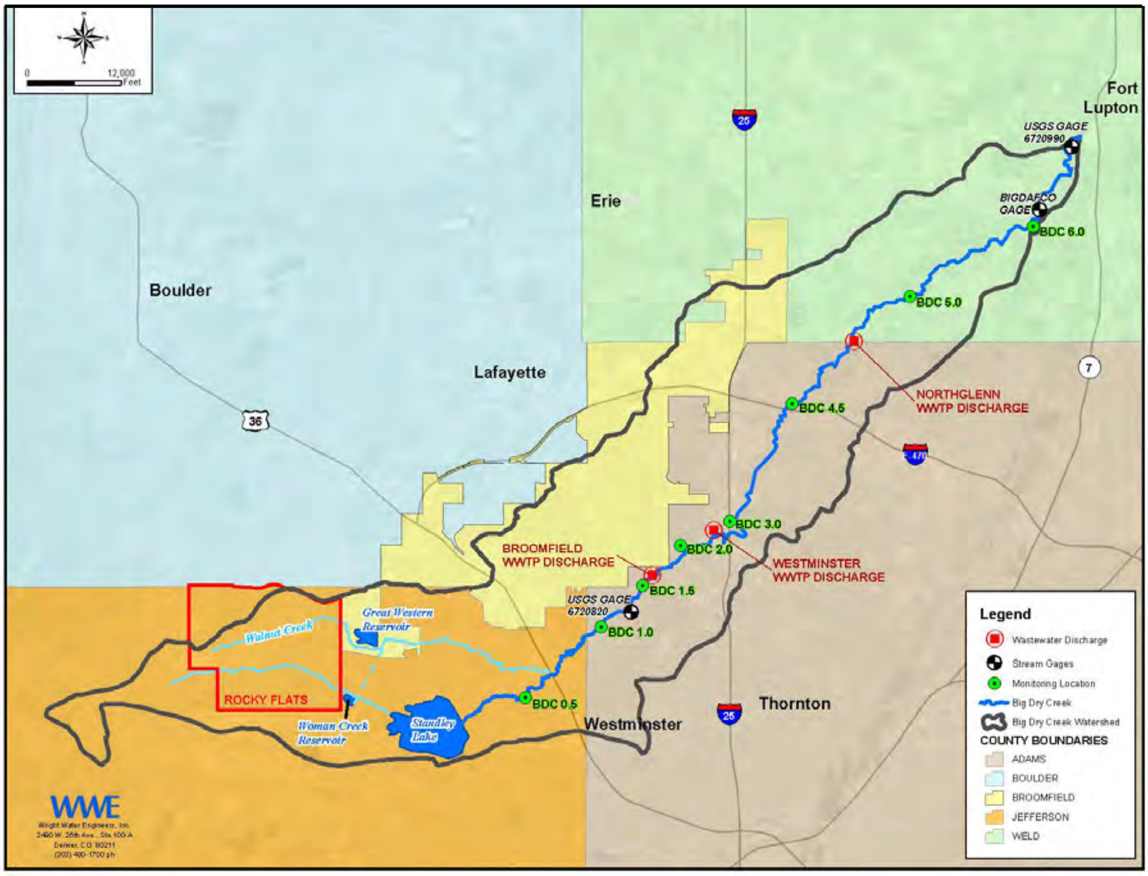


Figure 3-2: Big Dry Creek Watershed (Source: Big Dry Creek Watershed Association, 2020)

3.2.2 Regulatory Low Flow

The WQA performed for the 2019 permit renewal documented low flows used for permit calculations. Colorado Regulations specify that the acute low flow (used for developing effluent limits for acute standards), referred to as 1E3, represents the one-day low flow recurring in a three-year interval. A 7-day average low flow, 7E3, represents the seven-day average low flow recurring in a 3-year interval, and is used in developing limitations based on a Maximum Weekly Average Temperature standard (MWAT). The chronic low flow (used to develop limitations based on chronic standards), 30E3, represents the 30-day average low flow recurring in a three-year interval.

The WQA states that "To estimate the low flows for the Northglenn WWTP discharge point, daily flows from USGS Gage Station 06720990 were added to daily flows from the Yoxall Ditch diversion, located between the Northglenn WWTP and USGS Gage Station 06720990. Daily flow releases from Northglenn WWTP and the Brantner augmentation were then subtracted. The Brantner augmentation is located between the Northglenn WWTP and USGS Gage Station 06720990." Low flows for the Northglenn WWTP are shown in **Table 3-6**.

Table 3-6: Regulatory Low Flows for Big Dry Creek at Northglenn WWTP (CDPHE, 2019)

Low Flow (cfs)	Annual	Jan	Feb	Mar	Apr	May	June	Jul	Aug	Sept	Oct	Nov	Dec
1E3 Acute	0.6	14	11	8.8	8.0	8.4	6.2	5.8	0.6	8.5	10	7.7	13
7E3 Chronic	8.6	14	12	9.8	9.9	16	12	8.6	8.6	9.4	12	8.7	13
30E3 Chronic	12	14	14	13	13	16	15	12	12	12	13	13	13

Because the assimilative capacity of Big Dry Creek is impacted by multiple dischargers, WQBELs in Northglenn's permit were calculated through an approach that modeled the Broomfield, Westminster, and Northglenn WWTPs together. Regulatory low flows for each facility are shown in **Table 3-7**. The lowest annual acute and chronic low flows were used for permitting calculations by the Division in the 2019 permit renewal.

Table 3-7: Annual Regulatory Low Flows for Big Dry Creek at Broomfield, Westminster, and Northglenn WWTPs (CDPHE, 2019)

Low Flow (cfs)	Broomfield	Westminster	Northglenn
1E3 Acute	0.3	0.6	0.6
7E3 Chronic	0.6	1.4	8.6
30E3 Chronic	1	2.8	12

3.2.3 Receiving Stream Water Quality Standards

Water quality standards vary depending on the receiving water (Bull Reservoir, Bull Canal, Thompson Ditch, or Big Dry Creek). The permit notes that "Bull Canal and Thompson Ditch are solely for agricultural use and do not enter classified waters of the state. Bull Canal has numerous laterals that terminate at various FRICO stockholder's fields. Thompson Ditch terminates at Northstar Reservoir, which is a privately-owned reservoir that consists of an approximately 8-foot berm, has no outlet, and is drained by agricultural users on an annual basis. The Division considers this reservoir and the Bull Reservoir water withdrawn for beneficial use as not being classified waters of the state at this time" (CDPHE, 2019). Water quality standards for Segment 1 of Big Dry Creek are listed in Regulation 38.

In 2020, Regulation 38 was updated with significant changes to stream standards for Segment 1 of Big Dry Creek. The changes include the adoption of more stringent designated uses including a change from Aquatic Life 2 to Aquatic Life 1, change from Potential Recreation to Existing Recreation, and the addition of a Water Supply use. Agricultural use standards continue to apply. **Figure 3-3** shows the updated classifications and standards in redline to show the changes that have been adopted.

1. Mainstem of Big Dry Creek, including all tributaries and wetlands, from the ~~source~~outlet of Standley Lake to the confluence with the South Platte River~~, except for specific listing in Segments 4a, 4b, 5~~. Walnut Creek, including tributaries and ~~5~~wetlands, from the outlet of Great Western Reservoir to the confluence with Big Dry Creek.

COSPBD01	Classifications
Designation	Agriculture
UP	Aq Life Warm ~~2~~1
	Water Supply
	Recreation ~~P~~E
Qualifiers:	
Other:	

*chlorophyll a (mg/m^2)(chronic) = applies only above the facilities listed at 38.5(4).
*Phosphorus(chronic) = applies only above the facilities listed at 38.5(4).
*Selenium(acute) = 19.1 ug/L from 11/1 - 3/31 TVS from 4/1 - 10/31. Refer to Section 38.6(4)(d).
*Selenium(chronic) = 15 ug/L from 11/1 - 3/31 7.4 ug/L from 4/1 - 10/31. Refer to Section 38.6(4)(d).
*Uranium(acute) = See 38.5(3) for details.
*Uranium(chronic) = See 38.5(3) for details.

Physical and Biological	DM	MWAT
Temperature °C	WS-I	WS-I
	acute	chronic
D.O. (mg/L)	---	5.0
pH	6.5 - 9.0	---
chlorophyll a (mg/m^2)	---	150*
E. Coli (per 100 mL)	---	~~205~~126

Inorganic (mg/L)	acute	chronic
Ammonia	TVS	TVS
Boron	---	0.75
Chloride	---	~~—~~250
Chlorine	0.019	0.011
Cyanide	0.005	---
Nitrate	~~100~~10	---
Nitrite	---	4.5
Phosphorus	---	0.17*
Sulfate	---	~~—~~WS
Sulfide	---	0.002

Metals (ug/L)	acute	chronic
~~Aluminum~~	~~—~~	~~—~~
Arsenic	340	---
Arsenic(T)	---	0.02-10
~~Beryllium~~	~~—~~	
Beryllium(T)	---	100
Cadmium	TVS	TVS
Cadmium(T)	5.0	---
Chromium III	~~TVS~~---	TVS
Chromium III(T)	~~—~~50	~~100~~---
Chromium VI	TVS	TVS
Copper	TVS	TVS
Iron	---	WS
Iron(T)	---	1000
Lead	TVS	TVS
Lead(T)	50	---
Manganese	TVS	TVS/WS
Mercury(T)	---	0.01~~(t)~~
Molybdenum(T)	---	150
Nickel	TVS	TVS
Nickel(T)	---	100
Selenium	---	varies*
Selenium	varies*	---
Silver	TVS	TVS
Uranium	~~—~~varies*	~~—~~varies*
Zinc	TVS	TVS

Figure 3-3: 2020 Regulation 38 Revisions for Segment 1 of Big Dry Creek

Table 3-8 contains the currently applicable water quality standards based on the changes shown in Figure 3-3. Criteria that have changed since the 2019 permit are highlighted in the table. Criteria that are based on stream hardness have been calculated using a mean hardness value of 361 mg/L which is consistent with the permit. Calculated Table Value Standards are shown in bold blue type.

Table 3-8: Numeric Water Quality Criteria for Segment 1 of Big Dry Creek (CDPHE, 2020)

Parameter	Criteria	Units	Parameter	Criteria	Units
Dissolved Oxygen (DO)	5	mg/L, min	Copper, D, acute	**45**	ug/L
pH	6.5 - 9.0	su	Copper, D, chronic	**27**	ug/L
E. coli	126	CFU/100 mL	Iron, D, WS	300*	ug/L
Temperature March-Nov	24.2	°C MWAT	Iron, TR, chronic	1000	ug/L
	29	°C DM	Lead, D, acute	**253**	ug/L
Temperature Dec-Feb	12.1	°C MWAT	Lead, D, chronic	**9.8**	ug/L
	24.6	°C DM	Lead, T, acute	50	ug/L
Chlorophyll a	150	mg/m^2	Manganese, D, acute	**4579**	ug/L
Phosphorus	0.17	mg/L	Manganese, D, WS	50*	ug/L
Total Ammonia	**TVS**		Manganese, D, chronic	**2530**	ug/L
Chloride, chronic	250	mg/L	Molybdenum, TR, chronic	150	ug/L
Chlorine, acute	0.019	mg/L	Mercury, T, chronic	0.01	ug/L

Parameter	Criteria	Units	Parameter	Criteria	Units
Chlorine, chronic	0.011	mg/L	Nickel, D, acute	**1387**	ug/L
Free Cyanide, acute	0.005	mg/L	Nickel, D, chronic	**154**	ug/L
Sulfide, chronic	0.002	mg/L	Nickel, T, chronic	150	ug/L
Boron, chronic	0.75	mg/L	Selenium, D, acute		ug/L
Nitrite, chronic	4.5	mg/L as N	April to October	18.4	ug/L
Nitrate, WS	10	mg/L as N	November to March	19.1	ug/L
Arsenic, D, acute	340	ug/L	Selenium, D, chronic		ug/L
Arsenic, TR, Aq and WS	0.02-10	ug/L	April to October	7.4	ug/L
Beryllium, TR, chronic	100	ug/L	November to March	15	ug/L
Cadmium, D, acute	**9.1**	ug/L	Silver, D, acute	**18**	ug/L
Cadmium, D, chronic	**1.9**	ug/L	Silver, D, chronic	**2.9**	ug/L
Cadmium, T, acute	5	ug/L	Sulfate, WS	250	mg/L
~~Chromium +3, TR, chronic~~	~~100~~	ug/L	Uranium, D, acute WS	16.8-30	ug/L
Chromium +3, TR, acute	50	ug/L	Uranium, D, chronic WS	16.8-30	ug/L
~~Chromium +3, D, acute~~	~~**1630**~~	ug/L	Zinc, D, acute	**514**	ug/L
Chromium +3, D, chronic	**212**	ug/L	Zinc, D, chronic	**389**	ug/L
Chromium +6, acute	16	ug/L	Nonylphenol, acute	28	ug/L
Chromium +6, chronic	11	ug/L	Nonylphenol, chronic	6.6	ug/L

WS = Water Supply Aq and WS = Aquatic Life and Water Supply Standard
D = Dissolved TR = Total Recoverable T = Total TVS = Table Value Standard
*standards can be applied as either of the less restrictive of the following two options: 1. existing quality as of January 1, 2000; or 2. the table value criterion

The range of values for both arsenic and uranium are applied to streams that have a Water Supply designated use. The first number in the range is a health-based standard. The second number in the range is a maximum contaminant level, established under the federal Safe Drinking Water Act (SDWA) that has been determined to be an acceptable level in public water supplies. For discharge permit effluent limitations, the Division uses the first number in the range as the ambient water quality target, provided that no effluent limitation shall require an "end-of-pipe" discharge level more restrictive than the second number in the range.

The Basic Standards and Methodologies notes that the Water Supply use standards may be applied as the less restrictive of either the existing quality as of January 1, 2000 or at the Table Value Standard (Iron = 300 µg/L (dissolved); Manganese = 50 µg/L (dissolved); Sulfate = 250 mg/L) (Regulation 31.11(6), CDPHE). The Division's standard practice is to utilize only water quality data collected prior to December 1999 unless fewer than 10 data points are available. The Big Dry Creek Watershed Association (BDCWA) produces annual reports to document water quality and regulatory changes. The BDCWA Annual Report for 2019 provides the following information for each of these parameters using samples available from all locations within Segment 1 of Big Dry Creek (Wright Water Engineers, 2020):

- **Manganese** - *Based on dissolved manganese in the Division's existing quality library, which does not currently include BDCWA's data set, existing quality for 1995-1999 would be 85 ug/L. Using the entire period of record in the library, the existing quality value would be 78 ug/L. BDCWA's database only includes dissolved manganese for 30 samples in 1999, with an 85th percentile value of 57 ug/L.*

- **Iron** - *BDCWA does not currently monitor for dissolved iron but intends to add it to the routine sampling program. Metro's dissolved iron monitoring in the lower watershed indicates instream concentrations of dissolved iron of 62.4 ug/L in 2019, suggesting that the stream is likely to attain the dissolved iron standard. Additionally, review of the Division's existing quality data library for Big Dry Creek shows an existing condition for dissolved iron of 90 ug/L, further indicating that dissolved iron is likely to attain the new stream standard.*

- **Sulfate** - *Based on sulfate data in the BDCWA database from 1995-1999, the existing quality standard for sulfate would be 380 mg/L. Based on sulfate in the Division's existing quality library, which does not currently include BDCWA's data set, existing quality for 1995-1999 would be 308 mg/L. Using the entire period of record in the library, existing quality would be 383 mg/L. Based on review of sulfate data, Big Dry Creek would not be expected to attain the sulfate standard.*

Note that these values are lower than those used for calculating WQBELS in the current permit. WQBELs for Water Supply Use parameters were calculated using low flows and water quality of the South Platte River near the confluence with Big Dry Creek based on where the Water Supply Use existed at the time of permit renewal. Future permits will use Big Dry Creek data which is now designated for Water Supply Use resulting in lower WQBELs for these parameters.

3.2.4 Receiving Stream Water Quality

The Division relied on data collected at site BDC 1.5 (located approximately 1 mile upstream from the Broomfield facility – refer to Figure 3-2) to assess upstream ambient water quality. Data presented in the WQA were available for a period of record from February 2013 through December 2017 and are summarized in **Table 3-9**.

Table 3-9: Ambient Water Quality for Big Dry Creek upstream of Broomfield WWTP (CDPHE, 2020)

Parameter	Number of Samples	15th Percentile	50th Percentile	85th Percentile	Mean	Maximum	Chronic Stream Standard	Notes
Temperature March-Nov (Summer) °C MWAT	33	12.5	18.5	22.8	17.8	26.3	24.2	3
Temperature March-Nov (Summer) °C DM	33	12.3	21.8	27.5	21.1	29.8	29	3
Temperature Dec-Feb (Winter) °C MWAT	14	5	7.6	9.6	7.7	12.6	12.1	3
Temperature Dec-Feb (Winter) °C DM	14	7.1	9.3	10.9	9.1	12.9	24.6	
DO, mg/L	47	7.4	9.1	12	9.3	13	>5	
pH, su	49	7.3	7.6	7.9	7.6	8.2	6.5 - 8	
E. coli, CFU/100 mL	60	115	462	1,013	374	2,420	205	1, 3

Parameter	Number of Samples	15th Percentile	50th Percentile	85th Percentile	Mean	Maximum	Chronic Stream Standard	Notes
Nitrite, mg/L as N	49	0	0.014	0.02	0.013	0.1	4.5	2
Nitrate+Nitrite, mg/L as N	49	0.47	1	1.9	1.1	2.9	NA	
Total Inorganic Nitrogen, mg/L as N	40	0.47	0.92	1.8	1.1	2.9	NA	
Total Phosphorus, mg/L as P	49	0.031	0.053	0.12	0.069	0.21	0.17	
TSS, mg/L	49	7.5	21	48	30	100	30	1
Arsenic, TR, ug/L	17	0	0	0	0.059	1	100	2
Cadmiuim, D, ug/L	17	0	0	0	0	0	1.1	2
Chromium, D, ug/L	17	0	0	0	0	0	NA	2
Copper, D, ug/L	17	2.3	5.3	12	6.9	17	27	
Manganese, D, ug/L	17	22	60	176	102	380	2,530	
Nickel, D, ug/L	17	0	2	2	1.4	2	154	2
Selenium, D, ug/L (Nov - Mar)	10	4.4	9.4	13	8.8	13	15	
Selenium, D, ug/L (Apr - Oct)	9	1	1.7	4.2	2.5	5	7.4	
Silver, D, ug/L	17	0	0	0	0	0	2.9	2
Zinc, D, ug/L	17	0	2	3.6	2	7	389	2

D = Dissolved TR = Total Recoverable
1. The calculated mean is the geometric mean. The value of one was used where there was no detectable amount.
2. When sample results were below detection limits, the value of zero was used.
3. The ambient water quality exceeds the water quality standards for these parameters.

Data in Table 3-9 were pulled from the WQA and reflect data for parameters for which the previous permit required monitoring and/or that the Division has determined have reasonable potential to cause or contribute to an exceedance of water quality standards. The WQA did not include a data summary for parameters that will now be relevant due to the 2020 addition of the Water Supply use to the receiving water (chloride, sulfate, dissolved iron, and dissolved manganese).

3.2.4.1 303(d) Listings and Monitoring and Evaluation (M&E) Listings

Impairment of designated uses and instances where additional data collection may be needed to assess impairment are documented in Colorado's Section 303(d) List of Impaired Waters and Monitoring and Evaluation List (Regulation 93, CDPHE). The WQA for permit CO0036757 included the following information regarding 2018 303(d) and M&E Listings that were considered during permit renewal (**Table 3-10**):

Table 3-10: Additional Regulatory Considerations for Permit CO0036757

Threatened and Endangered Species	303(d) (Reg 93)	M&E (Reg 93)	Existing TMDL	Temporary Modifications	Control Regulations
No	Total Recoverable Iron*	None	*E. coli* (9/28/2016)	None	Regulation 85

*Total Recoverable Iron impairment is approximately 8 miles downstream of the Northglenn WWTP

The BDCWA 2019 Annual Report (Wright Water Engineers, 2020) also noted that Segment 1 (the main stem) of Big Dry Creek is listed on the 2020 303(d) List for Colorado for nonattainment of stream standards for *E. coli* for the entire segment and for total recoverable iron for the portion of the stream below Weld County Road 8. The iron impairment in the lower watershed is based on data collected by Metro Wastewater Reclamation District (Metro). A brief synopsis of these two known regulatory issues as of 2019 includes:

- ***E. coli***: Big Dry Creek did not meet the *E. coli* standard during 2019. A TMDL for *E. coli* in Big Dry Creek segment COSPBD01 was approved by the U.S. Environmental Protection Agency (EPA) in September 2016. This TMDL was based on a Potential Recreational Contact standard of 205 cfu/100 mL. As a result of the 2020 change to Big Dry Creek Segment 1, this standard is now 126 cfu/100 mL. Special studies to identify sources of *E. coli* in the watershed are currently underway for the stream reach between Standley Lake and I-25.

- **Iron**: Although BDCWA's long-term water quality dataset shows attainment of the total recoverable iron standard, the portion of Big Dry Creek below Weld County 8 was identified as impaired on the 2016 303(d) List based on data submitted by Metro.

The 2020 changes to the applicable stream standards for Segment 1 of Big Dry Creek result in additional anticipated impairment listings, driven primarily by the addition of Water Supply standards based on identification of alluvial wells used for drinking water in the lower watershed. These impairments may include sulfate, chloride, dissolved manganese and nitrate.

Other future impairment concerns include total nitrogen (TN) and TP, which are constituents included in the Division's 10-Year Water Quality Road Map. Currently, interim values for TN (2,010 ug/L) and TP (170 ug/L) are exceeded (based on an annual median value with allowable exceedance of once in five years) for the portion of the stream segment beginning below the WWTP discharges. A final decision by the Water Quality Control Commission (Commission) on application of these instream standards is expected in 2027.

3.2.4.2 TMDLs and/or WLAs or Reductions

The Division's Restoration and Protection Unit completed a TMDL for *E. coli*; *COSPBD01 Mainstem of Big Dry Creek, including all tributaries and wetlands, from the source to the confluence with the South Platte River, Segment 1, Escherichia coli (E. coli)* which was approved by USEPA on September 28, 2016. The TMDL states that the *E. coli* WLA for the Northglenn WWTP is 205 cfu/100ml which has been implemented in the permit. The Big Dry Creek Water Quality Summary for 2019 (Wright Water Engineers, 2020) notes that the BDCWA is working on a watershed plan update to identify next steps related to the *E. coli* TMDL in terms of source identification and potentially feasible load reductions. It is recommended that Northglenn continue participation in the BDCWA and monitor updates to the watershed plan.

As documented in Section 3.2.3, Segment 1 of Big Dry Creek was updated from Potential Recreation to Existing Recreation which results in a more stringent *E. coli* standard (126 cfu/100ml versus 205 cfu/100ml). Northglenn should be aware that this may result in an updated TMDL for the receiving water and should anticipate future effluent limits of 126 cfu/100ml.

3.3 Future Level of Treatment Required

The current permit is set to expire on December 31, 2024. Recent regulatory updates to the classifications of Segment 1 of Big Dry Creek and the resulting changes to a number of applicable water quality standards will likely affect monitoring requirements and/or effluent limits in the next permit renewal. Any new parameters added to the next permit would include a compliance schedule as needed based on data availability (upstream and effluent). The interim nutrient standards found in Regulation 31 (**Table 3-11**) also provide values to be used for planning purposes but will not be considered for statewide adoption until 2027 and may be revised prior to that date (refer to roadmap timeline below).

Table 3-11: Interim Numeric Nutrient Criteria (Regulation 31.17)

Parameter	Rivers and Streams - Warm
Total Phosphorus	170 ug/L*
Total Nitrogen	2,010 ug/L*
Chlorophyll-a	150 mg/m^{2}**

*Annual median, allowable exceedance frequency 1-in-5 years
** Summer (July 1 – September 30) maximum attached algae, not to exceed.

The Division has created the Nutrient Incentives Program to encourage facilities to make nutrient reductions in exchange for an extended compliance schedule. The program also creates certainty regarding the year the facility will need to meet WQBELs for nutrients. Additional information on the incentives program is included in Section 3.3.1.1.

The Division's 10-year water quality roadmap also plans for:

- The adoption of statewide chlorophyll *a* standards in 2022.
- The release of draft arsenic and NH_3 criteria in 2023.
- The release of draft selenium criteria in 2024.
- Statewide adoption of arsenic standards in 2024.
- The release of draft TN and TP criteria for streams in 2025.
- Statewide adoption of NH_3, selenium, TN and TP standards in 2027.

The Division is also continually evaluating other emerging contaminants of concern. A new policy document (Policy 20-1) was submitted to the Commission in July 2020 that established a "Policy for Interpreting the Narrative Water Quality Standards for Per- and Polyfluoroalkyl Substances (PFAS)". PFAS are an emerging public health challenge that originate from a family of chemicals found in toxic firefighting foam and other sources that have been used for decades in many consumer products and industrial processes to repel oil and water, resist heat, and reduce friction. Colorado's narrative standard states that "state surface waters shall be free from substances attributable to human-caused point source or nonpoint source discharge in amounts, concentrations or combinations which are harmful to the beneficial uses or toxic to humans, animals, plants, or aquatic life (Section 31.11(1)(a)(iv))."

Based on best professional knowledge, implementation of regulations pertaining to PFAS will start with the establishment of maximum contaminant levels (MCLs) through the SDWA. Drinking water systems and dischargers with the potential to affect drinking water systems may be required to monitor for the presence of PFAS to determine where they are present and may be an issue. The Division would likely form a working group with a goal of proposing draft surface and groundwater criteria for review. The eventual adoption of criteria for PFAS and ultimate implementation into discharge permits is not anticipated during the planning horizon of this Plan.

3.3.1 Potential Effluent Limits

Potential future effluent limits provided below are set at the water quality standards due to a lack of assimilative capacity, meaning the instream ambient water quality currently and/or will likely, exceed the water quality standard in the future. The BDCWA Annual Report noted that "the 2020 changes to the applicable stream standards for Big Dry Creek result in additional anticipated impairment listings, driven primarily by the addition of Water Supply standards based on identification of alluvial wells used for drinking water in the lower watershed. These impairments include sulfate, chloride, dissolved manganese and nitrate". And that "currently, interim values for TN and TP are exceeded for the portion of the stream segment beginning below the WWTP discharges". Northglenn should anticipate future limits as follows:

- ***E. coli*** – A limit of 126 cfu/100mL will likely be included in the next permit cycle based on the updated recreational use classifications of the receiving water.
- **Total Phosphorus** –An eventual TP limit of 0.17 mg/L will likely be incorporated into a permit renewal after 2027 with a compliance schedule that can be extended through participation in the voluntary incentives program (discussed further in Section 3.3.1.1).
- **Total Nitrogen** – An eventual TN/TIN limit of 2.01 mg/L will likely be incorporated into a permit renewal after 2027 with a compliance schedule that can be extended through participation in the voluntary incentives program (discussed further in Section 3.3.1.1).
- **Total Recoverable Arsenic** – A range of values (0.02 - 10 ug/L) is now applicable to Segment 1 of Big Dry Creek. The range is applied to streams that have a Water Supply designated use. The first number in the range is a health-based standard. The second number in the range is the MCL, established under the federal SDWA that has been determined to be an acceptable level in public water supplies. For discharge permit effluent limitations, the Division uses the first number in the range as the ambient water quality target, provided that no effluent limitation shall require an "end-of-pipe" discharge level more restrictive than the second number in the range. Northglenn should anticipate a total recoverable arsenic limit of 10 ug/L in the next permit cycle with a compliance schedule as needed/applicable.

- **Dissolved Uranium** – A range of values (16.8 - 30 ug/L) is now applicable to Segment 1 of Big Dry Creek. The range is applied as described in the previous bullet. Northglenn should anticipate a dissolved uranium limit of 30 ug/L at some point in the future. The Division may require monitoring in the next cycle to determine reasonable potential.

- **Chloride –** A limit of 250 mg/L will likely be included in the next permit cycle with a compliance schedule as needed/applicable.

- **Dissolved Manganese** – As discussed in Section 3.2.3, Water Supply use standards can be applied at the existing water quality. BDCWA determined that existing water quality is above the table value standard of 50 ug/L. A limit somewhere between 50 and 85 ug/L is likely in the next permit with a compliance schedule as needed/applicable.

- **Dissolved Iron** - BDCWA data showed that existing quality on Big Dry Creek is below the table value standard. Northglenn should anticipate a limit near 300 ug/L in the next permit with a compliance schedule as needed/applicable.

- **Sulfate** - BDCWA determined that existing water quality is above the table value standard of 250 ug/L. A limit somewhere between 250 and 385 ug/L is likely in the next permit with a compliance schedule as needed/applicable.

- **Temperature** - Currently, the municipal WWTP dischargers to Big Dry Creek are required to "report only" under terms of the 2019 permits. BDCWA is in the process of collecting additional instream temperature data at 15-minute intervals along Big Dry Creek. Preliminary data from Northglenn (collected at 30-minute intervals from outfall 007A, 004A, and upstream instream) have been reviewed and summary information is presented in **Figures 3-4** (DM) and **3-5** (MWAT). Note that the calculated maximum monthly effluent MWAT at outfall 007 has exceeded the instream standards during winter months. The preliminary data show that there is reasonable potential and that limits will likely be included in the next permit. There appears to be assimilative thermal capacity in Big Dry Creek during the winter months (Figure 3-5). The additional instream data collected by BDCWA and the Northglenn WWTP should be reviewed as available as effluent temperature limits will be calculated based on all available data at the time of permit renewal. Continued participation in the BDCWA will help Northglenn anticipate potential future temperature limits. New limits will include a compliance schedule as needed/applicable.

Northglenn should anticipate new effluent limits for temperature, *E. coli*, and the Water Supply use parameters in the next permit cycle. Timing for implementation of these limits may include compliance schedules, as needed, through the term of the next permit (through December 31, 2029).

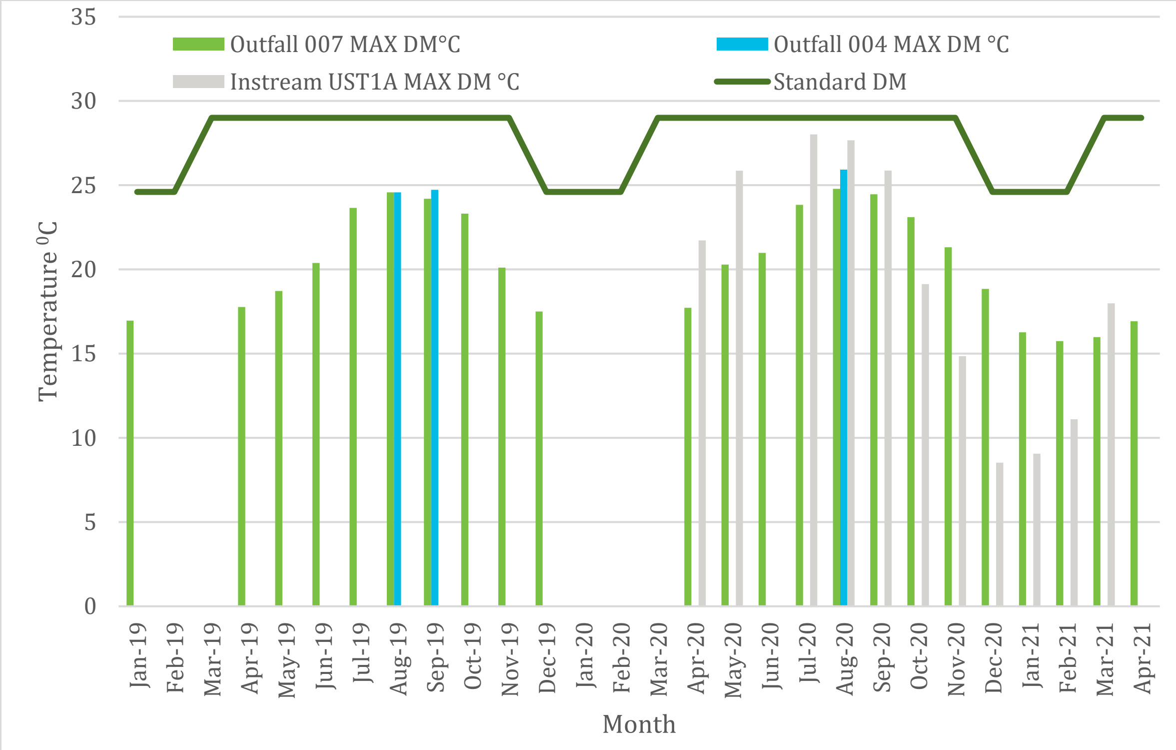


Figure 3-4: Monthly Maximum DM

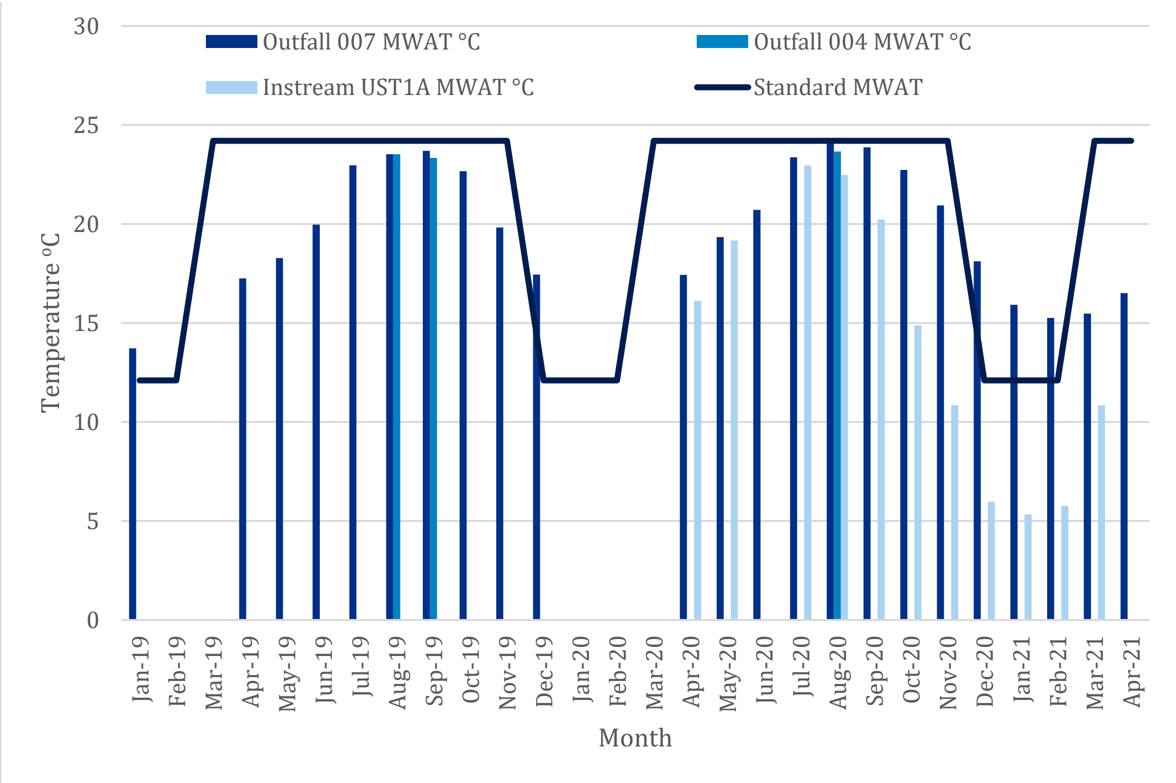


Figure 3-5: Monthly Maximum MWAT

3.3.1.1 Future Nutrient Limits

The Commission will consider adoption of in-stream numeric criteria for TP and TN in 2027. Policy 17-1 describes the Voluntary Incentive Program for Early Nutrient Reductions in accordance with Regulation 85. This voluntary program allows dischargers to collect and report samples for TIN and TP. Samples are reported annually to the Division. In exchange for making voluntary reductions in effluent nutrients, facilities will receive extended compliance schedules for nutrients as well as certainty about the year in which the facility will need to meet WQBELs for nutrients.

The 2030 permit may include nutrient limits based on the actions of the Commission. Northglenn can guarantee a compliance schedule of up to fifteen years for nutrients through continued participation in the voluntary incentives program. This is based on a 5-year compliance schedule for new limits regardless of participation in the incentive program and a maximum of up to 10 additional years earned through the incentive program which could mean compliance may not be required until as far out as 2045 (**Figure 3-6**) depending on the number of nutrient reductions earned each year and the number of years the WWTP participates in the program.

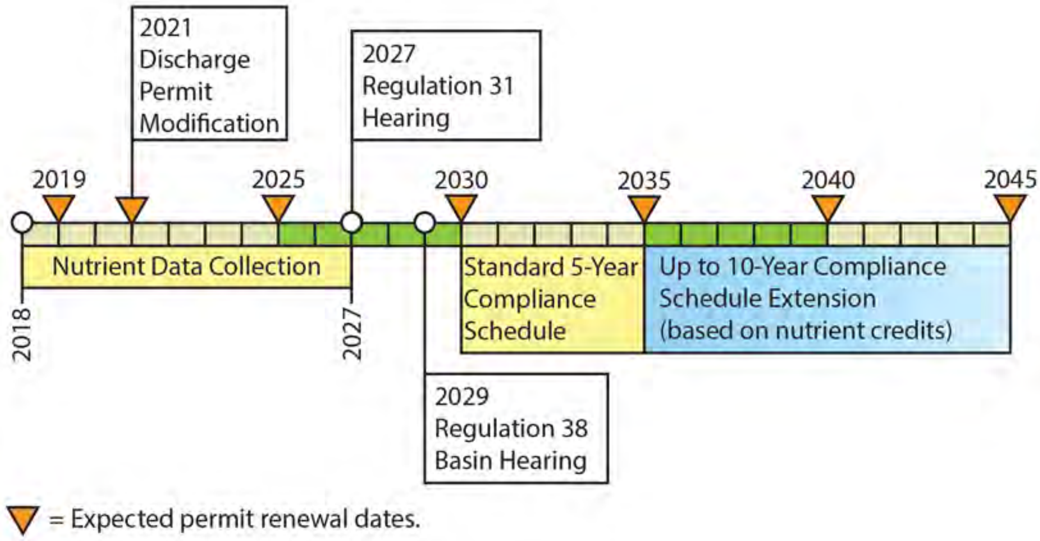


Figure 3-6: Potential Nutrient Effluent Limit Implementation Timeline

There are four caveats for Policy 17-1. First, only existing facilities that were discharging prior to December 31, 2017 are eligible. Second, to participate in the program, facilities were required to submit a nutrient reduction plan to the Division prior to December 31, 2019. Third, the maximum extension time for compliance schedules is 10 years. Finally, facilities must submit 12 months of data to receive credits for a given year. Entities may collect and submit data beginning in 2018 and ending in 2027.

For the Barr Milton watershed, dischargers that maintain an annual median effluent TP concentration between 0.7 mg/L and 1.0 mg/L will earn up to 12 incentive months for each year that they meet the standard. Dischargers that maintain an annual median TP concentration between 0.1 and 0.7 mg/L will be awarded up to 24 incentive months for each year that they meet this standard. Northglenn typically discharges around 0.4 mg/L TP and is eligible for the greater credit. Similarly, dischargers may earn incentive months for reducing TIN concentrations. Dischargers that maintain an annual median TIN concentration between 7 and 15 mg/L will be awarded up to 12 incentive months each year. The actual number of incentive months awarded is based on the annual median effluent TIN and TP concentrations according to the following equations:

$$\frac{15 - TIN}{15 - 7} \times 12 = \text{Months Credit Earned for Year}$$

$$\frac{0.7 - TP}{0.7 - 0.1} \times 12 + 12 = \text{Months Credit Earned for Year}$$

Northglenn can potentially earn the maximum 10-year extension (in addition to the compliance schedule that would be included in the permit regardless of program participation) with a combination of lower TIN and TP concentrations. For the calculations in **Table 3-12**, effluent annual median TIN and TP concentrations of 10 mg/L and 0.5 mg/L were assumed. Northglenn would only need 5 years of TIN data and 6 years of TP data at these concentrations to earn the maximum extension.

Table 3-12: Example Nutrient Incentive Program Calculations based on Northglenn Nutrient Effluent Concentrations

	Annual Median Concentrations		Incentive Credits Earned		
	TIN	TP	TIN	TP	TIN+TP
Year	mg/L	mg/L	mg/L	mg/L	month
2018	10	0.5	7.5	16	
2019	10	0.5	7.5	16	
2020	10	0.5	7.5	16	
2021	10	0.5	7.5	16	
2022	10	0.5	7.5	16	
2023		0.5		16	
2024					
2025					
2026					
2027					
		Total Months	**37.5**	**96**	**133.5**
		Eligible Months	**37**	**96**	**133**
		Total Years	**3.08**	**8**	**10**

Note: Calculator from CDPHE website at https://cdphe.colorado.gov/nutrients-incentive-program

Northglenn submitted effluent nutrient data to the Division for years 2018, 2019, and 2020. At present, the Division website is not showing annual median concentrations for Northglenn on their website. The Division website also has the incorrect discharge permit number listed. Northglenn has made the Division aware of both issues.

Because some of Northglenn's effluent is stored in Bull Reservoir and then intermittently discharged, Northglenn was unable to submit twelve consecutive months of effluent data for 2018, 2019, or 2020. The Division is aware of the uniqueness of the facility design and is in internal discussions to determine if they might be able to issue some credits for the months when Northglenn was discharging. If the Division decides against issuing credits, Northglenn will not be able to push out a future compliance schedule based on the previous years of submissions but still has opportunity to earn credits before 2027.

Northglenn could consider starting to discharge effluent to Big Dry Creek for at least one day a month starting in January 2022. This would allow collection of final effluent samples and accrual of nutrient compliance schedule credits. There is enough time remaining between January 2022 and December 2027 for Northglenn to maximize the nutrient credit earned.

Discharging would require Northglenn to collect samples for all their permitted parameters including WET testing and trace metals. The cost of these laboratory analyses must be balanced against the benefit of an extended compliance schedule. Sample collection for WET testing may require multiple days of discharge and could impact Northglenn's water rights.

3.3.2 Water Quality Target Limits Discussion

Planning recommendations based on the regulatory information presented above include but are not limited to:

- Compliance with Water Supply Use parameter effluent limits by 2030 based on permit incorporation in 2025 and compliance schedules as needed and appropriate.
- Compliance with *E. coli* and temperature limits by 2030 based on permit incorporation in 2025 and compliance schedules as needed and appropriate.
- Continued participation in the voluntary incentives program. Information on the incentives program can be found in Commission Policy 17-1 (https://drive.google.com/file/d/1faoaeB_z4TcFu5eGux8Qrf6Rj9WtfE5M/view). Northglenn should continue discussions with the Division to obtain credit for previous years and continue to submit annual reports by March 31 each year to earn additional credits towards extended compliance schedules. In order to accrue the maximum compliance schedule extension, Northglenn may be required to discharge at least one day per month to obtain adequate samples to meet the Policy 17-1 data requirements. The calculation tool is available through the Division and it is suggested that Northglenn continue to tally months earned annually to more accurately forecast ultimate compliance dates for nutrient limits.
- Continued participation in the BDCWA, review of the BDCWA annual reports, and review of the future update to the watershed plan.
- Participation in the Division's 10-year water quality roadmap working group. Information on how to sign up for email alerts, the schedule of upcoming meetings, and documentation of previous meetings can be found at https://cdphe.colorado.gov/water-quality-10-year-roadmap. Participation in the working group will keep Northglenn up to date on draft criteria, standard adoption, and anticipated timing for implementation into permits.
- Monitor developments in PFAS regulations. Although complying with eventual effluent limits for PFAS is not anticipated during the planning horizon of this study, the Division has signaled an intent to work toward future regulations.

Section 4

Existing Conditions

Existing wastewater characteristics for the Northglenn WWTP have been developed based on an eleven-year record from 2010 through 2020. Recommended improvements to maintain compliance at the WWTP are based on a review of existing treatment system performance and a conditions assessment which included onsite inspection of equipment, documentation review and discussions with operations and maintenance (O&M) personnel.

4.1 Current Planning Service Area

Northglenn is divided into two service areas: the NSA and the SSA.

4.1.1 South Service Area

The SSA serves a population of approximately 39,000 people and covers roughly seven square miles (**Figure 4-1**). The boundaries of the SSA are irregular, extending from 95th Avenue north to 124th Avenue and from Madison Street west to Zuni Street. Interstate Highway I-25 runs through the center of the SSA and runs north-south. The SSA is surrounded by other municipalities and cannot easily expand its boundaries.

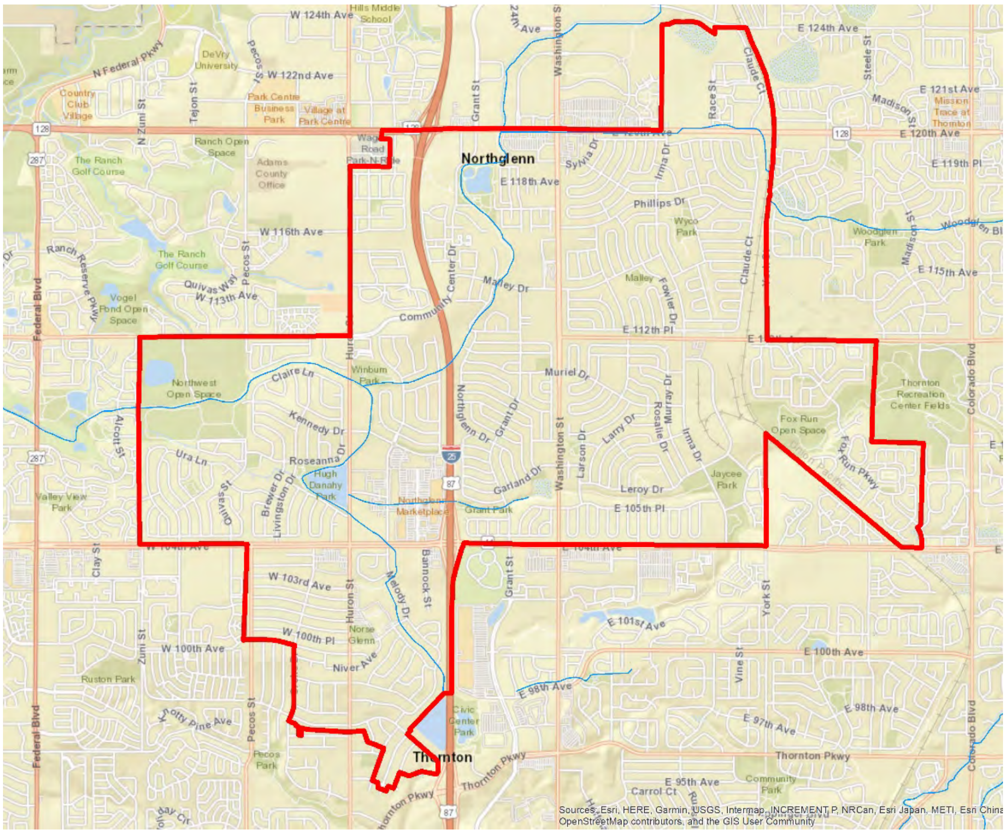


Figure 4-1: City of Northglenn South Service Area

4.1.2 North Service Area

The NSA consists primarily of undeveloped farmland located north of 168th Avenue in Weld County (**Figure 4-2**). The NSA includes some single-family homes, farms, and businesses.

Areas highlighted in light blue consist of Bull Reservoir, the Northglenn WWTP, recreational vehicle storage, a power booster station, concrete supply company, and oil and gas extraction sites. Areas highlighted in pink consist of single-family homes and farm buildings. The 100-year flood plain is indicated in blue and yellow. The remainder of the NSA is farmland. Apart from Section 36, the NSA is unincorporated (**Figure 4-3**).

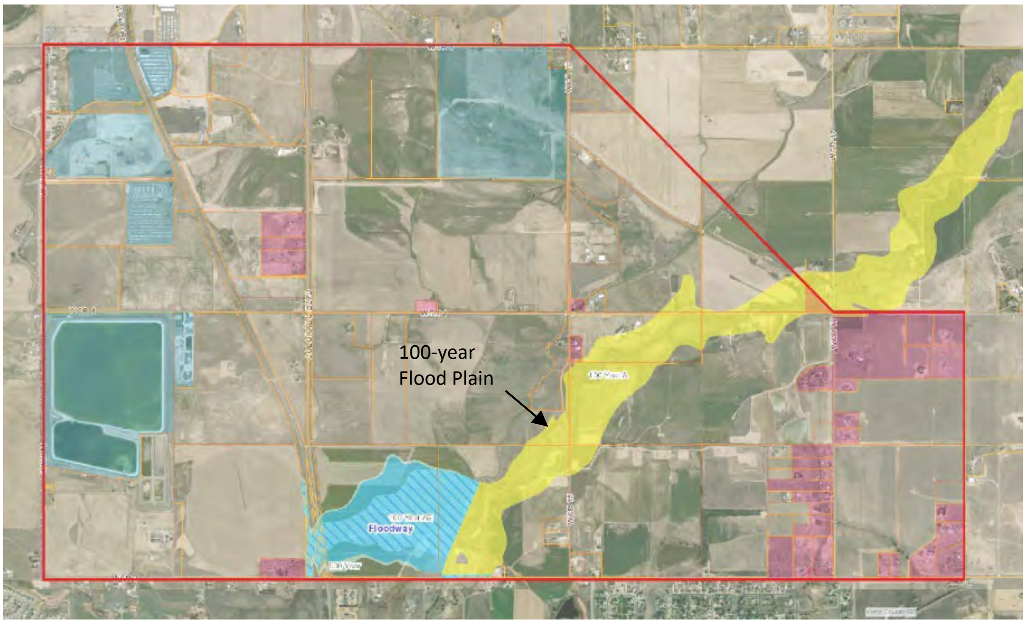


Figure 4-2: City of Northglenn North Service Area

Figure 4-3: Section Map for North Service Area

4.1.3 Land Use

Land use and zoning are managed by Northglenn's Planning and Development Division and guided by the Comprehensive Plan (Northglenn, 2010). **Figure 4-4** shows the "current land use" map from the Comprehensive Plan for the SSA and Section 36 of the NSA. Northglenn also provided 2020 acreage by land use category (**Table 4-1**).

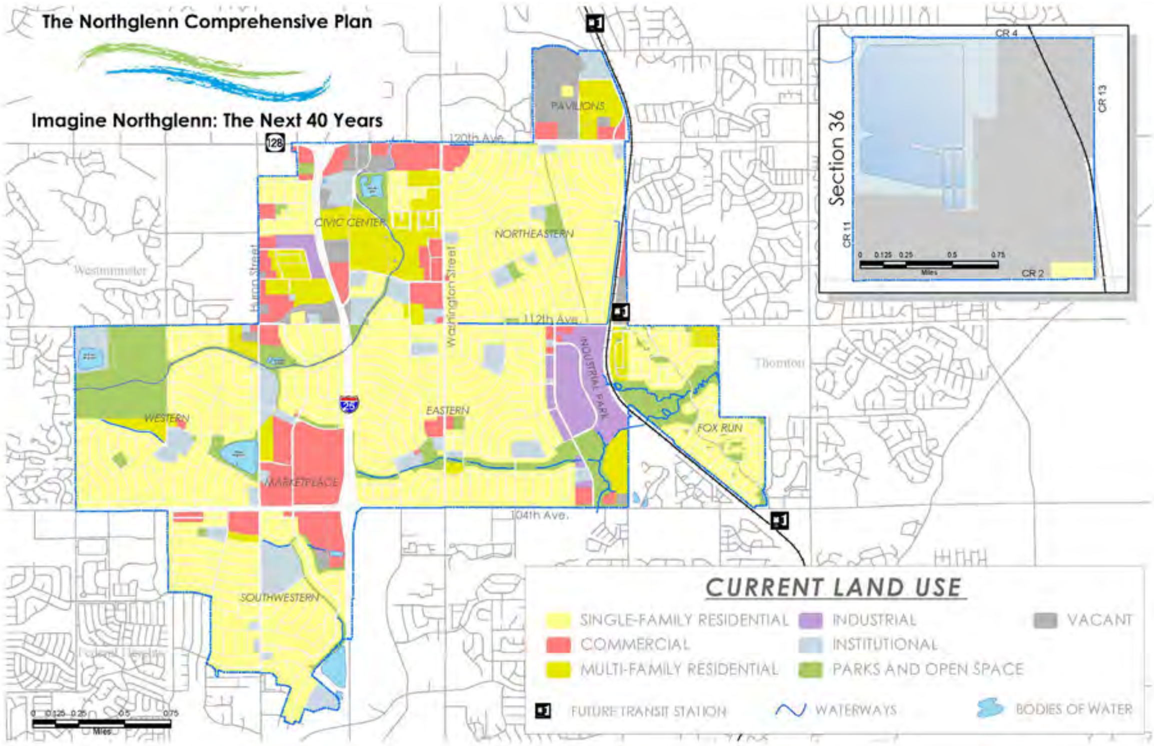


Figure 4-4: Current Land Use. Source: City of Northglenn Comprehensive Plan

Table 4-1: 2020 Acreage by Land Use Category (City of Northglenn, 2020)

Land Use Category	Acres
Agricultural	665
Commercial	298
Industrial	160
Single-Family Residential	1,621
Multi-Family Residential	160
Parks and Open Space	428
Public Facilities	143
Planned Development	497

4.1.4 Zoning

Northglenn updated the names and designations for all zoning districts as part of the newly adopted Unified Development Ordinance (UDO) (Northglenn, 2019). The 2019 Zoning Map can be found at the following link and is provided in **Appendix D**:
https://www.northglenn.org/Departments/Planning%20&%20Development/Planning/UDO/UDOZoningMap_Feb2019_resized.pdf

The recently completed 2020 Water Treatment Plant (WTP) MP identified four remaining areas within the SSA that may be developed prior to reaching buildout (JVA Consulting Engineers, 2020). These areas and the anticipated number of water taps for each area are summarized in **Table 4-2**.

Table 4-2: Summary of Tap Development from 2020 Water Master Plan

Area	Water Taps
Current SSA	10,295
Karl's Farm	800
112th Station Area	216
Civic Center	154
Market Place	185
TOTAL	**11,650**

Northglenn's potable water system does not extend beyond the SSA. Development of the NSA is hampered by a lack of potable water. Nearby municipalities to the NSA include the City and County of Broomfield, City of Dacono, City of Thornton, and Todd Creek. There are no plans for any of these entities to provide water to the NSA. Existing homes and businesses are presumed to rely on well water.

The 20-year planning horizon includes development of NSA Section 36 only. Section 36 includes Northglenn's WWTP and Bull Reservoir (refer to Figure 4-2). The remainder of Section 36 is planned to be a mix of industrial and commercial uses with some green space.

4.1.5 Current Wastewater Utility Service Area (WUSA) and Growth Management Area (GMA)

The WUSA is the portion of the GMA requiring wastewater utility service through the 20- year planning horizon. The collection system for the service area is shown in **Figure 4-5**. The figure is unchanged from the previous MP (HDR, 2012) and Northglenn noted that there have been no significant changes to the collection system with the exception of the build-out of Karl's Farm. The build-out of Karl's Farm has been included in flow projection calculations.

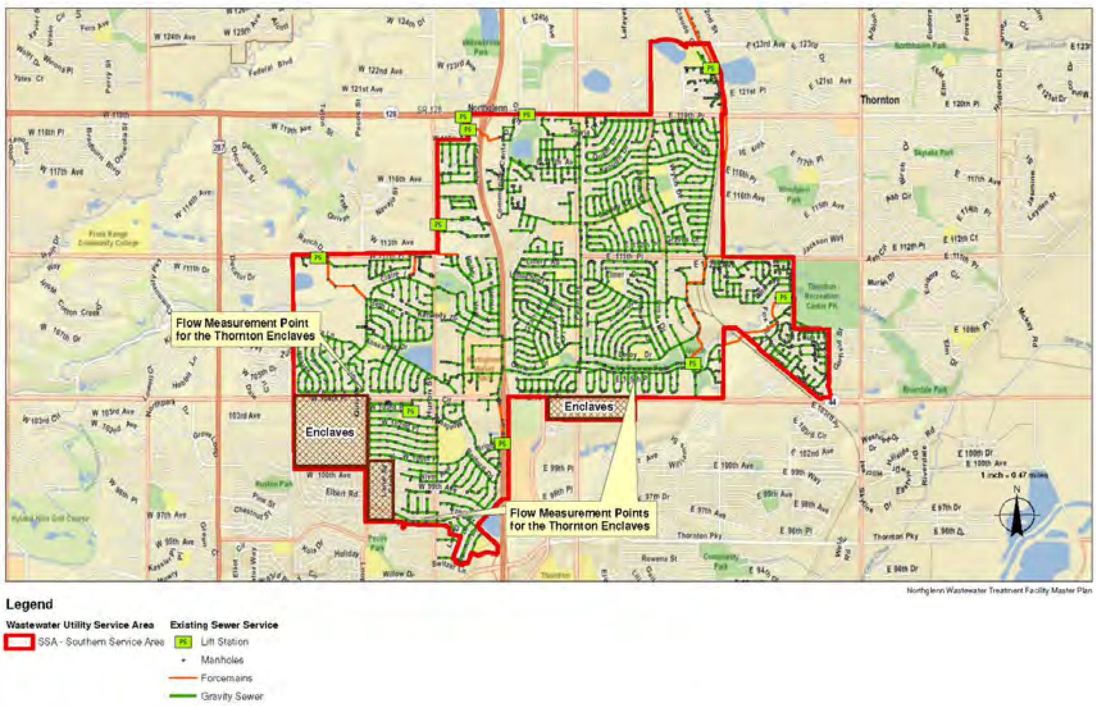


Figure 4-5: Existing Sewer Service. Source: HDR, 2012

4.1.6 Current Service Area Population

Historic population growth is summarized in **Table 4-3**. Between 2010 and 2016, the population increased at an average rate of 1.7% per year. The last four years have seen slight decreases in the total population from 39,163 down to 38,694 residents. This is an average population decrease of 0.3% per year.

Table 4-3: Northglenn Population

Year	Population	Data Source
2010	35,759	https://www.census.gov/quickfacts/northglenncitycolorado
2011	36,428	https://www.colorado-demographics.com/northglenn-demographics
2012	36,933	
2013	37,448	
2014	38,478	
2015	38,939	
2016	39,163	
2017	39,017	
2018	38,918	https://www.colorado-demographics.com/northglenn-demographics
2019	38,819	https://www.census.gov/quickfacts/northglenncitycolorado
2020	38,694	https://www.northglenn.org/residents/about_northglenn/demographic_information.php

4.2 Current Wastewater Flows and Loads

Eleven years of influent and effluent data from 2010 through 2020 were evaluated to document current wastewater flows and loads. Northglenn provided individual sample results and average daily flows to enable calculation of daily, weekly, and monthly flows and loading rates, per capita generation rates, and peaking factors. Influent flows were paired with sample results collected on the same day when calculating loading rates.

4.2.1 Historical Influent Flow Data

Influent treatment plant flows are summarized in **Table 4-4**. Annual average flows ranged between 3.02 and 3.53 mgd from year 2010 to 2020. The highest annual average flow was reported in 2015 and corresponds to extreme weather events in Colorado that caused extensive flooding in the Denver metro area and the north front range. Annual average flows were calculated by taking the average of the daily flow readings reported for each day throughout each calendar year. These flows are remarkably similar to those reported in the 2003 MP, which reported annual average flows between 3.14 and 4.01 mgd from year 1990 through 2002 (Integra Engineering, 2003). Although population has increased since 2002, the use of low water use fixtures in new construction and renovated properties likely contributes to keeping average daily flows low.

Table 4-4: Average Daily Influent Flow

Month	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020
January	3.07	3.02	3.13	2.94	2.99	2.98	3.10	2.85	3.03	2.93	2.90
February	3.05	3.02	3.12	2.85	2.91	3.03	3.22	2.77	3.04	2.94	2.94
March	3.20	2.93	3.02	3.02	3.05	3.17	3.40	3.15	3.07	3.07	3.07
April	3.63	2.90	2.97	3.19	3.32	3.33	3.99	2.99	3.21	3.11	3.16
May	4.00	3.68	3.14	3.36	3.51	5.19	4.21	3.67	3.38	3.36	3.25

Month	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020
June	4.09	3.46	3.06	3.18	3.28	4.50	3.79	3.81	3.34	3.43	3.28
July	3.57	3.82	3.05	3.01	3.37	3.72	3.41	2.87	3.43	3.43	3.13
August	3.28	3.37	3.00	2.94	3.33	3.33	3.20	3.37	3.29	3.22	3.05
September	3.43	3.13	3.09	3.90	3.23	3.31	3.00	3.17	3.14	3.15	3.07
October	3.08	3.02	2.87	3.13	3.13	3.20	2.90	3.35	2.98	2.96	2.89
November	3.19	3.20	2.91	3.08	3.05	3.29	2.94	3.03	2.92	2.92	2.85
December	2.92	3.12	2.93	3.01	2.93	3.21	2.87	2.96	2.85	3.04	2.82
AAF	3.37	3.22	3.02	3.13	3.18	3.53	3.33	3.17	3.14	3.13	3.09
Min Day	1.61	1.28	1.67	1.36	2.29	1.55	1.65	0.08	2.64	1.97	2.75
Max Day	4.99	5.00	4.00	5.08	5.26	6.21	5.80	5.17	3.86	3.73	3.65
MDPF	1.48	1.55	1.32	1.62	1.66	1.76	1.74	1.63	1.23	1.19	1.18
MM	4.09	3.82	3.14	3.90	3.51	5.19	4.21	3.81	3.43	3.43	3.29
MMPF	1.21	1.19	1.04	1.25	1.10	1.47	1.26	1.20	1.09	1.10	1.06

Note: AAF = annual average flow, MM = Average daily flow in the maximum months, MMPF = MM divided by AAF, MDPF = maximum day divided by AAF

4.2.1.1 Averages, Peaks, and Unit Volumes

Minimum and maximum daily flows are important when sizing hydraulic components including piping, pump systems, screening equipment, grit basins, secondary clarifiers, and chemical feed systems. The minimum and maximum daily flows are the lowest and highest average daily flows for a single day in each calendar year. The maximum day peaking factor was calculated by dividing the maximum daily flow for each year by the annual average daily flow for the year. Maximum day peaking factors ranged from 1.18 to 1.76 times the average daily flow. This is consistent with the maximum day peaking factors reported in the 2003 MP of 1.19 to 1.67 times the average daily flow (Integra Engineering, 2003). The maximum day peaking factor selected for this analysis was 1.75, which is the 95th percentile peak day peaking factor for the previous eleven years.

Monthly flows are important when sizing biological treatment processes like activated sludge. The permitted capacity of the treatment plant is based on the MM average daily flow. Large differences in flow rates from month to month can indicate excessive inflow and infiltration (I/I) to the collection system. Monthly average flows were calculated by taking the average daily flows for each day in a calendar month and averaging them together.

Average monthly flows were calculated for 128 months from January 1, 2010 through August 30, 2020. The average monthly influent flow exceeded 80% of the current permitted capacity of 4.2 mgd for 22 months and exceeded 95% of the current permitted capacity for 6 additional months.

The MM peaking factor was calculated by taking the highest average monthly flow for each year and dividing it by the annual average flow for that year. MM peaking factors range from 1.04 in 2012 to 1.47 in 2015. Exceptionally high influent flows were reported for May 2015. The front range received over 8-inches of rain in the first three weeks of May 2015. Northglenn typically receives 2.2 inches of precipitation in May. Extensive flooding throughout the Denver metro area covered manholes in streets and easements and is the likely cause of increased influent flows that month. This amount of rainfall was referred to by the National Weather Service as a 100-year probability event. For this reason, the MM peaking factor of 1.47 for 2015 was excluded from consideration when selecting the MM peaking factor to use for planning purposes. Instead, a MM peaking factor of 1.27 was selected for planning purposes. This is the 95th percentile peaking factor after excluding the average monthly flow for May 2015.

Figure 4-6 shows the average influent flows by month from years 2010 through 2020. The error bars indicate one standard deviation. Influent flows are higher, on average, in May, June, and July and more variable during this three-month period.

Peak hour and instantaneous flow data were not available for evaluation. All the wastewater generated in the SSA is conveyed to lift station A. Pump cycle times determine the peak hour and instantaneous flows observed at the WWTP headworks. Diurnal data was collected at lift station A for one-week in 2001 for the 2003 MP (Integra Engineering, 2003). The 2003 MP selected a peak hour peaking factor of 3.0 based on the 2001 diurnal data and a peaking factor calculation from the Denver Regional Council of Governments (DRCOG). Note that lift station A is currently under replacement and the cycle times may change.

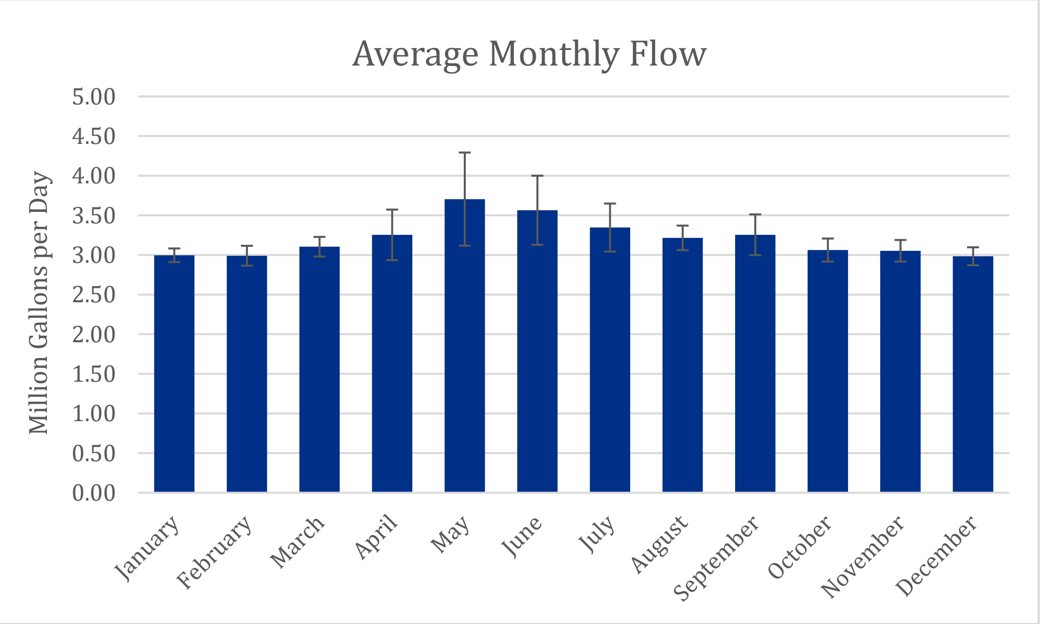


Figure 4-6: Average Monthly Influent Flows by Month 2010-2020

The peak hour peaking factor was estimated using the formula given in Water Pollution Control Program Policy Number: WPC-DR-1 (CDPHE, 2012). The calculated peaking factor was less than 2.5 for the current population and for projected populations. Section 3.2.2.d of WPC-DR-1 does not allow for peak hour peaking factors lower than 2.5. Therefore, a peak hour peaking factor of 2.5 was selected for planning purposes.

$$\frac{Q\ peak\ hourly}{Q\ design\ average} = \frac{18 + \sqrt{P}}{4 + \sqrt{P}}$$

4.2.1.2 Assessment of Inflow and Infiltration (I/I)

I/I contributions were also analyzed during the WQA during the most recent permit renewal. The permit fact sheet (CDPHE, 2019) states:

"The permittee operates a separate sewer system that conveys wastewater to the WWTP. I/I into the collection system has been evaluated for this renewal. Inflow is water, other than wastewater, that enters a sewer system from sources such as roof leaders, cellar drains, yard drains, area drains, foundation drains, drains from springs and swampy areas, manhole covers, cross sections between storm drains and sanitary sewers, catch basins, cooling towers, storm waters, surface runoff, street wash waters or other drainage. Inflow does not include, and is distinguished from, infiltration. (40 CFR 35.2005 Definitions)

Infiltration is water other than wastewater that enters a sewer system (including sewer service connections and foundation drains) from the ground through such means as defective pipes, pipe joints, connections, or manholes. Infiltration does not include, and is distinguished from, inflow. (40 CFR35.2005 Definitions)

Contributions from I/I are assessed by calculating the gallons per capita per day. Gallons per capita per day is calculated by using the daily average influent flows for the three maximum flow months during the past calendar year, reported in Part D of the facility's permit application. If the data on the application is outdated or not reported in the application, the three maximum 30-day average influent flows for the past calendar year may be used instead. The facility reports the total estimated flows for residential, industrial, commercial, and also the population of the service area in Part C of the permit application. The calculation to determine gallons per capita per day is:

$$gallons\ per\ capita\ per\ day = gal.\ per\ day/population\ X\ \%residential\ flows$$

$$\%\ residential\ flows = residential\ flows/(residential + commercial + industrial\ flows)\ X\ 100\%$$

For this facility the average of the daily average influent flows for the maximum three flow months is 3,383,333 gallons per day (gpd). Based on data submitted in the permit application, the facility's percent of residential flows is 94%. Based on the service area population of 38,905, the estimated influent flow is 84.5 gallons per capita per day.

The facility does not exceed the 120 gallons per capita per day threshold used by the division to screen for excessive infiltration."

Note that Northglenn has a recurring sanitary sewer rehabilitation program to further reduce their I/I.

4.2.2 Historical Wastewater Loadings Data

The same eleven years of data were reviewed to document historical wastewater loading to the WWTP. Influent flows were paired with sample results collected on the same day when calculating loading rates.

4.2.2.1 Biochemical Oxygen Demand (BOD)

BOD is one method of measuring the organic content of wastewater. In the United States, the test is conducted over 5-days and is abbreviated as BOD_5. The test extrapolates the concentration of biodegradable organic compounds from the amount of oxygen consumed by microorganisms in the test. The influent BOD and NH_3 loads determine the size of the secondary treatment process, aeration system, solids handling processes, and quantity of biosolids produced.

Influent BOD_5 concentrations are summarized in **Table 4-5**. Concentrations range from 154.5 mg/L to 327.3 mg/L. BOD_5 concentrations are typically between 133 and 400 mg/L, depending on per capita water usage in the service area and I/I (Metcalf & Eddy, AECOM, 2014). Northglenn monitors the ratio between influent TSS and BOD_5 as a secondary check against potential sampling and analysis errors. For domestic wastewater, the carbonaceous BOD_5 to TSS ratio is expected to be between 0.82 and 1.43.

Table 4-5: Average Influent BOD_5 Concentration, mg/L

Month	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020
January	241.6	259.2	246.9	241.8	216.9	285.5	244.4	244.2	245.7	253.7	255.1
February	226.6	253.7	223.4	245.4	227.9	264.6	247.5	232.6	271.4	250.0	265.0
March	199.3	243.4	230.8	233.8	189.0	206.4	214.3	235.8	287.5	268.1	268.5
April	193.1	221.4	204.3	230.0	215.3	250.9	201.3	227.3	266.2	228.1	237.8
May	178.3	192.5	204.8	233.5	213.1	176.5	188.0	228.7	239.2	227.3	208.8
June	174.2	182.5	193.9	204.7	187.0	154.5	178.5	181.0	248.4	231.7	174.0
July	178.4	168.3	205.0	223.0	218.9	197.0	198.4	195.7	226.4	210.6	178.6
August	194.2	174.3	201.6	225.7	201.5	204.2	215.0	204.9	229.4	224.2	176.4
September	169.4	261.8	218.3	200.9	221.7	238.3	213.4	214.0	231.3	218.1	215.0
October	198.4	213.7	231.2	197.9	232.5	218.6	227.7	218.5	219.4	219.5	242.4
November	229.0	218.3	247.0	208.1	233.2	248.1	232.4	229.0	227.9	327.3	241.0
December	245.3	235.7	253.7	243.4	268.8	265.6	240.2	267.0	240.2	297.5	228.8
Avg Day	208.2	217.9	221.9	224.0	218.9	225.3	217.4	222.9	243.5	242.0	228.3
Min Day	138.3	144.1	122.2	121.5	106.1	95.6	141.2	58.8	169.6	106.0	154.0
Max Day	305.3	406.1	342.2	304.1	323.9	551.0	371.3	361.5	445.5	400.0	321.2

Influent BOD_5 loads are summarized in **Table 4-6**. Influent flows were paired with sample results collected on the same day when calculating loading rates. All load calculation results for a month are then averaged together to find the average monthly load. The mass of BOD_5 entering the facility has been relatively constant over the last 10 years. Loads increase between 2010 and 2015 as population increases. Loads decrease between 2016 and 2020 with decreasing population.

Table 4-6: Average Influent BOD_5 Load, Pounds per Day

Month	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020
January	6219	6516	6338	6086	5406	7103	6288	5987	6164	6217	6245
February	5788	6386	5919	5948	5561	6652	6627	4545	6684	6031	6516
March	5276	5949	5870	5800	4704	5451	6119	6264	7244	6712	6862
April	5825	5042	5078	6025	6033	6871	6928	5198	6987	5916	6157
May	5973	5879	5139	6658	6461	7411	6541	6616	6744	6395	5677
June	6034	5243	4954	5373	5128	5625	5775	5707	6853	6638	4703
July	5245	5352	5168	5641	6160	6150	5941	4457	6705	6116	4698
August	5267	4889	5046	5457	5511	5859	5758	5777	6313	5987	4461
September	4642	6803	5606	6287	5974	6603	5957	5548	6075	5748	5423
October	4992	5485	5557	5084	6159	5798	5326	6026	5463	5120	5862
November	6046	5830	5987	5392	5848	6746	5715	5652	5497	7884	5678
December	5898	6085	6138	6041	6550	7104	5668	6493	5787	7487	5355

Month	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020
AAD	5648	5783	5571	5817	5788	6425	6021	5725	6345	6305	5823
MM	6219	6803	6338	6658	6550	7411	6928	6616	7244	7884	6862
MMPF	1.10	1.18	1.14	1.14	1.13	1.15	1.15	1.16	1.14	1.25	1.18

Note: AAD = annual average day, MM = maximum months, MMPF = MM Peaking Factor

A MM peaking factor (MMPF) was calculated for each calendar year by taking the highest monthly average load and dividing it by the annual average daily load. The MMPF ranged from 1.10 to 1.25. A MMPF of 1.22 was selected for BOD_5 loading for planning purposes. This peaking factor represents the 95th percentile of estimated values.

4.2.2.2 Total Suspended Solids (TSS)

TSS is the measure of all particles suspended in water larger than about 1.2 microns. TSS loading is an important parameter for the sizing of primary clarifiers, secondary treatment processes, and solids handling processes.

Influent TSS concentrations are summarized in **Table 4-7**. Concentrations range from 199.8 mg/L to 356.7 mg/L. TSS concentrations in domestic wastewater are typically between 120 mg/L and 400 mg/L depending on per capita water usage and the quantity of I/I (Metcalf & Eddy, 2007).

Table 4-7: Average Influent TSS Concentration, mg/L

Month	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020
January	231	246	237	255	241	250	287	271	273	294	274
February	228	229	237	242	257	270	268	245	264	260	270
March	223	232	256	240	215	254	251	244	247	306	278
April	208	252	277	212	240	357	230	243	258	264	285
May	246	248	297	251	246	273	200	248	264	255	303
June	242	264	288	267	246	227	213	214	269	235	307
July	285	259	316	291	278	276	247	246	268	266	291
August	283	303	317	302	292	242	305	248	273	285	288
September	237	288	320	282	267	310	250	252	305	263	296
October	327	322	250	223	225	286	265	251	272	237	319
November	234	252	261	234	227	284	264	241	248	225	306
December	264	247	239	281	239	280	238	253	268	264	297
Avg Day	242	262	275	257	248	277	252	246	268	263	287
Min Day	172	169	205	157	107	158	144	168	184	190	223
Max Day	556	436	388	385	380	578	414	492	352	387	497

Influent TSS loads are summarized in **Table 4-8**. Similar to BOD_5 loading trends, TSS influent loading has remained relatively consistent over the last 10 years. MMPF for TSS loading rates ranged from 1.11 to 1.49. A MMPF of 1.42 was selected for planning purposes. This peaking factor represents the 95th percentile of estimated values.

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Table 4-8: Average Influent TSS Load, Pounds per Day

Month	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020
January	5949	6183	6100	6492	6020	6205	7376	6850	6777	7195	6692
February	5817	5756	6267	5867	6274	6792	7085	4999	6529	6277	6639
March	5902	5645	6360	5963	5352	6701	7200	6487	6218	7655	7172
April	6327	5744	6857	5576	6758	9867	7567	5565	6774	6882	7508
May	8256	7573	7494	7174	7646	12024	6919	7245	7417	7194	8290
June	8404	7556	7339	7015	6736	8241	6763	6943	7428	6742	8401
July	8333	8209	7954	7364	8031	8582	7256	5621	7838	7675	7648
August	7673	8489	7940	7289	8027	6880	8142	6935	7504	7615	7304
September	6476	7526	8266	9106	7209	8547	6654	6519	7979	7025	7687
October	8168	8213	6010	5712	5968	7530	6280	6994	6774	5914	7778
November	6171	6768	6320	6116	5766	7722	6432	5968	5977	5444	7332
December	6362	6356	5781	7015	5825	7495	5627	6136	6494	6669	7048
AAD	6618	7035	6895	6719	6634	8065	6941	6385	7006	6886	7483
MM	8404	8489	8266	9106	8031	12024	8142	7245	7979	7675	8401
MMPF	1.27	1.21	1.20	1.36	1.21	1.49	1.17	1.13	1.14	1.11	1.12

Note: AAD = annual average day, MM = maximum months, MMPF = MM Peaking Factor

4.2.2.3 Ammonia-Nitrogen (NH_3)

Nitrogen is one of the two most prominent macronutrients that contribute to eutrophication within aquatic systems. Eutrophication occurs when there is an excessive growth of phytoplankton which can decrease dissolved oxygen concentrations leading to the destruction of aquatic life diversity (WEF, 2011). Nitrogen within aquatic systems can be generally broken down into two categories: organic and inorganic. Inorganic nitrogen exists in four stable forms: NH_3, nitrate (NO_3), nitrite (NO_2), and nitrogen gas (N_2) (WEF, 2011). Secondary treatment processes may be designed to remove inorganic nitrogen by converting NH_3 to NO_3 through nitrification followed by conversion of NO_3 to insoluble N_2 through denitrification. Determining influent NH_3 concentrations and loads is important for design of the secondary treatment process dedicated to nitrification and denitrification, sizing the aeration system, and for sizing solids handling and digestion processes.

Table 4-9 summarizes the average influent NH_3concentrations. Concentrations range between 11 and 40 mg/L as N. NH_3 concentrations are typically between 12 and 45 mg/L as N for domestic wastewater depending on per capita water usage (Metcalf & Eddy, 2007).

Table 4-9: Average Influent Ammonia Concentration, mg/L-N

Month	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020
January	29.1	29.1	30.2	30.9	30.1	28.1	24.8	28.3	40.3	39.1	35.0
February	28.7	28.1	28.2	32.3	24.7	26.9	20.7	28.6	37.8	36.4	35.1
March	26.7	30.6	29.2	24.7	24.8	25.8	18.7	29.8	39.8	35.0	34.2
April	24.7	31.3	26.6	27.3	24.3	21.4	20.2	31.0	38.2	35.5	33.6
May	21.8	25.7	27.3	26.8	22.2	10.8	14.5	27.1	34.9	32.9	30.6
June	22.1	22.9	30.1	18.5	22.0	16.1	21.3	29.0	38.7	34.7	28.6
July	24.6	23.2	27.3	22.2	23.2	22.3	21.3	33.1	40.1	34.3	31.4
August	26.7	26.1	26.3	32.1	22.6	23.6	22.9	31.7	33.0	36.3	29.3
September	26.3	26.5	27.8	22.7	12.9	21.1	22.2	34.4	33.5	37.2	33.1
October	29.7	25.8	31.8	28.0	27.2	19.8	25.3	34.3	35.6	36.8	32.7

Month	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020
November	30.5	29.5	32.0	22.0	27.1	20.7	30.8	30.9	37.2	35.2	35.5
December	30.1	29.4	31.9	30.6	27.9	21.5	29.8	33.0	37.5	33.1	38.6
Avg Day	26.6	27.9	29.1	26.6	24.3	21.5	22.8	31.1	37.5	35.6	33.8
Min Day	17.4	20.6	17.6	5.9	17.0	6.6	11.0	22.6	29.1	29.9	28.6
Max Day	33.8	48.0	33.8	42.7	33.7	30.2	32.3	41.1	45.2	43.7	37.2

Table 4-10 summarizes NH_3 loading rates and estimated MMPF values. Influent NH_3 loads decreased from 2010-2015 from 735 to 598 ppd but increased from 2015 to 2020 with current loads ranging between 900 and 1000 ppd.

MMPF values ranged from 1.01 to 1.22. A MMPF value of 1.20 was selected for planning purposes. This represents the 95th percentile of estimated values.

Table 4-10: Average Influent Ammonia Load, Pounds per Day as N

Month	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020
January	746	731	776	773	756	703	634	594	996	978	807
February	730	706	763	789	603	673	544	548	947	898	878
March	709	748	650	626	623	670	548	852	1016	877	900
April	745	716	663	727	631	576	757	786	1017	944	896
May	722	765	669	762	674	485	528	797	977	946	800
June	766	656	747	488	581	601	678	914	1074	1008	802
July	723	722	698	558	654	686	595	946	1163	994	825
August	718	735	659	771	603	502	605	908	889	1015	727
September	720	684	698	675	686	623	541	895	860	1000	840
October	746	651	763	708	742	515	612	959	917	968	792
November	809	786	780	579	678	557	752	763	901	857	829
December	725	759	782	773	673	578	693	805	944	853	880
AAD	736	726	721	688	653	598	623	819	984	947	878
MM	809	786	782	789	756	703	757	959	1163	1015	900
MMPF	1.10	1.08	1.08	1.15	1.16	1.18	1.22	1.17	1.18	1.07	1.03

Note: AAD = annual average day, MM = maximum months, MMPF = MM Peaking Factor

4.2.2.4 Total Nitrogen (TN)

Industrial contributions of NO_2 and NO_3 are assumed to be minimal. Northglenn adds bioxide (calcium NO_3) at lift station A for odor control. Northglenn will have the ability to add bioxide or ferric chloride at the new bunker hill lift station replacement (which may be renamed the Karl's Farm Lift Station) if desired. While not part of the initial construction, Northglenn has built the lift station to have future chemical injection. The added NO_3 is consumed in the force main between lift station A and the facility headworks. Influent NO_3 concentrations are monitored at least weekly for process control purposes (**Table 4-11**). Residual NO_3 is generally kept below 1 mg/L as N and does not contribute significantly to the influent TIN concentrations. For planning purposes, it has been assumed that the influent NH_3 and influent TIN concentrations are equivalent. A two-year pilot study was approved for ferric chloride and ferric sulfate beginning in May 2021. Pending results of the evaluation, Northglenn may switch from bioxide to ferric chloride for odor control in the future.

Table 4-11: Average Influent Nitrate-Nitrogen Concentrations, mg/L-N

Month	2014	2015	2016	2017	2018	2019	2020
January	0.57	0.90	0.51	0.67	0.92	0.60	0.63
February	0.42	0.73	0.69	0.72	0.65	0.61	0.60
March	0.36	0.65	1.46	0.64	0.60	0.86	0.63
April	0.40	0.57	2.03	0.71	0.57	0.57	0.52
May	0.42	3.72	1.45	0.66	0.53	0.56	ND
June	0.45	0.95	0.69	0.66	0.68	0.57	ND
July	0.37	0.52	0.86	0.78	2.65	0.57	ND
August	0.41	0.52	0.98	0.93	3.61	0.69	ND
September	0.55	0.58	1.54	1.00	2.06	0.62	ND
October	0.45	0.57	0.91	1.09	1.14	0.55	ND
November	0.47	0.48	0.89	1.28	0.62	0.57	ND
December	0.50	0.67	0.72	0.95	0.64	0.56	ND

Note: Influent NO_3 monitoring was temporarily discontinued in 2020 due to staffing shortages related to COVID-19.

4.2.2.5 Total Phosphorus (TP)

Phosphorus is another prominent macronutrient in aquatic systems that contributes to eutrophication. Phosphorus may be present in wastewater and aquatic systems in organic and inorganic forms. Inorganic phosphorus is present as orthophosphate (H_xPO_4) (WEF, 2011). Orthophosphates can be removed in wastewater treatment systems either through biological treatment or chemical precipitation.

Table 4-12 summarizes the average influent TP concentrations. Concentrations vary from 4.4 to 9.7 mg/L as P. Typical concentrations in municipal wastewater are between 4 and 12 mg/L as P (Metcalf & Eddy, 2007). Limited to no data were available for TP concentrations for the years 2010-2012 and 2014-2016.

Table 4-12: Average Influent Total Phosphorus Concentration, mg/L-P

Month	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020
January	ND	ND	ND	9.69	8.62	ND	ND	5.11	6.64	6.65	5.94
February	ND	ND	ND	9.53	7.20	ND	ND	5.54	6.71	6.64	6.64
March	ND	ND	ND	9.15	ND	ND	ND	5.20	6.55	6.75	6.17
April	ND	ND	ND	9.66	ND	ND	ND	5.30	6.24	6.22	5.85
May	ND	ND	ND	7.99	ND	ND	ND	4.41	6.17	5.74	5.69
June	ND	ND	ND	7.90	ND	ND	ND	4.68	6.17	5.96	4.66
July	ND	ND	7.32	8.17	ND	ND	ND	5.68	6.38	6.03	5.24
August	ND	ND	6.99	7.55	ND	ND	ND	5.80	6.70	6.60	6.25
September	ND	ND	8.55	7.69	ND	ND	ND	5.59	6.41	6.31	5.57
October	ND	ND	7.56	8.25	ND	ND	5.38	6.59	6.97	5.96	5.61
November	ND	ND	9.43	8.39	ND	ND	5.23	6.46	6.28	5.85	5.67
December	ND	ND	9.29	8.75	ND	ND	5.08	6.15	7.17	5.75	5.61
Avg Day	ND	ND	8.13	8.60	8.05	ND	5.20	5.64	6.52	6.21	6.08
Min Day	ND	ND	3.92	6.40	7.00	ND	4.62	3.43	3.52	4.82	4.66
Max Day	ND	ND	11.68	12.64	9.09	ND	6.14	8.32	7.58	7.58	7.02

Influent TP loads from 2012 to 2014 averaged around 210 ppd. From 2017 to 2020, phosphorus loads decreased to an average of around 160 ppd. **Table 4-13** summarizes TP loading rates and estimated MMPF values. MMPF values ranged from 1.03-1.25 between 2012 and 2020. A MMPF value of 1.22 was chosen as a basis for planning, which is representative of the 95th percentile of estimated values.

Table 4-13: Average Influent Total Phosphorus Load, Pounds per Day as P

Month	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020
January	ND	ND	ND	244	218	ND	ND	107	164	166	137
February	ND	ND	ND	237	174	ND	ND	107	168	164	166
March	ND	ND	ND	232	ND	ND	ND	149	167	169	162
April	ND	ND	ND	257	ND	ND	ND	135	166	166	158
May	ND	ND	ND	227	ND	ND	ND	130	172	165	156
June	ND	ND	ND	208	ND	ND	ND	147	171	173	131
July	ND	ND	189	206	ND	ND	ND	162	185	175	138
August	ND	ND	176	181	ND	ND	ND	166	180	185	155
September	ND	ND	215	234	ND	ND	ND	145	165	169	142
October	ND	ND	181	209	ND	ND	130	185	179	157	136
November	ND	ND	223	216	ND	ND	127	160	152	142	133
December	ND	ND	228	219	ND	ND	118	150	181	148	128
AAD	ND	ND	201	223	200	ND	124	148	171	165	159
MM	ND	ND	228	244	218	ND	130	166	185	185	166
MMPF	ND	ND	1.14	1.15	1.09	ND	1.04	1.25	1.08	1.12	1.04

Note: AAD = annual average day, MM = maximum months, MMPF = MM Peaking Factor

4.2.2.6 Temperature

Influent temperature is not monitored; however, operations staff record the water temperature in the activated sludge process daily. Aeration basin temperature data is summarized in **Table 4-14** and shown graphically in **Figure 4-7**. Average monthly water temperatures have a strong seasonal pattern as expected. Average monthly water temperatures in the aeration basins range between 12.7 °C and 23.2 °C. Daily water temperatures range between 8.3 °C and 24.3 °C.

Table 4-14: Average Monthly Water Temperature in the Aeration Basins (°C)

Month	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020	Avg
January	14.9	15.7	15.3	14.6	13.9	14.2	13.4	14.0	15.3	15.3	14.2	14.6
February	14.5	14.7	14.1	13.1	13.2	13.9	13.0	13.8	15.1	14.4	12.7	13.9
March	14.4	15.0	15.1	13.2	13.8	14.4	14.3	14.5	15.1	14.3	13.4	14.3
April	15.3	16.3	16.8	14.3	15.2	15.8	14.4	15.9	16.1	16.0	16.2	15.7
May	16.6	16.8	18.6	16.6	16.7	16.0	15.6	18.1	18.3	17.2	17.7	17.1
June	18.9	18.7	20.3	19.1	19.2	18.7	18.8	20.2	20.5	19.4	19.7	19.4
July	20.8	21.0	22.3	21.1	21.2	20.8	20.7	21.9	22.5	21.5	21.4	21.4
August	22.2	22.4	23.2	22.1	21.7	21.8	21.6	22.3	22.9	22.9	22.7	22.3
September	22.3	22.5	22.9	21.7	21.2	21.6	21.5	22.6	22.5	22.7	22.8	22.2
October	21.6	21.7	20.7	20.0	20.0	20.1	20.3	20.3	20.6	19.9	21.6	20.6
November	19.4	18.8	18.6	18.1	17.3	17.4	18.6	19.5	18.7	17.7	17.8	18.3
December	17.2	16.4	16.2	15.4	15.4	14.3	15.0	17.4	16.6	15.8	16.9	16.1

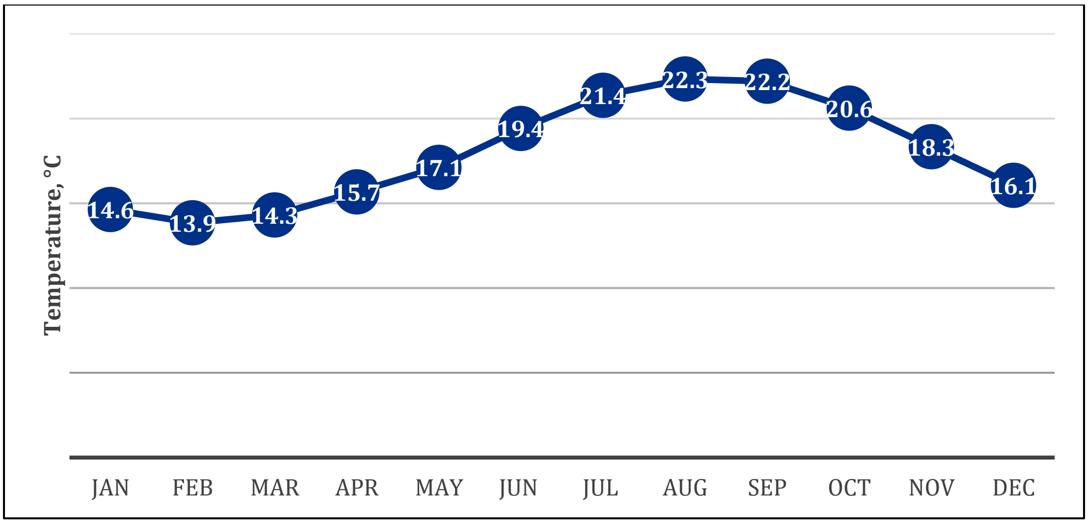


Figure 4-7: Average Aeration Basin Water Temperature

4.2.2.7 Other Constituents of Concern

No additional constituents of concern were reviewed for existing loading documentation in this Plan. Discussion of other constituents of concern based on the anticipated implementation of future regulatory updates were discussed in Section 3.

4.2.2.8 Peaking Factor Summary

Table 4-15 summarizes the peaking factors selected for this Plan as well as those previously estimated in the 2012 and 2003 MPs. The peak hour peaking factor was calculated using the NFRWQPA method.

Table 4-15: Summary of Flow and Load Peaking Factors

Parameter	Peak Hour	Peak Day	Peak Week	MMPF		
				2020 Plan	2012 Plan	2003 Plan
Flow	2.5	1.75	1.65	1.27	1.20	1.22
BOD_5	Not applicable		1.65	1.22	ND	1.40
TSS			2.04	1.42	ND	1.50
NH_3-H			1.67	1.20	ND	1.35
TP			1.54	1.22	ND	1.20

Note: MMPF = Maximum Month Peaking Factor

4.2.2.9 Per Capita Generation Rates

Per capita generation rates were estimated for each influent parameter by dividing the average daily flow or load by the estimated population served. Population numbers were listed in Table 4-3. **Table 4-16** summarizes the estimated average day per capita generation rates from 2010-2020. Per capita generation rates are consistent with published values, summarized in **Table 4-17**.

Table 4-16: Average Day Per Capita Generation Rates

Year	Flow	BOD5	TSS	NH_3-N	TP
	gpcd	ppcd			
2010	94.4	0.16	0.19	0.021	ND
2011	88.5	0.16	0.19	0.020	ND
2012	81.8	0.15	0.19	0.020	0.005
2013	83.7	0.16	0.18	0.018	0.006
2014	82.6	0.15	0.17	0.017	0.005
2015	90.5	0.16	0.21	0.015	ND
2016	85.2	0.15	0.18	0.016	0.003
2017	81.2	0.15	0.16	0.021	0.004
2018	80.7	0.16	0.18	0.025	0.004
2019	80.7	0.16	0.18	0.024	0.004
2020	79.9	0.15	0.19	0.023	0.004
Average	84.5	0.16	0.18	0.020	0.005
95th Percentile	92.5	0.16	0.20	0.025	0.006
99th Percentile	94.0	0.16	0.21	0.025	0.006

Note: gcpd = generation per capita per day, ppcd = pounds per capita per day

Table 4-17: Typical Average Day Per Capita Generation Rates

Parameter	units	Range	Typical without ground-up kitchen waste	99th Percentile Northglenn
Flow	gpcd	40 - 130	-	94.0
BOD_5	ppcd	0.11 – 0.26	0.18	0.16
TSS	ppcd	0.13 – 0.33	0.20	0.21
NH_3-N	ppcd	0.011 – 0.026	0.017	0.025
TP	ppcd	0.003 – 0.010	0.007	0.006

Source: Metcalf & Eddy, AECOM, 2014; Metcalf & Eddy, 2007

4.3 Existing Wastewater Treatment System

4.3.1 Description of Existing Treatment System

The Northglenn WWTP has a permitted capacity of 4.2 mgd and 7,916 ppd BOD_5, which are specified in Site Approval 4806 (**Appendix E**). Permitted capacity is based on the maximum average monthly flow and load in a calendar year.

The liquid stream of the Northglenn WWTP includes headworks with screening, degritting, and flow measurement followed by a 3-stage biological nutrient removal (BNR) activated sludge process, secondary clarification, and UV disinfection. Treated effluent may be discharged directly to either the Thompson Ditch or Big Dry Creek or be stored in Bull Reservoir prior to discharge. Bull Reservoir discharges to either Bull Canal, Thompson Ditch, or Big Dry Creek. Screenings and grit are sent to a landfill. Waste activated sludge (WAS) is transferred to one of two solids handling ponds located adjacent to Bull Reservoir and north of the mechanical treatment plant.

The Northglenn WWTP process diagram and hydraulic profile are shown in **Figures 4-8 and 4-9**, respectively.

Figure 4-8
NORTHGLENN WWTP
PROCESS FLOW SCHEMATIC

HYDRAULIC PROFILE

SCALE: NTS

LEGEND

5112.61
5112.24
= (PEAK HOUR FLOW) / (MAX MONTH FLOW) = 11.0 MGD / 5.22 MGD

(RE:NOTE 1, DWG G-004)

CDM Smith

Figure 4-9
NORTHGLENN WWTP
PROCESS FLOW SCHEMATIC

4.3.1.1 Collection System

The collection system contains approximately 117 miles of pipeline and 10 pump stations. Most of the wastewater generated in the SSA is eventually conveyed to lift station A. Lift station A is located at the intersection of 105th Place and Irma Drive. A 27-inch diameter, 48,100-foot-long force main conveys wastewater from lift station A to the WWTP. The force main was constructed in 1981. The Bunker Hill pump station, located at 12301 Claude Court, also discharges into the 27-inch diameter force main. The force main crosses into the NSA at 168th Avenue about halfway between Weld County Road 11 and the mechanical treatment plant. The force main angles up to the northeast before entering the force main termination vault.

Northglenn completed an evaluation of the hydraulic capacity of the collection system in 2007 utilizing H2OMap Sewer® by MWH Soft (Integra Engineering, 2007). System maps may be found in that report. Detailed information on all ten of Northglenn's pump stations may be found in section 5 of the 2003 MP (Integra Engineering, 2003).

4.3.1.2 Headworks

Headworks facilities consist of the force main termination vault, mechanical bar screens, screenings washer compactors, manual bar screen, stacked tray grit basin, flow monitoring, and odor control system. Design flows are listed in **Table 4-18** (Providence, 2016). Design flows are hydraulic capacities only and do not reflect downstream process treatment capacities.

Table 4-18 Headworks Design Flows

Parameter	mgd
Average Daily Flow	4.35
Maximum Month Flow	5.22
Peak Hour Flow	11.0

Reference: Providence (2016)

The force main from lift station A connects to the force main termination vault via a 30-inch diameter PVC force main. A knife gate allows operations staff to close off the force main and isolate the termination vault. Influent wastewater may be directed into either or both of two 42-inch diameter pipelines using slide gates. The transition from single to multiple pipelines is made through a contoured channel (**Figure 4-10**). Each of the 42-inch pipelines transitions into an open channel within the headworks building (**Figure 4-11**). Each channel leads to a mechanical bar screen. A second set of slide gates allows operations staff to close off the influent pipelines from within the headworks building.

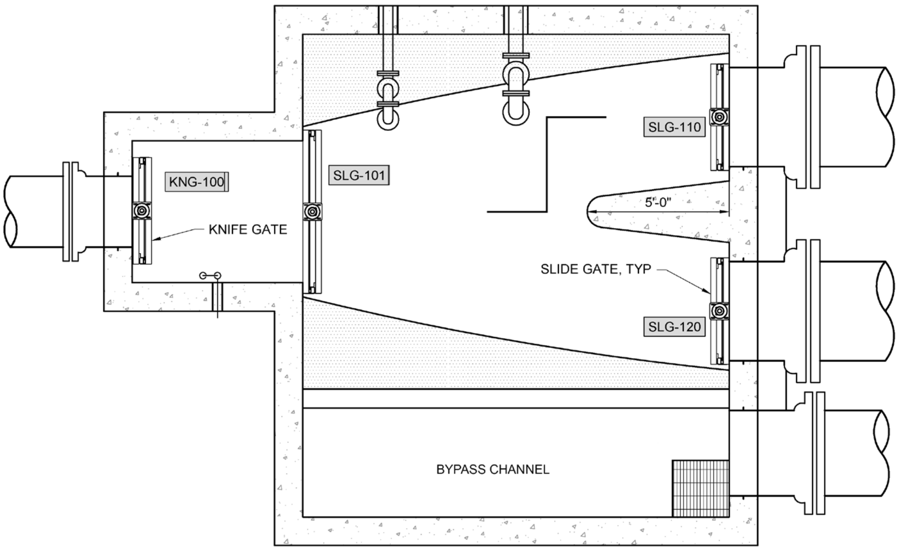


Figure 4-10: Force Main Termination Vault

The termination vault also contains a bypass channel. In the event of a high flow event or a blockage in either of the 42-inch influent lines, influent will accumulate in the termination vault until the water level becomes high enough to overflow into the bypass channel. The bypass channel feeds into a 36-inch diameter pipeline, which transitions into an open channel within the headworks building that contains a manual bar screen. The bypass channel does not contain a slide gate for isolation.

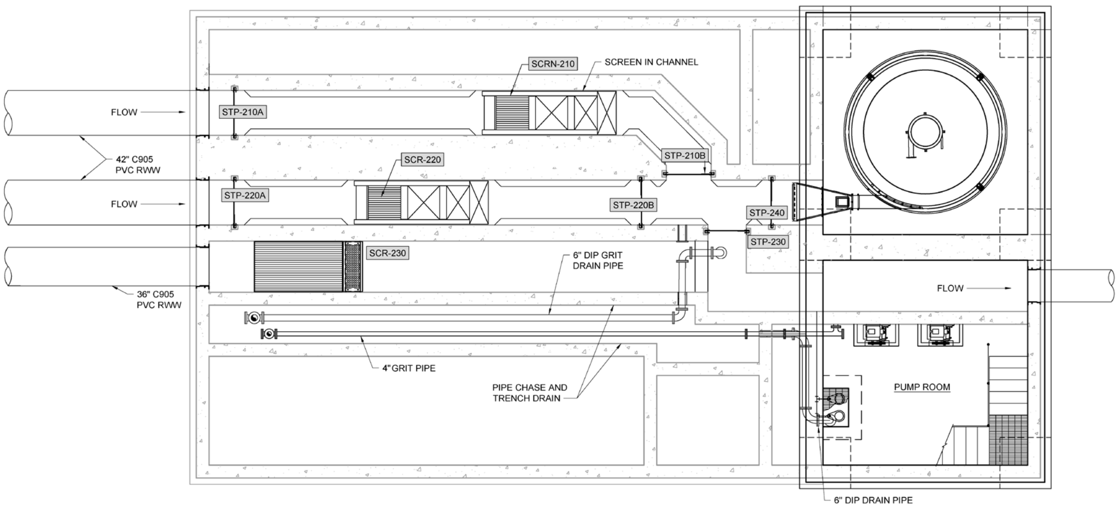


Figure 4-11: Headworks Plan View

4.3.1.2.1 Screenings Equipment

Screening equipment consists of two, Vulcan Model ESR Stair Screens, each located in a dedicated channel (Figure 4-11). Each channel is 3'-6" wide. One screen is normally in operation at all times while the other screen provides backup capacity. The stair screens are self-cleaning, fine screens with ¼-inch openings. Each stair screen has a hydraulic capacity of 11 mgd at peak hour flow. Flow velocity through a single screen at 11 mgd will be 2.12 fps when clean and 3.53 fps when 40% impeded (Providence, 2016).

Screened material is washed and compacted in two, dedicated Vulcan model EWP-250/800 washing presses to remove organic material and water. Each washing press has a capacity of 33 cu ft/hr. Rinse water is returned to the influent channel. Screened material is compacted, collected in dumpsters, and sent to landfill.

The bypass channel contains a manual bar screen with 1-inch openings. Screened material must be removed manually.

4.3.1.2.2 Grit Removal Equipment

Following screening, the three flow channels recombine to form a single channel. Screened influent is directed to a single, multi-tray vortex grit separator. Influent flow enters tangentially at the outer rim of the unit and is distributed to each of the stacked trays. Settled grit collects on the trays and is conveyed toward the center of the grit separator. Grit is funneled into the bottom of the basin where it is removed by two grit pumps located on the lower level of the headworks building. Degritted wastewater exits the unit by flowing over an effluent weir to move to the next unit process. Design criteria for the grit basin and grit pumps are listed in **Table 4-19**.

Table 4-19 Grit Removal Equipment Design Criteria

Parameter	Value
Grit Unit Manufacturer	Eutek / Hydro International
Model	HeadCell
Quantity	1
Diameter	12 feet
Design Capacity	12.5 mgd
Hydraulic Retention Time at Design Flow	4 minutes
Grit Pump Manufacturer	WEMCO
Quantity	2
Type	Dry pit, recessed impeller
Design Flow	200 gpm
Design Head	30 feet
Grit Pump Motor Manufacturer	Baldour
Horsepower	7.5

Reference: Providence (2016)

Grit is then pumped to a single Eutek TeaCup grit washer/classifier, also located in the headworks building, where it is washed to remove organic matter. The TeaCup uses centrifugal force to separate grit from lighter organic matter. Degritted wastewater exits the TeaCup through the top and is returned to the influent channel. Grit settles to the bottom of the TeaCup where it is discharged to a Eutek grit snail. The snail is a bucket style conveyor system that moves clean grit from the TeaCup into a dumpster. Water drains from the grit as it is conveyed and is returned to the influent channel.

The grit separator may be taken off-line for cleaning and maintenance by directing flow into a bypass channel with stop plates.

4.3.1.2.3 Odor Control System

The headworks building is equipped with an air ionization system for odor control. Ionized air is delivered to three areas of the headworks facility: the headspace of the force main termination vault, the headspace of the covered channels in the headworks building, and the occupied process space in the headworks building (Providence, 2016). The heating, ventilation, and air conditioning (HVAC) system was designed to provide 12 air changes per hour to the channel headspace and occupied process space and 24 air changes per hour to the force main termination vault. The ionization system design assumed a maximum hydrogen sulfide (H_2S) of 25 ppm in the force main termination vault and headworks channels and a maximum H_2S concentration of 5 ppm in the occupied process space. Additional design information may be found in the "Headworks Odor Control Design Memorandum" by Providence Infrastructure Consultants (January 8, 2016).

The headworks HVAC system was designed to operate under negative pressure to mitigate odors and minimize H_2S concentrations within the headworks building and force main termination vault. Three exhaust fans on the roof are rated for 4000 cubic feet per minute (cfm) each. Four exhaust vents extend down the inside walls of the headworks building; two on each side. The vents terminate within 18-inches of the floor. A fourth, smaller exhaust fan, rated for 1600 cfm, pulls air from the covered flow channel downstream of the grit basin. A makeup air unit (MAU-1) was designed to draw fresh air into the building at a rate of 11,300 cfm.

The headworks building is equipped with four air stream ionization units. They are located within an isolated mechanical HVAC room on the west side of the building. AIU 1 treats air withdrawn from the force main termination vault. AIU 2 treats air withdrawn from the headworks channels. AIU 3A and 3B treat air withdrawn from process areas within the headworks building.

The air exhaust and H_2S treatment systems do not function as intended. At start-up, unacceptably high concentrations of H_2S gas prevented safe entry into the headworks building. Operation staff added additional modules to the air ionization units and turned all units up to their maximum operating capacity. H_2S levels could not be reduced below OSHA Permissible Exposure Limit (PEL) standard of 15 ppm.

Ion generators work by charging particles in the air so that they are attracted to surfaces. They work well for removing small particles such as those found in tobacco smoke, but they do not remove gases or odors and are not very effective at removing larger particles like pollen and water droplets. Ion generators may be effective in oxidizing some odorous organic compounds and for controlling H_2S; however, independent peer-reviewed studies could not be found to confirm the manufacturer's claims.

Operations staff determined that the four exhaust vents, being located so close to the floor, were pulling air out of the covered channels and into the main building space. They have modified the HVAC system to operate under positive pressure by turning off all the rooftop exhaust fans. The makeup air unit (MAU-1) now forces air into the headworks building. The access hatches on the force main termination vault are kept open to provide a path for air to move from the headworks building, through the channels, into the termination vault, and out into the atmosphere. Air stream ionization unit AIU 1 has been taken out of service to help force air from the headworks building through the remaining units as it passes through to the termination vault.

The exhaust fan in the east wall of the headworks building remains in service. Operations staff have modified the exhaust duct by extending it below the covers and approximately 10 feet further into the room. It terminates perpendicular to the stairwell that leads to the grit pump room.

These operational changes have decreased H_2S concentrations within the headworks building to safe levels. However, odors remain high both in the headworks building and in and around the force main termination vault. Several processes, such as scrubbing, adsorption and condensation, could be considered for the treatment of H_2S. The physicochemical methods that can be used to remove pollutants from gas emissions have relatively high energy requirements and high chemical and disposal cost. Biofiltration is one of the processes of waste gas treatment and of odor control. Other options for Northglenn to consider are biofilters on the exhaust at the force main termination vault. Adding two additional H2S sensors and transmitters on the main level of the headworks building is also recommended.

Northglenn is currently piloting a super-oxygenation system. Super-oxygenating the raw influent wastewater at the force main termination vault may convert liquid phase H_2S into sulfate. Sulfate does not pose the same health, odor, and corrosion concerns as H_2S.

Northglenn has also recently received tentative permission from CDPHE to begin adding ferric chloride at lift station A to precipitate H_2S. CDPHE issued a modified discharge permit allowing ferric chloride addition in April 2021.

4.3.1.2.4 Side Stream Return Pipelines

The force main termination vault was originally designed and constructed to receive side stream flows from an existing groundwater lift station and from the solids handling lagoons (refer to Figure 4-10). The supernatant line from the solids handling lagoon was equipped with a flow meter. This enabled operations staff to subtract the supernatant flow from the influent flow.

CDPHE noted in their review of the Phase 2B: Process Design Report (Headworks and Clarifier Project, Phase 2B) that the side stream flows were being returned upstream of the influent autosampler. They were concerned that this practice might impact influent concentrations and loading calculations. Since the review comment was received, operations staff have relocated both supernatant lines to discharge downstream of the influent flow meter and autosampler.

4.3.1.2.5 Influent Flow Measurement

Influent flow leaves the headworks building through a 30-inch diameter pipeline. A meter vault is located adjacent to the southeast corner of the headworks building. The vault is outside of the headworks building. It contains a 30-inch diameter magnetic flow meter with sensor/recorder.

An autosampler sits on top of the vault. The autosampler intake line passes through the concrete top of the vault, down the inside wall of the vault, and into the 30-inch diameter pipeline. The intake line terminates at the center of the pipeline as shown in **Figure 4-12**.

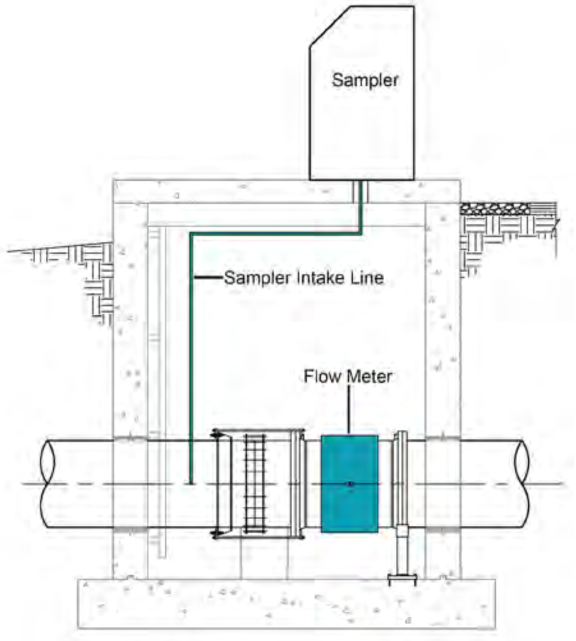


Figure 4-12: Influent Flow Meter Vault

4.3.1.3 Influent Splitter Box

A splitter box was added upstream of the activated sludge basins as part of the 2016 headworks project. In 2019, to help mitigate odors, the entire top of the splitter box was sealed off. The splitter box replaced a system of piping and valves that was previously used to distribute influent flow to the three activated sludge basins. A 36-inch diameter pipeline enters the splitter box from the influent flow meter vault. Three, 30-inch pipelines lead to each of the activated sludge basins. Each 30-inch pipeline is equipped with a downward opening weir gates.

4.3.1.4 Three-Stage Biological Nutrient Removal Activated Sludge Process

Secondary treatment consists of three activated sludge basins followed by three secondary clarifiers. Two treatment trains and two clarifiers are normally in service. Each activated sludge basin contains an anaerobic zone, divided anoxic zone, and an aerated zone. Design criteria are listed in **Table 4-20**.

The secondary treatment process was originally intended to operate in conjunction with upstream primary clarifiers. The primary clarifiers were anticipated to reduce the BOD_5 and TSS load to the activated sludge process by 30% and 50%, respectively. Primary clarifiers were not constructed due to financial constraints. Instead, the two smaller ponds from the original WWTP were repurposed to serve as both headworks and primary clarifiers. Influent was pumped from lift station A to the pond inlet structure. It passed through both ponds before being conveyed to the activated sludge basins. The ponds performed similarly to traditional primary clarifiers. The ponds were taken out of service and decommissioned as part of the headworks project in April 2017. This increased the BOD_5 and TSS loading rates to the activated sludge basins. As a result, CDPHE downrated the permitted capacity from 6.5 mgd and 12,650 ppd BOD_5 to 4.2 mgd and 7,916 ppd BOD_5. The hydraulic retention time (HRT) and organic loading criteria presented in Table 4-20 reflect the decrease in permitted capacity.

Table 4-20 Design Criteria for Activated Sludge Basins

Parameter	Value		
Number of Treatment Trains	3		
Basin Dimensions	Anaerobic	Anoxic	Aerobic
Length, feet	26	50	134
Width, feet	40	40	40
Depth, feet	16	16	16
Volume, cu ft, each	16,640	32,000	85,760
Volume, cu ft, total	49,920	96,000	257,280
Volume, mil. gal, each	0.127	0.24	0.64
Volume, mil. gal, total	0.38	0.72	1.92
HRT at MMF of 4.2 mgd, hours	2.2	4.1	11
Total HRT with all trains in service	17.3		
SRT (winter), days	18		
MLSS (winter), mg/L	3000		
Organic Loading Rate, lbs BOD5/1000 cu ft•d	19.6		

A concrete wall separates the anaerobic zone from the anoxic zone. A 4-foot slide gate at the bottom of the wall allows operations staff to isolate the anaerobic zone. The anoxic zones are divided into two smaller compartments with baffles as shown in **Figure 4-13**. The baffles mitigate transfer of oxygen from the aerobic zone into the anoxic zone. The baffles do not extend the full distance across the basins, which allows water to flow through the process. The baffles were originally made of a flexible geomembrane but were later replaced with stainless steel. Both the concrete wall and stainless-steel baffles extend above the water surface. MLSS exits the activated sludge basins through submerged slide gates. Foam trapping is an ongoing operational problem, especially during the winter months.

Figure 4-13: Baffle Walls in Anoxic Zone

The anaerobic and anoxic zones are equipped with 8.3 horsepower (HP) mixers. Each anaerobic zone contains one mixer. Each anoxic basin includes two mixers, one in each section.

Figure 4-14: Internal Mixed Liquor Recycle Header

Three, 25 HP internal mixed liquor recycle (IMLR) pumps transfer nitrified wastewater from the ends of each aerobic zone into their associated anoxic zones. The IMLR header is bifurcated to return MLSS into each half of the anoxic zone as shown in **Figure 4-14**. Each 25 HP pump is rated for 4,200 gpm (6.04 mgd) at a head of 13 feet. At the previous permitted capacity of 6.0 mgd, the IMLR pumps allowed for an internal recycle of up to 300% of influent flow with all three treatment trains in service. At the derated permitted capacity of 4.2 mgd, the IMLR pumps allow for an internal recycle of up to 432% of influent flow.

Adding another DO probe per basin is recommended to enhance process control.

4.3.1.4.1 Aeration System

The aerobic zones of each treatment train are equipped with fine bubble diffusers. Design criteria are presented in **Table 4-21** (HDR 2012, Providence 2016). A third blower was added as part of the headworks project in 2017.

Table 4-21 Aeration System Design Criteria

Parameter	Value		
Diffuser Type	Fine bubble		
Diffuser Size, inch diameter	9		
Number of Diffusers per Bank, each	300		
Number of Banks per Aerobic Zone, each	3		
Min/Max Airflow per Diffuser, SCFM	0.67/2.29		
Number of Blowers, each	3		
Purchase Date	2005	2000	2016
Manufacturer	Turblex	Turblex	Siemens
Make/Model	KA10 SVGL210	KA5 SVGL210	STC-GO-5SV-GL210
Horsepower	350	250	250
Maximum Air Flow	6000	4000	4000
Minimum Air Flow	3000	2000	2000

Temperatures in the blower room are elevated during the summer months, leading to alarms and equipment shutdowns due to high temperature interlocks in the blower controls. The existing blowers have air to oil coolers with fans located inside the building. The heat loss into the room overwhelms the HVAC system in the summer, and it is unable to adequately cool the room. The lubrication cooling system is provided by the blower manufacturer and integrated into the blower local controls. Any changes made to the system should be coordinated with Howden since maintaining adequate lubrication to the bearings is critical to operating and protecting the blowers. Fans likely could be located outside, but included below are some considerations:

- Evaluate if existing fans can be located outside or if new fans would be required. In either case, the fans would require a snow cover at a minimum.
- Consider noise impacts of locating fans outside. Quieter fans can be supplied (<85 dBA), but they may be larger than the existing fans.
- Consider temperature impacts on the oil that would have to be piped out to the fans. The oil lines would likely have to be heat traced, and it is best to keep the oil pipe runs as short as possible.
- Coordinate power and control of the fans with the blower local control panel.

4.3.1.4.2 Heat Exchange System

Heat exchange coils are located on the north and south walls of each aerated zone as shown in **Figure 4-15**. The heat exchange system was intended to waste heat from the blower and pump rooms into the aeration basin. The water in the aeration basin would function as a heat sink. However, the heat exchange system did not function as expected and operations staff took it out of service.

Figure 4-15: Heat Exchange Coils in the Aeration Basin

From 2006 until 2017, the lack of a mechanical headworks allowed excessive amounts of rags and trash to enter the treatment process. That material became entangled in the coils and required manual removal. This task was extremely time consuming. Operations staff have cut up and removed the heat exchange coils from the basins (**Figure 4-16**). All equipment within the aeration basins has been removed. It is recommended that Northglenn also demolish the associated heat exchange equipment in the main process and UV disinfection buildings.

Figure 4-16: Removal of Heat Exchange Coils

4.3.1.4.3 Secondary Clarifiers

There are three secondary clarifiers. Two are original to the mechanical plant and were built in 2006. The third clarifier was added in 2017. The clarifiers are flat-bottomed with inset launders. A suction header mechanism collects and removes settled sludge. Design criteria are listed in **Table 4-22**. Calculations assume an MLSS concentration of 3000 mg/L and a return activated sludge (RAS) flow equivalent to 70% of the facility influent flow. CDPHE design criteria may be exceeded if site-specific data supports higher surface overflow and/or loading rates.

Table 4-22 Secondary Clarifier Design Criteria

Parameter	Value	
Number of Clarifiers, each	3	
Diameter, feet	65	
Sidewater Depth, feet	14	
Surface Area, sq ft (each)	3316.6	
Volume, cu ft (each)	46432.8	
Surface Overflow Rate, gpd/sq ft	Design Criteria	CDPHE Criteria
Annual Average Daily Flow (3.06 mgd)	307.5	N/A
Maximum Month Flow (4.2 mgd)	422.1	600
Peak Day Flow (7.35 mgd)	738.7	N/A
Peak Hour Flow (10.5 mgd)	1055.3	1200
Solids Loading Rate, lb/sq ft*d	Design Criteria	CDPHE Criteria
Annual Average Daily Flow (3.06 mgd)	13.1	N/A
Maximum Month Flow (4.2 mgd)	18.0	29
Peak Day Flow (7.35 mgd)	31.4	N/A
Peak Hour Flow (10.5 mgd)	44.9	40

Assume MLSS = 3000 mg/L and RAS = 70% of Influent Flow

The inset launders trap algae and other floatable material since the skimmer arm does not reach the area between the inset launder and outer clarifier wall. Operations staff must manually clean this area regularly. Relocation of launders to the outer wall is recommended along with replacement of the skimmer arms with ones that extend all the way to the launders. This will minimize the amount of regular maintenance required.

Flows from the secondary clarifiers are combined prior to entering the UV disinfection channel.

4.3.1.4.4 Return Activated Sludge (RAS), Waste Activated Sludge (WAS), and Scum Pumps

The RAS and WAS pumps are located in the main process building east of the activated sludge basins. Design Criteria are presented in **Table 4-23**. The RAS pumps are manifolded together so that any RAS pump may be used with any secondary clarifier or activated sludge basin. The WAS pumps pull directly from the RAS manifold. This ensures a consistent WAS concentration. RAS is returned to the influent splitter box where it is blended with influent wastewater.

Table 4-23 RAS, WAS, and Scum Pump Criteria

Parameter	RAS Pumps	WAS Pumps	Scum Pumps
Number	3	2	2
Type	Centrifugal	Centrifugal	Progressing Cavity
Capacity, gpm (each)	1400	140	90
Capacity, mgd (each)	2.02	0.20	0.13
HP	20	5	5
Head, ft	30.3	54	90

CDPHE requires a firm RAS pumping capacity of 100 to 150% of the MM influent flow. The firm pumping capacity is defined as the pumping capacity available when the largest pump is out of service. The current permitted capacity is 4.2 mgd and the firm RAS pumping capacity is 4.04 mgd. Additional RAS capacity will be needed in the future.

4.3.1.5 Disinfection

Disinfection is accomplished with a UV disinfection system. UV light inactivates bacteria and viruses by damaging their genetic material. UV altered DNA or RNA inhibits cell replication or induces lethal mutations in daughter cells. Unable to reproduce, the damaged cells cannot reproduce and eventually die.

UV systems are sized based on the permitted effluent *E. coli* limits and the peak hourly flow rate. Design criteria for the UV disinfection units are presented in **Table 4-24**. The system contains two channels, each channel containing two banks. Under normal operating conditions, one bank per channel is in service while the other is in standby. The total capacity of the UV system is 14.0 mgd with both channels in service and one bank out of service per channel.

Disinfected effluent may be discharged directly to either the Thompson Ditch or Big Dry Creek through a Parshall flume or may be pumped to Bull Reservoir.

Table 4-24 UV System Design Criteria

Parameter	Value
Type	High-Efficiency Amalgam
Manufacturer	Trojan
Model	UV 3000-Plus
Number of Banks	4 (2 channels with 2 banks each)
Number of Modules per Bank	9
Number of Lamps per Module	8
Number of Lamps per Bank	72
UV Design Dose	30 mW/cm^2
UV Transmission	65%
Average Daily Design Flow (1 bank)	4.0 mgd
Peak Hour Design Flow (1 bank)	7.0 mgd

4.3.1.6 Solids Handling Lagoons

WAS is stabilized in one of two solids handling lagoons to produce Class B biosolids. The solids handling lagoons are located adjacent to and east of Bull Reservoir. The lagoons and Bull Reservoir were constructed between 1980 and 1982 and became operational on July 1, 1982. Design criteria are presented in **Table 4-25**.

Table 4-25 Solids Handling Lagoons Design Criteria

Parameter	Value
Number	2
Volume, mil. gal (each)	20.9
Liquid Depth, feet	25
Water Surface Area, ft (each)	218 x 818
Water Surface Area, acres (each)	4.09
Lagoon Bottom Area, ft (each)	118 x 718
Lagoon Bottom Area, acres (each)	1.95

Digested solids are removed from the lagoons annually by a contract hauler. After testing requirements have been met, the digested solids are land applied. Northglenn owns 142 acres of agricultural land adjacent to the wastewater treatment plant. While some biosolids are land applied here, most is hauled to other application sites owned by the contract hauler. Excess water (supernatant) is decanted from the lagoons and returned to the facility between the headworks and the splitter box.

4.3.1.7 Effluent Storage

Bull Reservoir provides 4049 acre-feet (1320 mgd) of effluent storage. At the current permitted capacity of 4.2 mgd, this represents 314 days of storage. No additional treatment takes place in the reservoir. Three centrifugal pumps transfer treated effluent from the end of the treatment process and into Bull Reservoir. Each 100 HP pump has a rated capacity of 3500 gpm against a discharge head of 100 feet.

Water is removed from Bull Reservoir via a pump station on the far north end of the reservoir. Three, vertical mixed-flow pumps can transfer flow either to Bull Canal or to a pipeline that leads to either the Thompson Ditch or Big Dry Creek. Each 150 HP pump is rated for 10,000 gpm against a discharge head of 40 ft. Northglenn is in the process of replacing these pumps, and in the future, each pump will be sized differently.

4.3.2 Performance of Existing Treatment System

The Northglenn WWTP produces high quality secondary effluent. Effluent data for BOD_5, TSS, NH_3, NO_3, TP, and alkalinity are presented in **Figures 4-17** through **4-22**. Non-detects are shown as equivalent to the reported method detection limit (MDL). Data gaps indicate periods of zero discharge.

As discussed earlier in this Plan, effluent may be discharged at multiple locations: two prior to effluent storage in Bull Reservoir and three after storage in Bull Reservoir. Not all parameters are required to be sampled at all locations. Bull Canal and Thompson Ditch are not considered waters of the state and have fewer monitoring requirements than Big Dry Creek.

Figures 4-17 through 4-22 include sample results for up to five locations each. They are:

1. **UV Effluent**: Orange data points. Effluent samples collected following UV disinfection and prior to discharge into Bull Reservoir. These samples are for process control only and may be either grab or composite samples.
2. **R4**: Yellow data points. Effluent samples collected at the discharge from Bull Reservoir. Water from this location may be discharged to Bull Canal, Thompson Ditch, or Big Dry

Creek. R4 is used to collect process control samples and may include both grab and composite samples.

3. **007 Big Dry Creek**: Gray data points. Effluent samples collected after UV disinfection and immediately prior to discharge to Big Dry Creek. This water has not been stored in Bull Reservoir.

4. **004 Big Dry Creek**: Blue data points. Effluent samples collected after Bull Reservoir and prior to discharge to Big Dry Creek. This water has been stored in Bull Reservoir.

5. **001 Bull Canal**: Green data points. Effluent samples collected after Bull Reservoir and prior to discharge to Bull Canal. This water has been stored in Bull Reservoir.

Effluent BOD_5 concentration have remained below the 30-day average discharge permit limit over the previous five years (Figure 4-17).

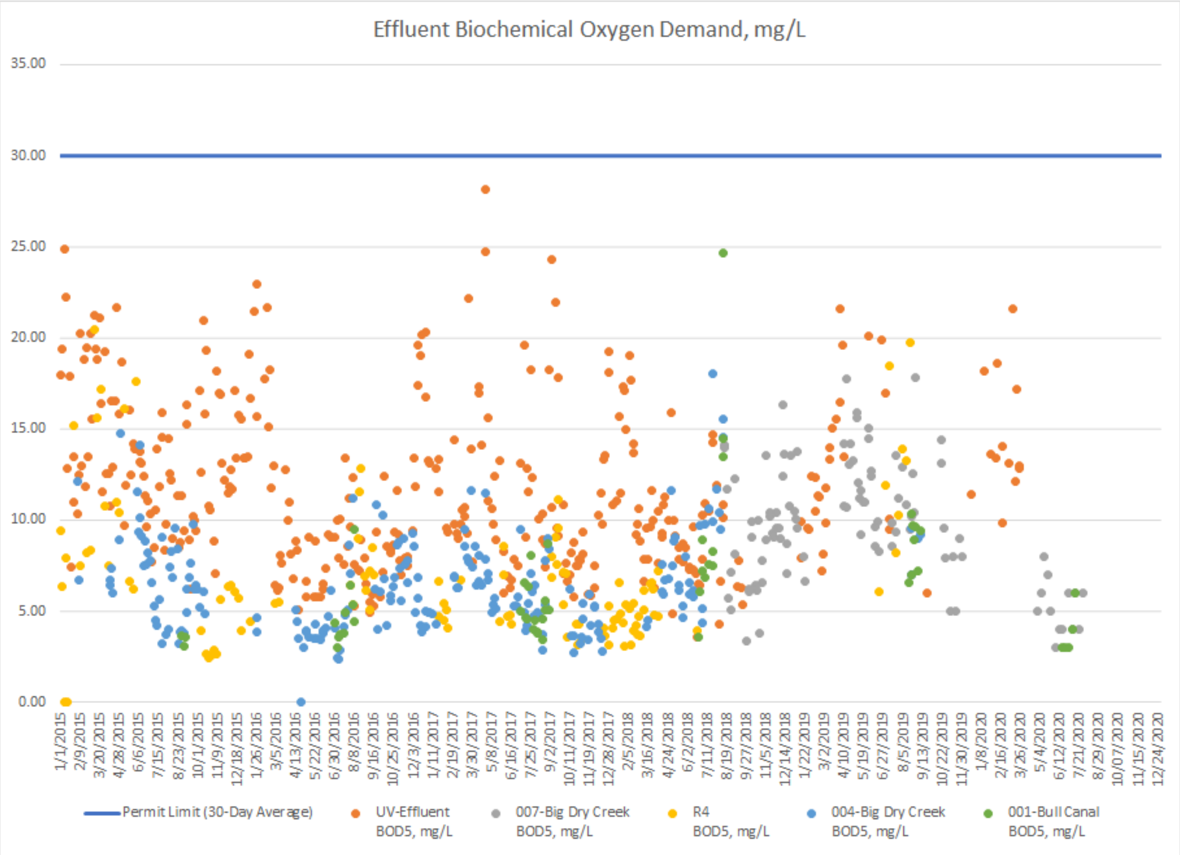


Figure 4-17: Effluent BOD_5 Concentrations

TSS concentrations have remained below 30 mg/L for samples collected prior to discharge to Big Dry Creek (Figure 4-18). A few samples collected at the discharge point from Bull Reservoir have higher TSS concentrations; most likely due to algae growth in the reservoir.

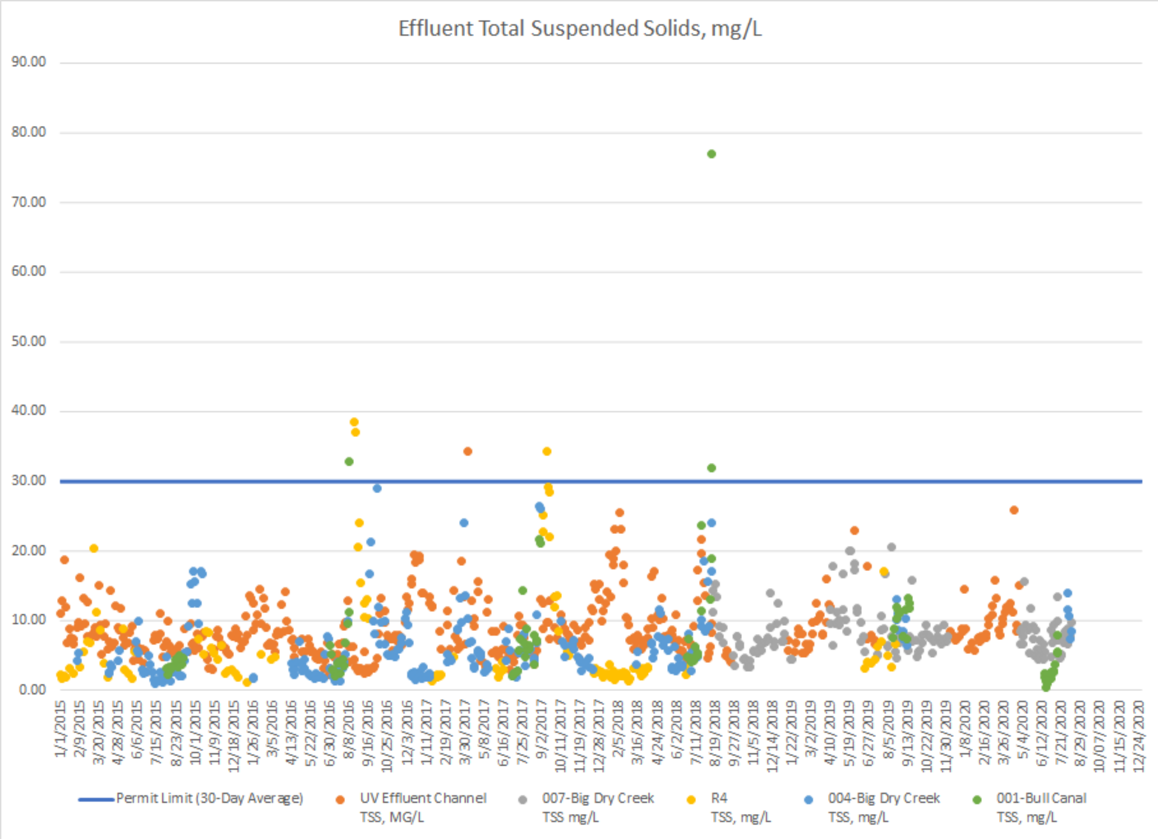


Figure 4-18: Effluent TSS Concentrations

Effluent NH_3 concentrations are monitored at three locations: 004 Big Dry Creek, 007 Big Dry Creek, and at the UV Effluent channel. Permit limits vary from month to month (Figure 4-19). All sample results for Big Dry Creek are well below the monthly discharge permit limits. NH_3 concentrations are typically less than 0.5 mg/L as N and are frequently non-detectable. Northglenn does not have a running annual average limit.

UV effluent samples are not required by the discharge permit. These samples may be either grab or composite. Periodic increases in UV effluent NH_3 are related to solids removal and decanting from the solids handling lagoons. The lagoon supernatant typically contains about 200 mg/L of NH_3-N. In 2017, a contractor decanted more than two feet of supernatant from one of the solids handling ponds over twenty-one hours without the consent of Northglenn. This resulted in a temporary spike in NH_3 of almost 12 mg/L as N going into Bull Reservoir. Discharges to Big Dry Creek remained below 0.5 mg/L as N.

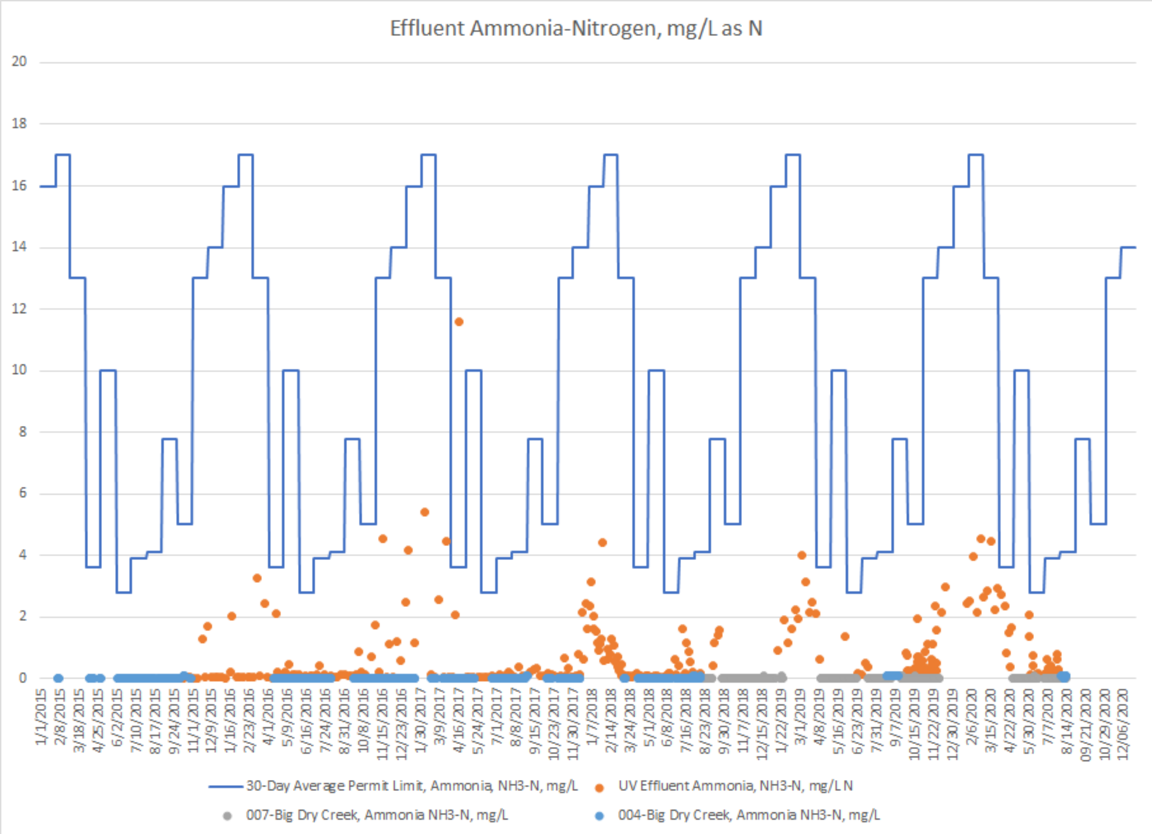


Figure 4-19: Effluent Ammonia-Nitrogen Concentrations, mg/L as N

The supernatant return from the solids handling ponds is a source of ammonia loading to the secondary treatment process and affects the effluent NH_3 concentrations measured at the UV effluent (Figure 4-19). The bacteria responsible for converting NH_3 into NO_2 and NO_3 grow slowly. Populations adapt to the average NH_3 load. Sudden increases in the NH_3 load, such as those returned in the supernatant, cannot be fully assimilated, which results in bleed through of some NH_3 to the final effluent.

Waste activated sludge is transferred from the mechanical treatment plant to the east solids handling pond. The water levels in the east and west solids handling ponds are normally maintained at slightly different levels. A valve between the ponds may be opened to equalize water levels. A pump is used to transfer excess water from the west pond back to the mechanical treatment plant for treatment. Operations staff manage the volume of supernatant returned in two ways. First, flow from the 1,200 gpm supernatant pump is divided to return 100 gpm to the mechanical plant and 1,100 gpm to the east pond. Second, an agricultural timer is used to cycle the pump on and off. During peak influent flows and loads, supernatant return is minimized. Supernatant return is increased during periods of low influent flow.

An equalization tank located at the mechanical plant would be more convenient for operations staff and could be equipped with a smaller pump to reduce energy costs. The larger supernatant pump currently in use could then be operated for short periods each day or every other day. Construction of an equalization tank for pond supernatant management could be deferred until upgrades to the solids handling processes are made since the current supernatant management strategy has been effective.

Effluent NO_3 concentrations and TIN concentrations have remained below the daily maximum discharge permit limit of 14 mg/L as N (Figure 4-20). Effluent TIN concentrations are typically below 10 mg/L as N. A three-stage BNR system operating with a 300% IMLR ratio should be able to remove up to 76% of total influent nitrogen provided the process is not carbon limited. The Northglenn WWTP consistently achieves better removal.

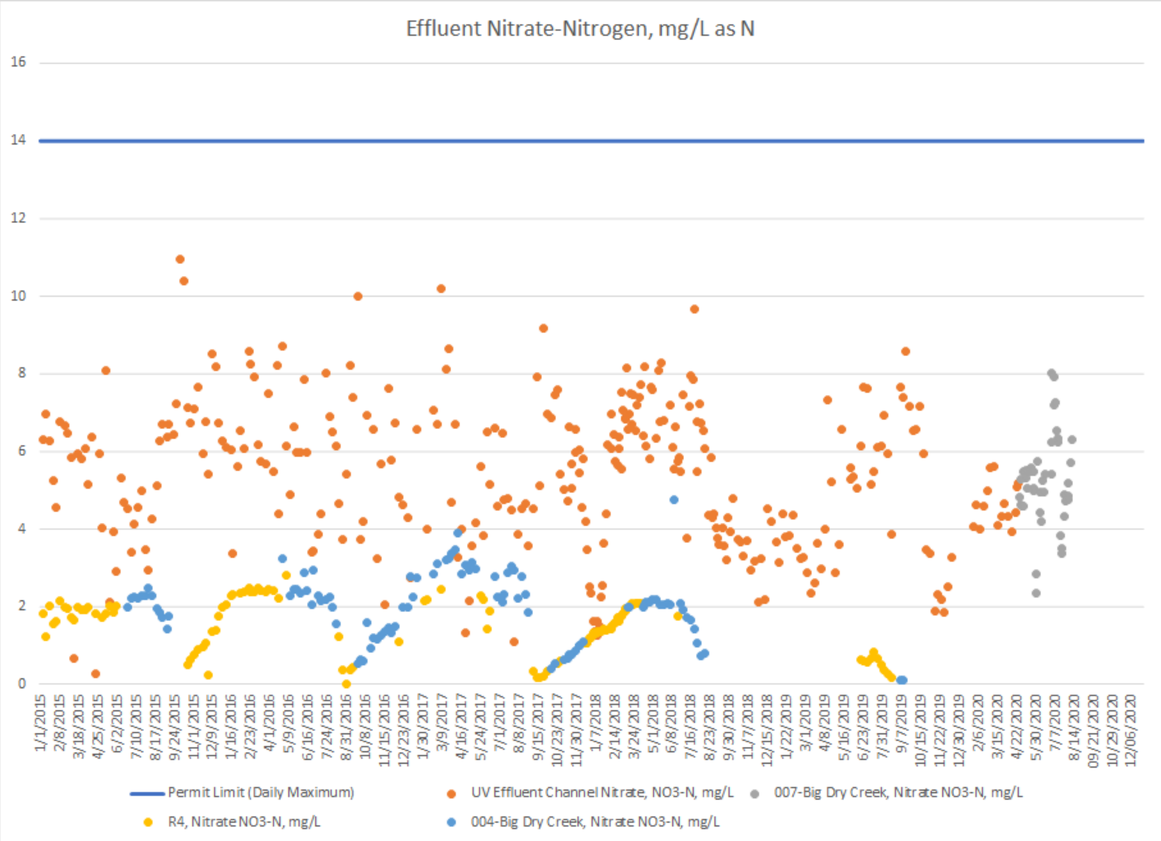


Figure 4-20: Effluent Nitrate-Nitrogen Concentrations, mg/L as N

NO_3 concentrations are consistently lower leaving Bull Reservoir than at the UV effluent channel (Figure 4-20). This is expected as NO_3 can be incorporated into algal biomass in the reservoir. Prior to April 27, 2017, the influent passed through aerated ponds prior to entering the activated sludge process. The ponds performed similarly to primary clarifiers and removed approximately 50 percent of the influent BOD. Sufficient BOD remained to support denitrification.

Northglenn began monitoring for phosphorus at the end of 2016. Effluent phosphorus concentrations discharged to Big Dry Creek have remained well below the discharge permit limit of 1 mg/L as P. Higher concentrations at the UV Effluent sampling location in 2017 and 2018 are related to excessive supernatant return from the solids handling lagoons. The lagoon supernatant typically contains about 24 mg/L of phosphorus as P. Phosphorus added to Bull Reservoir is taken up by algae growth.

Low phosphorus concentrations are attributable to both biological phosphorus removal taking place in the 3-stage BNR system and to the presence of water treatment plant residuals (alum) in the influent. Biological phosphorus removal can consistently reduce phosphorus concentrations below 1 mg/L as P, but usually not lower than about 0.7 mg/L as P. Northglenn consistently reports effluent P below 0.4 mg/L as P because some phosphorus is precipitated by the alum.

The WTP MP Update explored alternatives for upgrading residuals handling at the water treatment plant (JVA, 2020). Currently, the City sends alum sludge residuals from their WTP to the WWTP. This practice will be discontinued in the near future based on the proposed improvements in the WTP MP. Making upgrades at the water plant should improve its efficiency. However, loss of the alum residuals at the WWTP is likely to increase effluent phosphorus concentrations. A new chemical feed system may be needed at the WWTP to continue earning nutrient credits and to meet future effluent phosphorus limits.

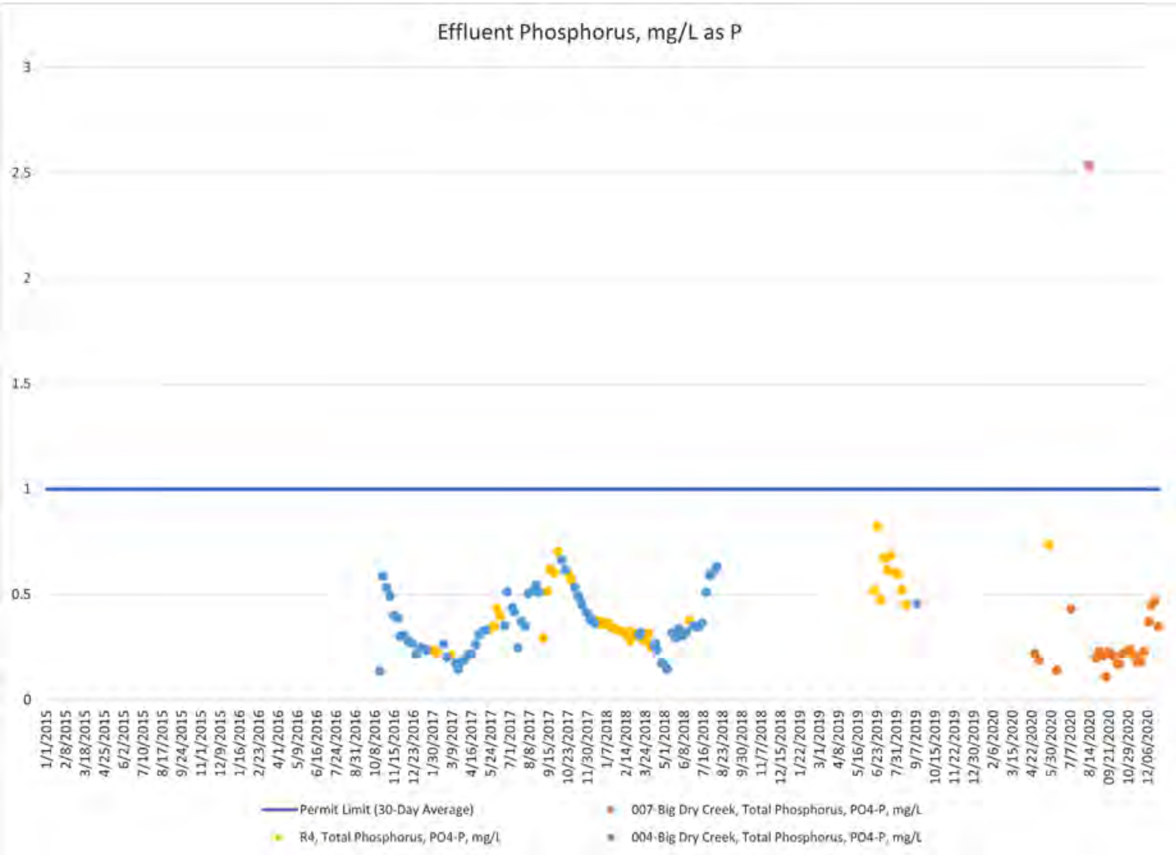


Figure 4-21: Effluent Phosphorus Concentrations, mg/L as P

Phosphorus may be precipitated using a number of different metal salts, with alum and ferric chloride being the most common. Stoichiometrically, 9.6 mg/L of alum ($Al_2(SO4)_3 \bullet 14H_2O$) are needed to precipitate 1 mg/L P. Similarly, 5.2 mg/L of ferric chloride ($FeCl_3$) are needed to precipitate 1 mg/L P. In practice, higher doses of both alum and ferric chloride are needed to overcome side reactions with water molecules and other chemical components in the wastewater. To reduce phosphorus to 1 mg/L P, a dose of 1.5 to 2 times the stoichiometric dose is typically needed. To reduce phosphorus concentrations below 0.1 mg/L, doses as high as 8 times the stoichiometric dose may be needed. Jar testing is recommended to determine actual doses. Precipitation effectiveness is determined by the dose and the chemical addition point. Metal salts can only precipitate ortho-phosphate. Raw domestic wastewater typically contains around 50% ortho-phosphate, 35% poly-phosphate, and 15% organically bound phosphorus. Therefore, adding metal salts at the influent can remove a maximum of about 50% of the soluble influent phosphorus.

The ferric chloride added to the facility headworks for odor control may also be used to precipitate phosphorus; however, higher dosages will be required. The final effluent iron concentration will require careful monitoring to ensure the new permit limit for dissolved iron is not exceeded. Increasing the ferric chloride dosage at headworks may also have the unintended side effect of increasing MLSS in the activated sludge basins.

E. coli limits vary by discharge point. For Bull Canal and the Thompson Ditch, the 30-day geometric mean for *E. coli* must be below 2000 #/100 mL and the 7-day geometric mean must be below 4000 #/100 mL. For Big Dry Creek, the 30-day and 7-day geometric means for *E. coli* are 205 and 410 #/100 mL, respectively. Limits for *E. coli* are easily met at all monitoring locations.

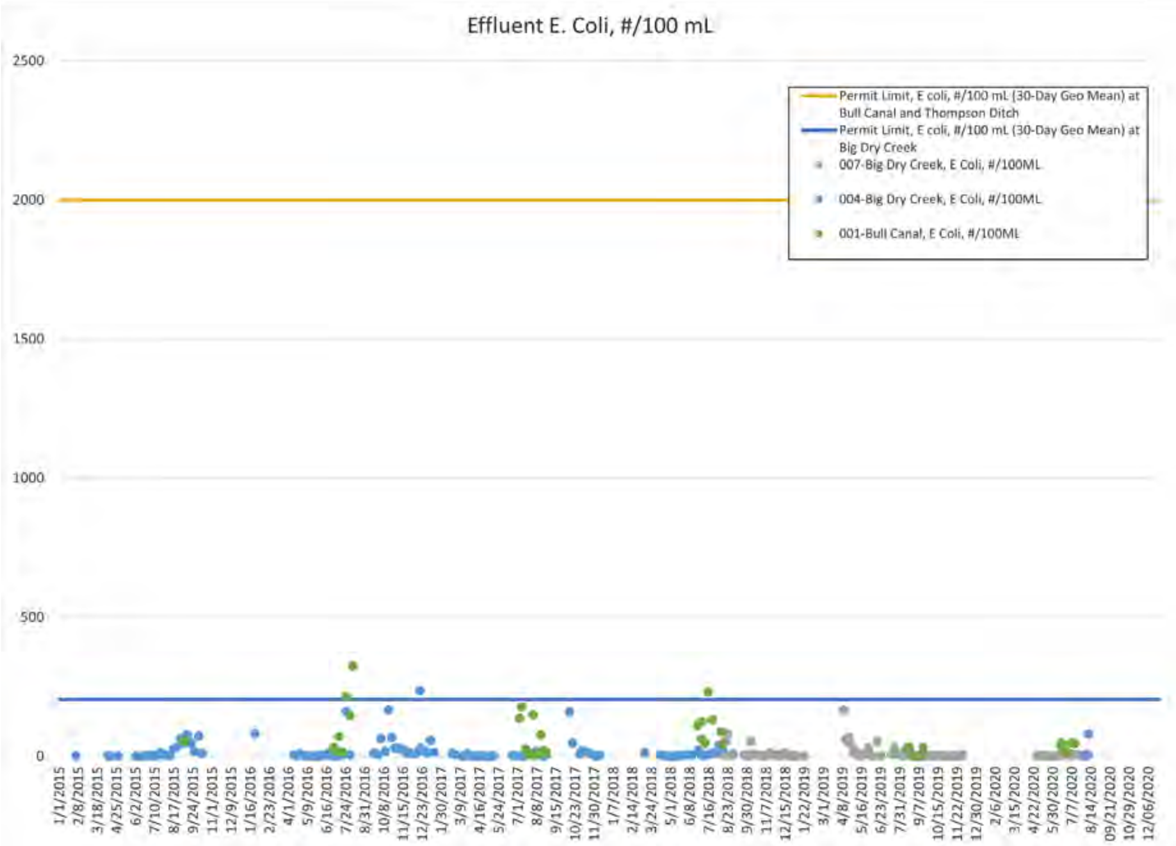


Figure 4-22 Effluent *E. coli* Concentrations, count/100 ml

The following table summarizes the effluent data reported on the Discharge Monitoring Reports (DMRs) for the previous permit term, from November 2013 through October 2018 (CDPHE, 2019).

Table 4-26: Summary of DMR Data for Outfalls 001A, 004A, and 007A

Parameter	# of Samples or Reporting Periods	Reported Average Concentrations Avg/Min/Max	Reported Maximum Concentrations Avg/Min/Max	Previous Avg/Max Permit Limit	Number of Limit Excursions
Effluent Flow (mgd)	45	1.2/0.05/7.5	2/0.17/12	6.5/NA	1
Temperature (°C)	45	15/3.9/24.6	16.7/4.3/26.4	NA/NA	
pH (su)	46	7.7/4.4/8.6	8.5/7.3/9	NA/NA	1
E. coli (#/100 ml)	46	34/1/326	76/1/326	205/410	
Total Ammonia as N (mg/L)	46	0.92/<0.2/2.9	1.2/<0.2/4.1	NA/NA	
January	2	0.96/0.74/1.2	1/0.74/1.3	3.7/4	
February	3	1.3/1.1/1.7	1.5/1.2/1.9	4.7/6.2	
March	3	1.4/0.92/1.8	1.6/1.2/1.9	2.6/4.4	
April	4	0.93/0.7/1.4	1.4/0.84/2.4	3.7/5.8	
May	4	1.6/0.68/2.9	1.8/0.77/3	3.5/6.1	
June	4	1.2/0.6/2.5	1.3/0.76/2.7	4/5.2	
July	5	0.64/<1/1.5	0.96/<1/2.3	3.4/5.7	
August	4	0.7/0.55/0.98	1.1/0.61/1.7	3.3/6.1	
September	4	0.57/<1/1.3	1.4/<1/4.1	3.3/8	
October	4	0.73/<0.2/1.1	1.3/0.3/2.2	3.4/6.3	
November	4	0.94/<1/1.7	1/<1/1.8	4.6/7.9	
December	3	0.85/<1/1.5	0.89/<1/1.5	5.9/7.4	
BOD_5, (mg/L)	46	7.1/3.6/18	8.7/3.9/25	NA/NA	
BOD_5, influent (mg/L)	60	314/3.3/5765	346/3.4/6050	NA/NA	
BOD_5, influent (lbs/day)	60	6,106/4,238/7,383	7823/5289/62240	NA/NA	
BOD_5, effluent (mg/L)	46	7.1/3.6/18	8.7/3.9/25	30/45	
BOD_5 (% removal)	46	96/90/99	NA/NA/NA	85/NA	
TSS (mg/L)	46	7.6/1.8/35	11/1.8/45	NA/NA	
TSS, influent (mg/L)	60	257/200/357	287/218/399	NA/NA	
TSS, effluent (mg/L)	46	7.6/1.8/35	11/1.8/45	30/45	1
TSS (% removal)	46	96/62/99	NA/NA/NA	85/NA	2
Oil and Grease (mg/L)	60	NV/NV/NV	NV/NV/NV	NA/10	
Copper, Dis (µg/L)	46	2.9/0.43/12	3.9/0.84/18	25.6/43	
Cyanide, Tot (µg/L)	46	0.11/<5/5.0	0.22/<5/5.0	NA/NA	
Selenium, Dis (µg/L) Nov-March	15	1.5/<0.8/3	1.3/<0.8/2	15/19.1	

Parameter	# of Samples or Reporting Periods	Reported Average Concentrations Avg/Min/Max	Reported Maximum Concentrations Avg/Min/Max	Previous Avg/Max Permit Limit	Number of Limit Excursions
Selenium, Dis (µg/L) April - Oct	30	1.4/<0.8/4.1	1.6/<0.8/4.1	7.6/18.4	
WET, chronic					
ceriodaphnia lethality, Stat Diff	20		100/100/100	NA	
ceriodaphnia lethality, IC25	20		100/100/100	NA	
pimephales toxicity, Stat Diff	20		98/81/100	Report	
pimephales toxicity, IC25	20		96/13/100	Report	
ceriodaphnia toxicity, Stat Diff	20		89/13/100	Report	
ceriodaphnia toxicity, IC25	20		88/12/100	Report	

Note: **Geometric Mean, NA = Not Applicable, NV = no visible sheen

Additional monitoring data were reviewed during permit renewal and include nutrient data that are relevant to future planning (**Table 4-27**).

Table 4-27: Summary of Additional Treatment Data (2017-2018)

Parameter	# of Samples or Reporting Periods	Reported Average Concentrations Avg/Min/Max	Reported Maximum Concentrations Avg/Min/Max
Nitrate as N (mg/L)	10	6.5/1.6/13	6.4/1.6/13
Nitrate+Nitrite as N (mg/L)	59	5.2/<0.01/12	5.2/<0.1/12
Total Ammonia as N (mg/L)	58	7.2/0.37/15	7.1/0.37/15
January	5	1.2/0.51/2.4	1.2/0.51/2.4
February	5	1.1/0.14/1.9	1.8/0.28/5.4
March	5	1.8/0.91/4.1	1.5/0.7/8
April	5	0.92/<1/2	1.3/<1/14
May	5	1.6/0.68/2.9	1.4/0.28/3
June	5	0.88/<0.1/2.5	1.1/<0.1/3
July	4	0.76/0.12/1.5	0.93/<0.1/2.3
August	4	0.61/0.4/1	0.64/<0.1/1.7
September	5	0.56/<1/1.4	0.72/<1/4.1
October	4	0.63/0.11/1.1	0.94/0.13/1.3
November	5	0.84/<1/1.7	1/<1/1.8
December	5	0.91/<1/1.8	1.1/<1/2.7

Parameter	# of Samples or Reporting Periods	Reported Average Concentrations Avg/Min/Max	Reported Maximum Concentrations Avg/Min/Max
Arsenic, TR (µg/L)	10	0.93/<0.6/3.7	0.91/<0.6/6.8
Beryllium, TR (µg/L)	10	0.04/<0.05/0.4	0.033/<0.05/0.4
Cadmium, TR (µg/L)	10	0/<0.1/0	0/<0.1/0
Chromium, TR (µg/L)	10	1.1/<1.5/6.1	0.91/<1.5/6.1
Chromium+6, TR (µg/L)	11	0/<10/0	0/<10/0
Iron, TR (µg/L)	16	76/20/239	83/20/416
Lead, TR (µg/L)	10	0.54/<5/1.3	0.54/<5/2.2
Manganese, TR (µg/L)	10	181/101/288	182/80/363
Molybdenum, TR (µg/L)	10	3.2/<20/5.3	2.8/<20/5.3
Mercury, Tot (µg/L)	15	0.0012/<0.001/0.0091	0.0011/<0.001/0.0091
Nickel, TR (µg/L)	10	3.3/<8/8.1	3.2/<8/8.1
Silver, TR (µg/L)	10	0.05/<0.1/0.4	0.063/<0.1/0.4
Zinc, TR (µg/L)	10	44/11/69	45/11/96
Phosphorus (mg/L)	36	0.45/0.1/1.1	NA/NA/NA
Sulfate (mg/L)	29	286/211/345	286/211/345

The permit fact sheet noted that permit violations were noted in the preceding tables. (CDPHE, 2019). The fact sheet went on to provide context to the instances of occurrence:

"One effluent violation for the daily minimum pH limitation was reported on the August 2018 DMR. The reason for this violation was not following the proper steps on how to adjust the acid for less flow. Staff regulated acid to achieve a pH between 6.5 s.u. and 9.0. Since then, the POTW has reviewed and updated their SOP for Big Dry Creek flow change. The POTW also did hands on training following the SOP with all of their operators. In addition, they have ordered a meter and pH probe to allow 24 hr monitoring and will be budgeting to integrate the pH into the PLC so that acid could be regulated based off of pH. With the PLC integration the POTW will be capable of adding alarms to be notified immediately when the pH falls out of the permitted range and will shut of the discharge pump as well.

One effluent violation for the 30-day average TSS limitation was reported on the August 2018 DMR. The reason for this violation was due to the algae bloom that happened at the end of the irrigation discharge season. Next season on top of running TSS twice a week the POTW monitored turbidity on a daily basis to help evaluate the quality of the water to help them determine if they need to shut off.

Two effluent violations for the percent removal TSS limitation were reported on the July 2017 DMR. The reason for this violation was due to a force main break that happened on July 25, 2017 and requiring the diversion of Lift Station A flow to Metro Wastewater. This meant that the facility was treating only 0.14 mgd. The effluent TSS concentration discharged was less than the effluent concentration limitations.

One effluent violation for the 30-day average effluent flow limitation was reported on the June 2015 DMR. The reason for this violation was due to the need to draw down Bull reservoir during an exceptionally wet spring. The steps that have been taken to abate the non-compliance are as follows; flow through Outfall 004A was reduced to 0.8 mgd on 6/24/2015, future start dates of reservoir draw-downs will occur earlier to reduce the water level, and they will monitor the total flow releases more closely to avoid exceeding the 30-day average limitation."

Note that Northglenn has completed the installation of pH monitoring equipment and integration with alarm notification as part of the new pump project. Additionally, an operator has completed a study on TSS and NTU correlation for Bull Reservoir discharge. Over the course of several weekends, several grab samples were taken and a series of TSS and NTU analyses were completed. The lab data were incorporated into an Excel spreadsheet to review the correlation between the parameters. The results showed strong correlation and Northglenn may use the results to quickly read NTU and estimate TSS. Because particle sizes may change, a previously calculated correlation may become invalid, and it is recommended that the correlation be re-verified periodically.

4.3.3 Existing Biosolids Management Program

WAS is stabilized in solids handling lagoons to produce Class B biosolids. The mass of solids removed from the lagoons varies from one year to the next due to budgetary constraints and/or weather restrictions on removal and land application. Northglenn contracts with Veris Environmental (formerly Parker Ag.) for solids removal and land application. The finished biosolids are land applied to a combination of Northglenn-owned property adjacent to the WWTP and land owned by Veris Environmental (**Table 4-28**).

Biosolids quantity and quality reported in **Tables 4-28** and **4-29** include two columns for year 2016. The primary ponds were decommissioned in 2016. Column 2016A is for biosolids removed from the solids handling ponds and column 2016B is for biosolids removed from the primary ponds during decommissioning.

Table 4-28 Quantity of Biosolids Land Applied from 2015 - 2020

	2015	2016 A	2016 B	2017	2018	2019	2020
Biosolids Produced, dt	558.45	936.75	627.56	652.159	699.71	524.42	954.72
Biosolids Applied On-Site, dt	0	518	0	411.75	0	215	689
Biosolids Applied Off-Site, dt	558.45	418.75	627.56	240.41	669.71	309.42	265.72

Note: dt = dry metric tons

Table 4-29 Biosolids Quality

Parameter	2015	2016A	2016B	2017	2018	2019	2020
Metric Tons Produced	558.5	936.8	627.6	652.6	699.7	524.4	954.7
Total Solids, %	4.17	3.76	4.46	0.9	4.26	4.37	4.94
Total Arsenic, mg/kg	5.9	0	1.4	4.7	<0.5	<0.5	<0.5
Total Cadmium, mg/kg	3.08	0	ND	2.2	<0.5	<0.5	<0.5
Total Copper, mg/kg	677.8	619	660	630	576	503	543.3
Total Lead, mg/kg	56.7	18.9	73.9	39.1	<5.0	<0.5	<5.0
Total Mercury, mg/kg	2.04	2.39	1.86	1	6.25	1.78	<0.05

Parameter	2015	2016A	2016B	2017	2018	2019	2020
Total Molybdenum, mg/kg	20.68	3.16	16.6	15.1	<1.0	<1.0	29.2
Total Nickel, mg/kg	16	5.4	23.4	18.3	<1.0	<1.0	37.6
Total Selenium, mg/kg	17.2	18	14.6	14.7	26.7	20.2	24.4
Total Zinc, mg/kg	1078	1004	1018	982	836	774.4	796.7
Total Kjeldahl Nitrogen, %	3.47	5.56	2.09	6.62	6.52	5.90	6.48
Ammonium Nitrogen, %	1.04	1.00	0.34	3.78	1.67	0.99	1.39
Total Phosphorus, %	3.45	3.44	2.16	3.46	3.84	3.43	3.66
Total Potassium, %	0.52	0.34	0.21	1.31	0.45	0.45	0.45

Metals concentrations for Northglenn's biosolids meet the definition for exceptional quality biosolids in the 503 regulations. Exceptional quality biosolids have metals concentrations lower than those shown in **Table 4-30**.

Table 4-30 Pollutant Concentration Limits for Exceptional Quality Biosolids

Parameter	Concentration Limit, mg/kg
Total Arsenic	41
Total Cadmium	39
Total Chromium	1200
Total Copper	1500
Total Lead	300
Total Mercury	17
Total Nickel	420
Total Selenium	36
Total Zinc	2,800

Table 4-31 compares influent BOD_5 loading to the mass of biosolids land applied each year. The pounds of biosolids applied per pound of BOD_5 received varies from year to year because the mass of biosolids removed from the lagoons also varies from year to year.

Table 4-31 Biosolids Applied per Pound of BOD_5 Received

Parameter	2017	2018	2019	2020
Total Influent Pounds of BOD per Year	2,089,641	2,315,845	2,301,468	2,125,535
Total Pounds Biosolids Removed	1,437,358	1,542,161	1,155,822	2,104,203
lbs Biosolids / lb BOD	0.69	0.67	0.50	0.99

4.3.4 Process Control Evaluation

4.3.4.1 Nitrification Requirements

The Northglenn WWTP is required to remove NH_3 and NO_3 to meet a TIN limit of 14 mg/L as N. Monthly NH_3 permit limits vary from month to month with the lowest monthly average limit near 2 mg/L NH_3-N. In practice, biological nitrification is an all or nothing proposition. Either there are sufficient nitrifying bacteria in the process to provide complete nitrification or there are not. Partial nitrification is not a realistic goal of process control. Operators attempting to maintain partial nitrification find that their activated sludge processes can rapidly lapse into complete nitrification or no nitrification taking place.

The growth rate of nitrifying bacteria is closely linked to water temperature. For every 10 degrees Celsius decrease in water temperature, the growth rate of the nitrifiers is reduced by fifty percent. To maintain stable nitrification, the aerobic solids residence time (SRT) must be longer than the time required for the nitrifying bacteria to reproduce. The theoretical minimum aerobic SRTs to prevent washout are about 2 days at 20 degrees Celsius and about 4 days at 10 degrees Celsius. In practice, most facilities operate at 2 to 3 times the minimum aerobic SRTs to avoid accidental loss of nitrification.

The filamentous bacteria *Microthrix parvicella* causes foaming and sludge settleability issues in activated sludge systems. Work by Eric Lynn and Bill Martin suggests that as water temperatures decrease, the growth rates of the nitrifying bacteria and *M. parvicella* get closer together until they eventually converge at about 10 degrees Celsius. Most activated sludge systems in temperate climates that are required to nitrify experience some foaming due to *M. parvicella* during their cold weather months. Northglenn is no exception. Operators can mitigate foaming and bulking events by maintaining an SRT long enough to retain nitrifying bacteria but short enough to wash out *M. parvicella*. This strategy is effective unless water temperatures drop below about 12 degrees Celsius in the activated sludge basins. Foam trapping, such as occurs at Northglenn, can allow M. parvicella to accumulate in the basins and even seed *M. parvicella* back into the liquid mixed liquor suspended solids (MLSS). The foam has an effective SRT that is much longer than the MLSS liquid phase SRT.

M. parvicella is best controlled through SRT management. Failing that, surface wasting of foam or application of polyaluminum chloride (PAX) to the process can mitigate the impacts of foaming episodes. Northglenn has used vac-trucks to remove accumulated foam from the surface of the activated sludge basins in the past. Removed foam should not be introduced into the treatment process and should be disposed of at a landfill or similar location.

4.3.4.2 SRT Control at Northglenn

Northglenn's activated sludge process is operated with two of the three treatment trains in service with the third train providing backup capacity. Operators are using SRT and aerobic SRT control. The aerobic SRT is approximately 64 percent of the total SRT at the Northglenn WWTP. In-basin water temperatures are recorded daily. Between January 2015 and December 20, 2020, daily in-basin water temperatures varied between 12.1 °C and 27.3 °C. At the end of each week, operations staff evaluate the average in-basin water temperature against the theoretical minimum aerobic SRT needed to maintain stable nitrification. The SRT is then increased or decreased to track with in-basin water temperature. This method of process control ensures gradual changes to SRT throughout the seasons. **Figure 4-23** illustrates the correlation between water temperature and SRT.

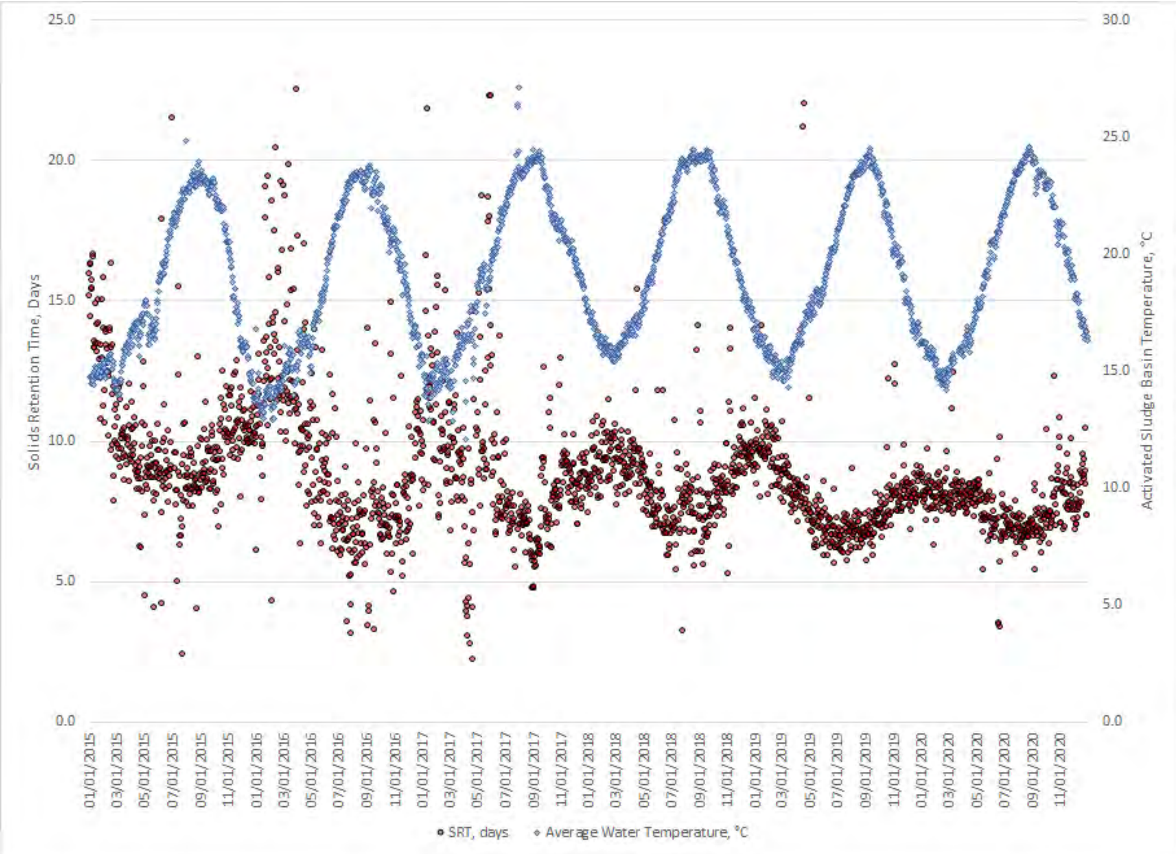


Figure 4-23: Total SRT versus Activated Sludge Basin Water Temperature

Target SRTs have been tightly controlled over the last three years. Additionally, the chief plant operator has gradually reduced target SRTs to prevent foam formation and sludge bulking. **Table 4-32** lists the monthly average total SRTs from 2015 through 2020. Monthly average SRTs have ranged from 7.4 days in August up to 10.8 days in January and February. This translates to aerobic SRTs between 4.7 days and 6.9 days, respectively. These SRTs are close to the theoretical washout aerobic SRTs of 2 and 4 days for the nitrifying bacteria. As noted previously, the "safe" operating range for aerobic SRT is at least twice and preferably three times the theoretical minimum washout aerobic SRT. On a few occasions, Northglenn has dipped temporarily below this threshold and observed an immediate increase in effluent NH_3 concentrations. When effluent NH_3 concentrations increase, effluent can be immediately diverted into Bull Reservoir to avoid a discharge permit violation. Dilution in Bull Reservoir brings effluent NH_3 concentrations well below permit limits. Addition of a TSS probe in the WAS stream was recommended to assist with SRT process control and has been installed.

Table 4-32 Monthly Average Total SRT

Month	2015	2016	2017	2018	2019	2020	Average	StdDev
January	14.0	11.9	11.8	9.2	9.8	8.2	10.8	2.15
February	12.2	14.4	10.7	9.7	9.4	8.3	10.8	2.22
March	10.1	13.2	9.6	9.2	8.5	8.3	9.8	1.81
April	9.3	10.6	8.8	9.5	8.9	8.0	9.2	0.87
May	8.8	9.4	12.9	8.1	7.3	7.7	9.0	2.05
June	9.2	8.5	9.0	7.6	6.8	6.9	8.0	1.08
July	12.4	6.6	11.2	7.4	6.7	7.0	8.6	2.56

Month	2015	2016	2017	2018	2019	2020	Average	StdDev
August	8.6	7.0	7.0	8.0	6.9	6.8	7.4	0.73
September	9.3	8.0	7.0	7.5	6.9	7.0	7.6	0.90
October	9.7	7.7	8.0	8.2	7.7	8.0	8.2	0.74
November	10.7	7.4	9.1	9.2	8.4	8.2	8.8	1.10
December	10.5	9.2	8.4	9.7	8.2	8.2	9.0	0.94

Figure 4-24 illustrates the relationship between SRT and sludge volume index (SVI) at the Northglenn WWTP. SVI is a measure of sludge settleability and compaction characteristics. Higher SVIs generally correlate to slower settling and poorer compaction. SVI is expressed in milliliters per gram (mL/g) or the volume one gram of settled sludge occupies in a Mallory type settleometer.

Northglenn has experienced two bulking events in the past five years where the SVI rose above 200 mL/g. In February 2017, the SRT temporarily increased to over 15 days. This event may be related to decommissioning of the primary lagoons. A second bulking event occurred in February 2019. The cause of this event is unclear as the liquid phase SRT remained below 10 days but may be related to foam accumulation in the basins. Overall, the SVI has remained below 160 mL/g for 95% of all samples collected.

Table 4-33 summarizes process control data collected between May 2015 and December 2020. MLSS concentrations fluctuate with the target SRT and are normally between 2120 mg/L (10th percentile) and 3270 mg/L (90th percentile). This is in agreement with Northglenn's consistent influent BOD_5 loading rates and narrow target range for SRT. In November 2016, MLSS concentrations spiked to over 4000 mg/L for 16 days straight. This appears to be related to decommissioning of the primary ponds.

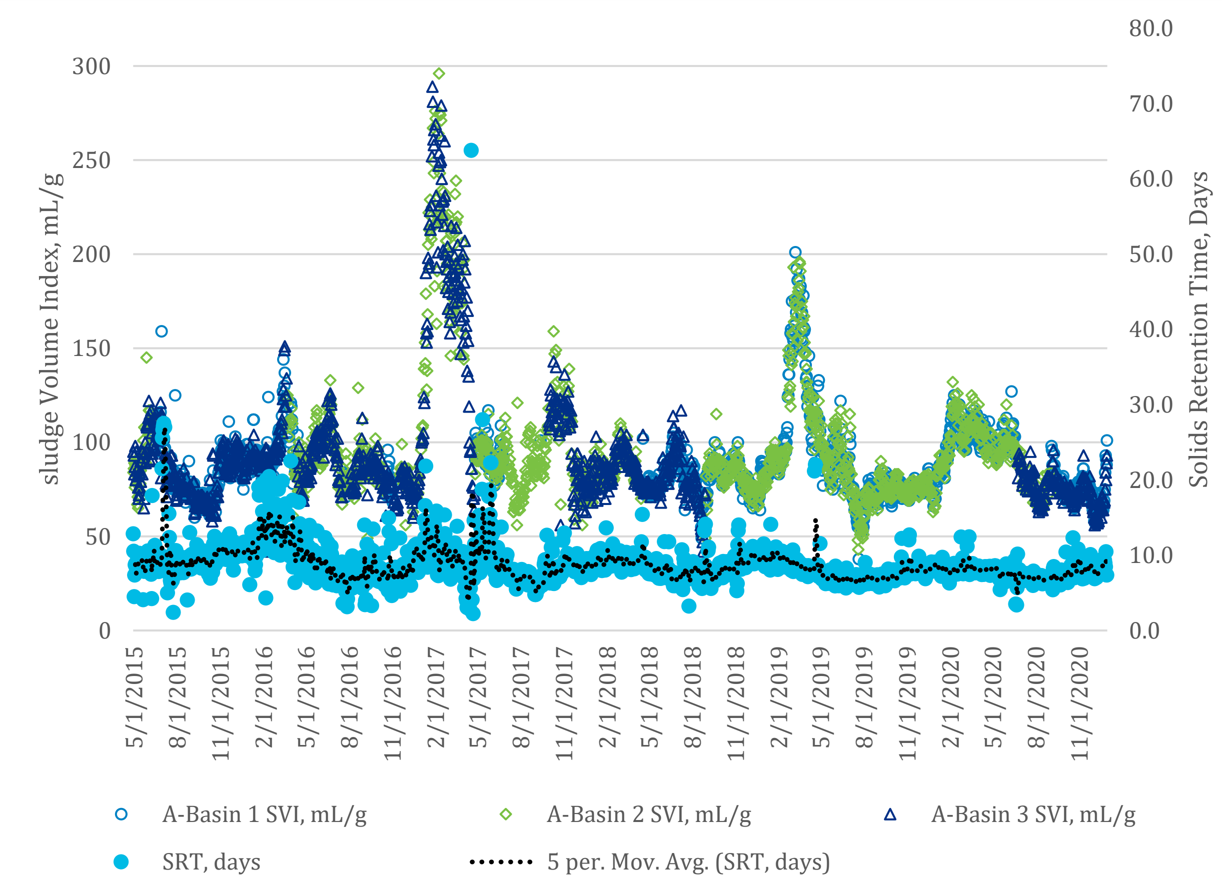


Figure 4-24: Sludge Volume Index, mL/g

Table 4-33 Process Control Data from May 1, 2015 through December 31, 2020

	MLSS, mg/L	SRT, days	SRTaerobic, days	SVI, mL/g
Average	2751.8	8.7	5.4	93
Minimum	1335	2.2	1.4	38
Maximum	4755	63.8	39.8	296
i	3270	11.1	7.0	159
99th Percentile	3400	12.7	7.9	226

4.3.4.3 Secondary Clarifier Capacity

Secondary clarifier capacity was evaluated using State Point Analysis (SPA) modeling. SPA is a visual representation of clarifier capacity. The point on the SPA graph where the overflow and underflow lines intersect is referred to as the state point. The bell curve is created from the SVI. When the state point is inside (underneath) the sludge settleability curve, there is adequate clarifier capacity available to settle the incoming MLSS.

For the analysis, the following values were utilized: two clarifiers in service, MLSS = 3500 mg/L, SVI = 250 mL/g, and RAS = 70% of influent flow. Northglenn has only experienced one poor settleability event of this magnitude once over the last five years. Results for several different SPA models are shown in **Figure 4-25**. The Ozinsky-Ekama model predicts adequate capacity with two clarifiers in service while the other models do not. Reducing the MLSS concentration to 3000 mg/L brings the state point within three of the SPA models.

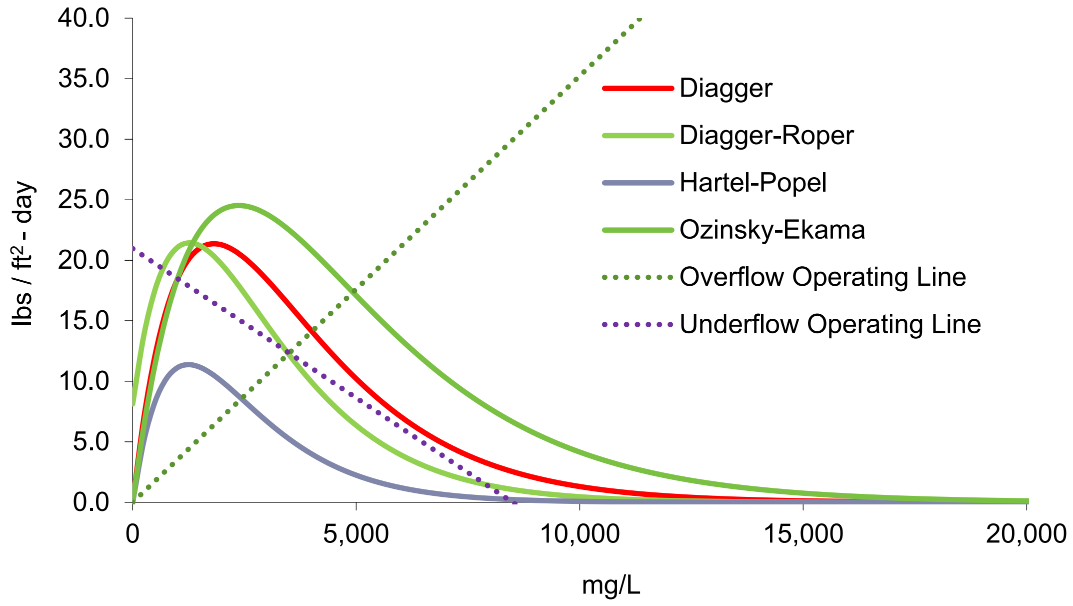


Figure 4-25 State Point Analysis at Influent Flow of 4.2 mgd

In the event of an extreme bulking event, operations staff could increase wasting to bring the MLSS concentration below 3000 mg/L, place a third clarifier into service to increase clarifier surface area, or place a third activated sludge basin into service to decrease the MLSS concentration.

4.3.4.4 Effluent Ammonia and Nitrate Concentrations

The process is capable of meeting effluent NH_3-N concentrations below 2 mg/L NH_3-N and NO_3–N concentrations below 8 mg/L NO_3-N. Occasional increases in effluent NH_3 are a result of keeping SRTs as low as possible to control M. parvicella.

The IMLR pumps are capable of returning 300% of influent flow. Further increases in the IMLR are unlikely to further lower effluent NO_3 -N concentrations without adding a supplemental carbon source. Addition of NO_3 analyzers in each anoxic zone is recommended to enhance process control. These could eventually be tied into control of the IMLR pumps.

4.3.5 Condition Assessment of Existing Treatment System

An assessment team visited the WWTP, inspected the major equipment, reviewed documentation, and interviewed O&M personnel regarding the O&M history of the major facilities and assets. The purpose of this effort was to provide Northglenn with a current assessment of system condition that can be used to plan system repair and refurbishments. The asset data in this section can also be reviewed in the future to develop a Computerized Maintenance Management System (CMMS) that can be used to schedule and track maintenance activities.

4.3.5.1 Overall Conditions Assessment

A condition assessment grade was assigned to each major piece of equipment according to the categories listed in **Table 4-34**. **Appendix F** includes the complete equipment list with the assigned grades, but in general, operating without screenings and grit removal has resulted in most of the valves and gates requiring replacement. Operations staff noted the difficulty with isolating basins for maintenance due to degraded seals on valves. The clarifier mechanisms also require replacement.

Table 4-34: System Condition Assessment Categories

Grade	Condition	Description
1	Very Good	Sound physical condition. Meets current needs. Operable and well maintained. Asset expected to perform adequately with routine Maintenance for 10 years or more. No work required
2	Good	Acceptable physical condition. Shows minor wear that has minimal impact on performance. Minimal short-term failure risk. Potential for deterioration or impaired performance over next 5-10 years. Minor work (If any) required.
3	Fair	Functionally sound but showing wear and diminished performance. Moderate short-term failure risk. Potential for further deterioration and diminished performance within next 5 years. Renewal or major component replacement expected within next 5 years. Minor work required but asset is serviceable.
4	Poor	Asset functions but requires high level of maintenance to remain operable. High risk of short-term failure. Likely to have significant deterioration in performance within next 2 years. Renewal or replacement expected within next 2 years. Substantial work required. asset barely serviceable
5	Replace	Asset failed or failure is imminent. Excessive maintenance required. No further service life expectancy. Significant health and safety hazard. Major work or replacement is urgent.

The capital expenditure associated with replacing each piece of "Grade 5" equipment was estimated. The construction cost for each item is provided as a planning-level opinion of probable construction costs (OPCC) including construction and implementation, direct and indirect costs, as well as a construction contingency and escalation to the assumed mid-point of construction. No program costs (i.e. project contingency and engineering and implementation) are included for the conditions assessment work since these items will be replaced in-kind and the scope is well-defined.

Specifically, OPCCs presented for the conditions assessment include:

- Direct costs:
  - Major equipment, equipment skids, and tank purchase costs based on information obtained from selected equipment vendors
  - Materials (e.g. piping, concrete) purchase costs
  - Mobilization, site work, and minor building improvements
  - Labor costs
- Indirect costs:
  - Building permits, bonding, insurance, sales tax, builder's risk insurance
  - Contractor's field general conditions, overhead and profit
- Construction contingency: 30% of the sum of direct and indirect costs
- Escalation of the construction total cost (including direct costs, indirect costs, and construction contingency) to October 2022 costs at 3% per year (1.5 years)

Table 4-35: Opinion of Probable Construction Cost to Replace Equipment

Description	Quantity	Total Amount
Aeration Basins		
Water Treatment Equipment		
9.3 HP Submersible Mixers	3	$260,000
Water Treatment Equipment		$260,000
Gates		
18" Sluice Gate	2	$62,000
30" Sluice Gate	3	$130,000
36" Sluice Gate	3	$150,000
42" Sluice Gate	5	$300,000
54" Sluice Gate	4	$330,000
Gates		$970,000
Valves		
12" Butterfly Valves	3	$15,000
14" Butterfly Valves	6	$45,000
Valves		$60,000
Aeration Basins Subtotal		**$1,300,000**
Process Basement - Secondary Treatment		
16" MLR-BFV	6	52,000
14" MLR-PV	4	100,000
12" MLR-PV	4	72,000
12" MLR-SCV	4	47,000
16" RAS-BFV	2	17,000
14" RAS-BFV	2	15,000
10" RAS-PV	2	32,000
12" RAS-PV	6	110,000
12" RAS-SCV	3	35,000
6" WAS-PV	6	74,000
4" WAS-DCV	2	3,800
6" WAS-DCV	2	6,100
8" EFF-BFV	3	8,000
10" EFF-BFV	3	14,000
8" EFF-SCV	3	15,000
Process Basement - Secondary Treatment Subtotal		**$600,000**
Secondary Clarifiers		
Clarifier Mechanism	3	$2,000,000
Secondary Clarifiers Subtotal		**$2,000,000**
UV Building		
36" Slide Gate	3	$150,000
UV Building Subtotal		**$150,000**
Total Opinion of Probable Construction Cost		**$4,100,000**

4.3.5.2 Sulfuric Acid Storage and Feed System

Sulfuric acid is fed at the Bull Canal pump station to lower pH prior to discharge into Bull Canal, Big Dry Creek, or the Thompson Ditch. Algae growth in the reservoir gradually increases reservoir pH throughout the summer. Discharges are timed to minimize sulfuric acid addition. Sulfuric acid is stored in three 1,500-gallon tanks and fed via a Watson Marlow positive displacement (peristaltic) pump.

Operations staff are concerned with the general safety and ease of operations of the system. Bulk deliveries are delivered through a C-PVC pipe. Chemical feed pump tubing occasionally fails, which creates a serious safety condition, as sulfuric acid is sprayed onto nearby equipment. Presently, the peristaltic pump is housed in a chemical proof box. The following pumps should be considered to replace the Watson Marlow feed equipment:

- Screw Centrifugal
- Magnetic Drive
- Progressive Cavity

Containment for the bulk storage should be evaluated to confirm that containment of 110% of the largest chemical storage container can be achieved. A preliminary code review was performed for classifying the sulfuric acid room and what may be required for compliance. These improvements would change the complexity of designing and constructing a new chemical feed facility to improve safety and operations. Included below are a preliminary list of safety features and installation requirements for a new chemical feed area, per the International Fire Code (IFC) Hazardous codes E102.1.11, E102.1.7, E102.2.2, 102.2.3:

- Hazard identification signage (National Fire Protection Association (NFPA) placards etc) per NFPA 704 is required.
- Separation from incompatible materials (e.g. acids, flammables, etc) is required.
- Secondary containment is required (must be sized to contain the largest tank + 2 hr fire sprinkler flow).
- Leak detection system (remote monitoring required).
- Fire sprinklers are required. Remote monitoring required.
- Continuous ventilation with emergency shutoff is required.
- Standby or Emergency power required.
- Emergency alarm (separate from fire alarm) with remote monitoring required.
- Wall between pump room and caustic room must be fire rated

Northglenn has had difficulty procuring sulfuric acid in a timely manner, if at all, due to the chemical supply market conditions. Switching to an alternate chemical, such as citric acid would alleviate the supply issues but increase the required size of equipment on site.

4.3.5.3 Opportunity for Solar Array on Site

The feasibility of installation of a photovoltaic (PV) system is highly dependent on the available area for an array, solar resource, distance to transmission lines, and distance to major roads. In addition, the operating status, ground conditions, and restrictions associated with redevelopment of a brownfield impact the feasibility of a PV system. Based on an assessment of these factors, the Northglenn site is suitable for deployment of a PV system. For every acre of available land, industry standard production is an average of 0.357 GWh or 357 MWh of energy per year. The costs estimated are taken from levelized cost of electricity (LCOE) models, such as that developed by the National Renewable Energy Laboratory (NREL).

The economics of the potential system were explored using the current Xcel Energy Solar*Rewards Community Program and NREL data. One referenced project was a City of Aurora PV feasibility study on a PV project that would be eligible to receive a portion of the solar energy generated as a bill credit from Xcel Energy in exchange for a land lease to a solar developer. The 500-kW PV system would require about 4 acres of land [a 2-MW (the upper limit of community solar) system would require 14 acres].

PV system costs in the United States fell across all market segments from 2019 to 2020 as module prices continued to drive down system costs. While shortages of glass and ethylene-vinyl acetate laminate caused solar module prices to increase at the end of 2020, they still fell on an overall year-on-year basis. According to Wood Mackenzie's recently released U.S. Solar PV System Price report, average 100-megawatt utility-scale system costs in 2020 are $0.94/W, with the potential to fall 19% by 2025 (watt figures are DC). These costs will fall 13% from 2021 to 2022, driven by module price reduction as Section 201 tariffs are expected to phase out for imported products.

Depending on the technology used, for Northglenn to integrate a Solar Power array equivalent to the 1.5 MW emergency generator on site at the WWTP, 12-15 acres would be required for panels and ancillary equipment. Capital costs would be between $1,600,000 and $2,000,000.

4.3.6 Recommended Improvements for Existing Treatment System

Listed below are recommended improvements to the existing treatment system in addition to the equipment replacement recommendations resulting from the conditions assessment. These items address specific concerns raised by Northglenn and will help maintain and potentially improve overall performance of the WWTP.

- Demolish the heat exchange equipment located in the main process building and the UV Disinfection Building. Northglenn staff is already removing the heat exchange coils from the aeration basins.
- Relocate the clarifier launders to the outside wall of the clarifiers and install new skimmer arms that extend all the way to the launders. The inset clarifier launders are a maintenance hassle for the WWTP staff since they trap algae and other floatable material. Relocation of the launders will minimize the amount of regular maintenance required.
- Install the following analyzers to assist with process control: one new DO probe in each aeration basin and one NO_3 analyzer in each aeration basin.

- Install two additional H_2S sensors and transmitters on the main level of the headworks building to improve operator safety.

Table 4-36 presents the OPCC and program costs for the recommended improvements to the existing treatment system. These items are in addition to the recommendations from the conditions assessment detailed in Table 4-35. The OPCCs were developed using the same approach.

Program costs include:

- The OPCC, as described above
- Project contingency: 20% of OPCC
- Allowance for engineering and implementation, including design, construction services, and equipment startup and testing: 20% of the sum of OPCC and project contingency

Table 4-36 Costs for Recommended Improvements to Existing Treatment System

Plant Improvements		Cost
Demolition of Heat Exchange Equipment		$50,000
Relocation of Existing Secondary Clarifier Launders		$750,000
Installation of Additional Process Control Analyzers[1]		$90,000
H_2S Sensors		$8,300
Subtotal of Process Improvements		$848,300
Indirect Costs (Permits, Bonding and Insurance)		$42,415
Subtotal		$890,715
Contractor's Field General Conditions, Overhead and Profit	10%	$89,072
Subtotal with OH&P		$979,787
Construction Contingencies	30%	$293,936
Total Construction Costs		$1,273,722
Construction Escalation to Mid-Point of Construction	5%	$63,686
Total Opinion of Probable Construction Cost (Rounded)		$1,000,000
Project Contingency	20%	$200,000
Subtotal		$1,200,000
Engineering and Implementation	20%	$240,000
Total Program Cost		$1,440,000
TOTAL PROGRAM COST (Rounded)		**$1,400,000**

[1] Instrument costs include transmitters for each analyzer and one new controller per basin.

Included below are some additional items Northglenn should consider in more detail to determine the best course of action.

- Coordinate potential changes to the air to oil cooler with the blower manufacturer. Adequate cooling of the lubrication oil is critical to performance of the blower, and the controls for the air-cooled system are integrated into the blower local control panels. Potential relocation of the fans outdoors should consider temperature, snow and noise impacts.

- Monitor the impact of the proposed improvements for upgrading residuals handling at the WTP. Since the practice of sending alum residuals to the WWTP is going to change, effluent phosphorus concentrations are likely to increase. A new chemical feed system may be needed at the WWTP to earn nutrient credits.
- Address the safety concerns associated with the existing sulfuric acid storage and feed system, including performing a detailed code review of the chemical storage area. Consider alternative acids to lower pH prior to discharge into Bull Canal. Northglenn has had difficulty procuring sulfuric acid in a timely manner, if at all, due to the chemical supply market conditions. Switching to an alternate chemical, such as citric acid, would alleviate the supply issues but increase the required size of equipment on site.

Section 5

Future Conditions and Treatment Alternatives

The capacity and type of treatment processes needed to meet future regulatory requirements and population growth depend on future development of the service areas, flow variations, and flow characteristics. Future needs for the Northglenn WWTP have been estimated based on an analysis of the service areas and historic flows and loads and are presented in this section. Treatment alternatives for meeting future regulations are also documented.

5.1 Land Use and Population Projections

As discussed in Section 4.1.4, the 2020 Water MP (JVA Consulting Engineers, 2020) identified four remaining areas within the SSA that may be developed prior to reaching buildout. These areas were summarized in Table 4-2. The projected buildout population for the SSA is 43,000 people by the year 2040.

Development of the NSA is hampered by a lack of potable water. Northglenn's potable water system does not extend beyond the SSA. Nearby municipalities to the NSA include the City and County of Broomfield, City of Dacono, City of Thornton, and Todd Creek. There are no plans for any of these entities to provide water to the NSA. Existing homes and businesses are presumed to rely on well water.

The twenty-year planning horizon includes development of Section 36 only. Section 36 includes the WWTP and Bull Reservoir (**Figure 5-1**). The remainder of Section 36 is planned to be a mix of industrial and commercial uses with some green space. The remainder of the NSA lacks potable water and is unlikely to develop during the planning period, if at all.

Figure 5-1: Section Map for North Service Area

5.2 Flow and Load Projections

Northglenn is expected to reach its buildout population by the year 2040. Buildout flows and loads were estimated for the SSA by multiplying the future anticipated population of 43,000 residents by the per capita generation rates listed in Table 4-16. MM flows and loads were then calculated by applying the MMPFs given in Figure 4-15. Future flows and loads are summarized in **Table 5-1**.

Planning documents provided by Northglenn indicate that development within Section 36 will consist entirely of commercial and industrial uses. Colorado's Design Criteria for Domestic Wastewater Treatment Works recommends using maximum month average daily flows of 1500 gpd per acre and 2000 gpd per acre for commercial and industrial land uses, respectively, when actual flows are unknown and cannot be measured (CDPHE, 2012). Section 36 contains 640 total acres. A significant portion of Section 36 is currently occupied by Bull Reservoir, the WWTP, a fire training facility, and some open space. This leaves 367.5 acres for possible future development. Because the percentage of commercial and industrial land use is unknown at this time, a MM flow of 1750 gpd per acre was selected for planning purposes resulting in a potential future flow of 0.64 mgd from Section 36.

CDPHE's design criteria are silent on the question of how much BOD_5, TSS, NH_3, and TP commercial and industrial uses are expected to generate. While wastewater from light commercial facilities is expected to be similar in strength and composition to domestic wastewater, industrial wastewater is highly variable especially for manufacturing. Northglenn has an EPA-recognized pretreatment program (Appendix G). Industrial users would be managed under the program, which should prevent excessively high concentrations of any constituent from being discharged to the WWTP. For planning purposes, it was assumed that wastewater generated in Section 36 will be twice the strength of the domestic wastewater currently received.

The MM flows and loads for Section 36 were added to the MM flows and loads for the SSA to find the total flow and load for year 2040. They are summarized in Table 5-1.

Table 5-1: Estimated Future Flows and Loads

Parameter	2040 SSA	2040 Section 36	2040 Total
Annual Average Flow, mgd	4.04	-	-
Maximum Month Flow, mgd	5.13	0.64	5.78
Peak Hour Flow, mgd	10.10	-	-
BOD_5, average load, ppd	7,086	-	-
BOD_5, max month load, ppd	8,625	3,169	11,794
TSS, average load, ppd	8,847		
TSS, max month load, ppd	12,590	3,680	16,269
NH_3-N, average load, ppd	1,083	-	-
NH_3-N, max month load, ppd	1,301	478	1,778
TP, average load, ppd	255	-	-
TP, max month load, ppd	311	110	421

Note: Because peaking factors for commercial and industrial users are unknown, average daily flows and loads were not estimated for Section 36.

Development of the NSA outside of Section 36 is not anticipated to occur within the 20-year planning horizon. There are five additional sections that could be developed for residential, commercial, or industrial uses should potable water become available. Conversion of agricultural water rights to domestic water rights could make potable water available in the future. Complete development of the NSA could generate an estimated 4.5 mgd of flow in addition to the projected flows shown in Table 5-1, but this flow is not considered in the alternatives analysis conducted in Section 6.

5.3 Secondary Treatment Capacity

The existing 3-stage BNR process is rated for 4.2 mgd and 7,916 ppd BOD_5 on a MM basis. The permitted capacity of the BNR process was lowered after the ponds being used for primary treatment were decommissioned in 2017. The estimated 2040 MM flow of 5.78 mgd and BOD_5 load of 11,794 ppd both exceed the current permitted capacity. The alternatives analysis in Section 6 considers options for addressing the shortfall in treatment capacity.

5.4 Regulatory Requirements

As of January 1, 2021, Northglenn is required to achieve an effluent TP limit of 1 mg/L. Northglenn also has an effluent TIN limit of 15 mg/L. The TIN limit reduces to 14 mg/L on July 1, 2024. The next permit cycle is also likely to include a TIN limit of 10 mg/L based on the Water Supply use designation for Segment 1 of Big Dry Creek. Nutrient limits are expected to decrease significantly after 2027 based on the adoption of stringent nutrient criteria in Regulation 31. **Table 5-2** presents the interim nutrient standards found in Regulation 31. These may be revised prior to statewide adoption in 2027.

Table 5-2: Interim Numeric Nutrient Criteria (Regulation 31.17)

Parameter	Rivers and Streams - Warm
Total Phosphorus	170 ug/L*
Total Nitrogen	2,010 ug/L*
Chlorophyll-a	150 mg/m^{2**}

*Annual median, allowable exceedance frequency 1-in-5 years
** Summer (July 1 – September 30) maximum attached algae, not to exceed.

The adoption of stringent nutrient criteria for rivers and streams is not expected to translate into effluent limits requiring compliance within the 20-year planning period of this Plan due to a combination of three factors: the timing of the next permit renewal after Regulation 31 limits go into effect (January 2030), a 5-year compliance schedule for new nutrient permit limits, and a maximum of 10 additional years earned through Northglenn's participation in the Nutrient Incentives Program. As discussed in Section 3, Northglenn will need to continue to work with the Division to determine how best to handle the unique nature of the facility design in order to obtain up to an additional 10 years for the compliance schedule. Northglenn is pursuing credits for 2018, 2019 and 2020 for the months when the WWTP discharged to Big Dry Creek. If the Division decides against issuing credits, Northglenn will still have opportunity to earn credits before 2027. There is enough time remaining between January 2022 and December 2027 for Northglenn to maximize the nutrient credit earned if Northglenn can work out a solution that may include discharging to Big Dry Creek 12 months out of the year.

A detailed alternatives analysis of technologies that can achieve the Regulation 31 nutrient limits was not performed since those requirements are expected to be outside the 20-year planning horizon. However, Section 5.5 presents an overview of treatment technologies capable of achieving the Regulation 31 limits. There are numerous options that can achieve the current interim TP limit whereas options to achieve the current interim TN limit are quite limited, with likely only one reliable solution.

5.5 Achieving Nutrient Permit Limits based on Regulation 31

5.5.1 Future Total Phosphorus Permit Limit

Complying with a TP limit of 0.17 mg/L is challenging but achievable with multiple types of tertiary processes. It requires the following: 1) conversion of most soluble, reactive phosphorus in the wastewater to a particulate form of phosphorus and 2) effective and reliable solids removal to capture the particulate forms of phosphorus. Northglenn can continue to utilize enhanced biological phosphorus removal to reduce secondary effluent concentrations, but chemical phosphorus removal will likely be required to convert as much reactive phosphorus as possible to particulate form. A chemical (metal salt) storage and feed system would be required to accomplish this. An add-on tertiary process for improved solids removal would also be required.

In addition to the orthophosphate and particulate phosphate associated with the TSS, there is a component of effluent phosphorus (soluble and non-reactive, sNRP) that will pass through chemical treatment and tertiary solids removal and essentially cannot be removed. The typical range of sNRP in domestic wastewater ranges from 0.005 to 0.05 mg/L P. This is a parameter that Northglenn should start to monitor, but sNRP typically does not cause compliance issues with TP limits in the 0.17-0.2 mg/L range. The level of sNRP present in the final effluent will determine the level of chemical phosphorus removal required, however, since the final effluent orthophosphate concentration will have to be lower if the sNRP concentration is higher than typical.

Various add-on phosphorus removal technologies are currently available to meet the 0.17 mg/L TP effluent limit. Except for the algal nutrient recovery systems discussed below, all of these processes are physical/chemical systems which rely on chemical phosphorus removal. If current BNR performance is maintained, the tertiary process would be required to reduce TP from approximately 0.3 to 0.4 mg/L in the secondary effluent down to safely below the 0.17 mg/L TP on an average basis. As shown in Section 4, secondary effluent concentrations are in the range of 0.75 to 1 mg/L TP on a peak day basis. The processes presented below are typically able to treat secondary effluent with influent TP concentrations of 1 mg/L TP down to below 0.1 mg/L or 0.05 mg/L, so the periodic high concentrations are not an issue. The impact of no longer sending alum residuals from the WTP to the WWTP will have to be assessed when implemented. The loss of alum residuals at the WWTP is likely to increase effluent phosphorus concentrations, but a combination of enhanced biological phosphorus removal (EBPR), chemical phosphorus removal and a tertiary treatment process will be able to achieve an effluent TP concentration of 0.17 mg/L. The biggest impact will be on the quantity of chemical required to achieve the limit, the additional sludge production, and the alkalinity consumed from adding a metal salt.

The lowest daily effluent alkalinity recorded since 2014 is 100 mg/L as $CaCO_3$. Average daily effluent alkalinity ranges between 136 and 174 mg/L as $CaCO_3$. A dose of 1 mg/L of alum consumes 0.51 mg/L of alkalinity as $CaCO_3$. Similarly, a dose of 1 mg/L of ferric chloride consumes 0.92 mg/L of alkalinity as $CaCO_3$. Nitrification may be inhibited when the alkalinity concentration leaving the activated sludge process falls below 100 mg/L as $CaCO_3$. Supplemental alkalinity may be needed depending on the dose of ferric chloride required to precipitate both H_2S and phosphorus. Dosages of ferric chloride needed to precipitate H_2S have been reported to range from 4 to 15 mg/L per 1 mg/L H_2S. At present, there appears to be sufficient alkalinity to support H_2S precipitation only. Jar testing is needed to determine actual dosages and alkalinity drop.

5.5.1.1 Phosphorus Removal Treatment Options

There are numerous physical/chemical treatment technologies that have demonstrated the ability to comply with the stringent TP limit of 0.17 mg/L. These physical/chemical technologies as well as other emerging processes, such as algal nutrient recovery systems, are presented to provide an overview of the currently available technologies. Northglenn should continue to monitor the development of phosphorus removal processes between now and the anticipated adoption of criteria (2027) to determine if any new technologies should or can be considered.

Most of the filtration processes discussed below were considered for phosphorus removal in the 2012 MP. The future effluent TP limit of 0.17 mg/L was unknown at the time, so the previous evaluation did not consider chemical addition in conjunction with effluent filtration. As noted previously, chemical addition will be required in addition to any of the filtration processes presented to achieve this low limit. The other types of processes presented below were not considered in detail.

5.5.1.1.1 Cloth Filtration

Cloth filtration is a physical process used to remove solids from wastewater. Filters use media of varying sizes and materials to physically strain out particles larger than the openings in the media. There are two main flow paths used by cloth filter manufacturers: outside-to-inside and inside-to-outside. In the first option, filter influent enters the tank and flows through the filter media into a central tube that conveys the filtered water out of the filter basin. In the second option, water flows in the opposite direction: it enters the central tube and then flows through the filter media into the tank. Both outside-to-inside and inside-to-outside systems provide a means for backwashing the filters and a means for removing solids from the system.

If soluble orthophosphate is converted to a particle prior to the filters, then cloth filters can produce a high-quality effluent that meets low-level phosphorus limits. Installation of flash mix tanks and flocculation tanks is typically recommended to meet very low phosphorus limits (effluent concentrations less than 0.1-0.15 mg/L). Flash mix and flocculation may not be required to achieve the 0.17 mg/L TP permit limit, but this would have to be evaluated in more detail. CDM Smith has designed cloth filter facilities that are successfully meeting 0.1 mg/L concentration limits without dedicated flash mix and flocculation, but the activated sludge clarifiers must have sufficient capacity to handle the higher solids loads associated with chemical addition to the mixed liquor and sufficient alkalinity. Providing flash mix and flocculation upstream would ensure the filtration facility has the flexibility to achieve lower limits, if needed.

There are a number of different cloth filter manufacturers (7 total), with the most established being Aqua-Aerobic Systems, Inc. and Veolia Water Technologies. The Aqua-Aerobic filters use a “pile” or a velour-style cloth, and this cloth has been adapted to a variety of mechanical configurations, including disks, drums, and diamonds. Aqua-Aerobic filters have an outside-to-inside flow path. The two offerings from Aqua-Aerobic Systems most suitable for a WWTP the size of Northglenn are AquaDisk® rotating cloth filters and AquaDiamond® traveling bridge filters. Both are presented below.

AquaDisk® – Rotating Cloth Filters

A schematic of the AquaDisk filter design is presented in **Figure 5-2**. Inlet water enters the tank and completely submerges the cloth media. Liquid passes through the media by gravity and enters the internal portion of the disk where it is discharged through the center shaft. The suspended solids are captured on the outside surface of the cloth disk filters, and as the solids accumulate, the water level in the tank rises. When the water level reaches a predetermined level, a backwash cycle is initiated. The disks rotate during the cycle with two being cleaned at a time; filtration is not interrupted during the cleaning. The backwash water is pulled from the filtered effluent and is directed away from the filters. Solids settle to the bottom of the tank and are periodically pumped out of the tanks. All backwash and tertiary solids are typically pumped back to the front of the liquid stream due to the low solids concentration.

Each disk is approximately 4.5 feet in diameter and there are 8 lightweight, removable segments for ease of maintenance. There can be up to 12 disks in a single filter. Five micron (PES-14) cloth would be recommended to meet the 0.17 mg/L permit limit.

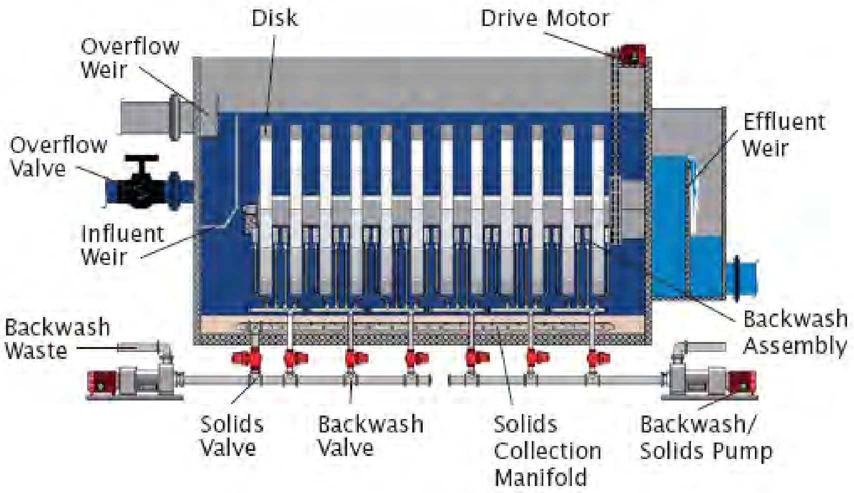


Figure 5-2: Schematic of the AquaDisk Filter

AquaDiamond® – Traveling Bridge Cloth Filters

An alternative cloth media option for a medium-size plant such as the Northglenn WWTP is the AquaDiamond filtration assembly by Aqua-Aerobic Systems as presented in **Figure 5-3**. The AquaDiamond assembly consists of 8 diamond-shaped lateral cloth media assemblies that are mounted along the length of the filter tank. Five micron cloth would be used to meet the 0.17 mg/L TP permit limit, as with the AquaDisk option. A traveling bridge is used for backwashing of the lateral media assemblies. As the solids build up on the filter cloth, the water level in the tank will rise. Once the water level reaches a high water level setpoint, the filter will initiate a backwashing mode to remove the solids. As with the AquaDisk, backwash and tertiary solids are typically pumped back to the front of the liquid stream due to the low solids concentration.

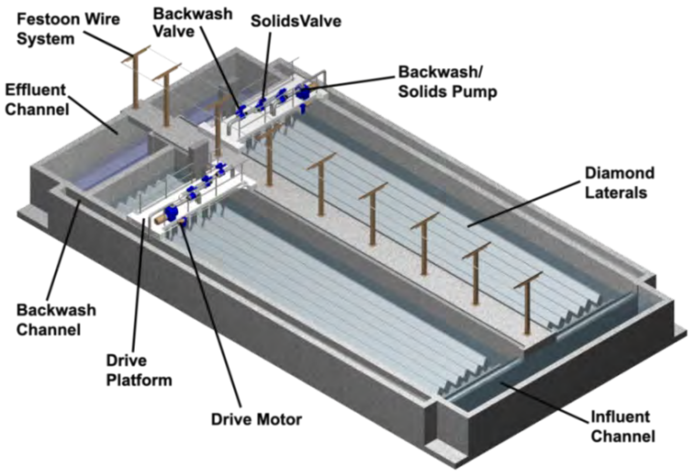


Figure 5-3: Schematic of the AquaDiamond Filter

The many existing treatment facilities that have conventional traveling bridge sand filters for phosphorus removal do so because these filters were installed before cloth filtration technologies were available. Some of these facilities have been converted to the traveling bridge cloth filters since an increased treatment capacity can be achieved in the same footprint. Conventional traveling bridge sand filters are not presented in this section since a new filtration system would have to be constructed at the Northglenn WWTP and the footprint of conventional traveling bridge sand filters is much larger than cloth filters.

5.5.1.1.2 Compressible Media Filtration

Compressible media filters are another physical process to remove solids from wastewater. These filters use synthetic compressible media within the filters. There are two established manufacturers of this type of filter: Schreiber Fuzzy Filter™ and WesTech FlexFilter™. **Figure 5-4** presents the schematic for the Fuzzy Filter with 1-1/4 inch fiber balls. In these filters, a moveable plate with a motor actuator compresses the media in preparation for operation. Flow is upward at hydraulic loading rates up to 30 gpm/ft^2, which results in a smaller footprint than other filtration technologies. When backwashing is required, the moveable plate is raised to decompress the media and air is used to scour the media. Similar to cloth media filtration, backwash water, including solids scoured from the media, would likely be returned to the front of the liquid treatment process. Installation of flash mix tanks and flocculation tanks could also be considered with this technology.

Figure 5-4: Schematic of Schreiber Fuzzy Filter

Figure 5-5 presents the schematic for the FlexFilter. The FlexFilter relies on hydrostatic pressure rather than a movable plate to provide compression. The influent applies the force to the compression bladder causing the media to compress. This results in a tapered compression with densely compressed media at the top that graduates to an expanded bed toward the surface. Larger particles get trapped in the upper portions and smaller particles are captured as the liquid works its way down. The porosity gradient allows for a higher mass load prior to backwash. Feed to the filter must be stopped for backwash, similar to the Fuzzy Filter, allowing the media to decompress. Water level increases as the filter bed becomes plugged, signaling the need for backwash. An air scour is also used to remove solids from the media.

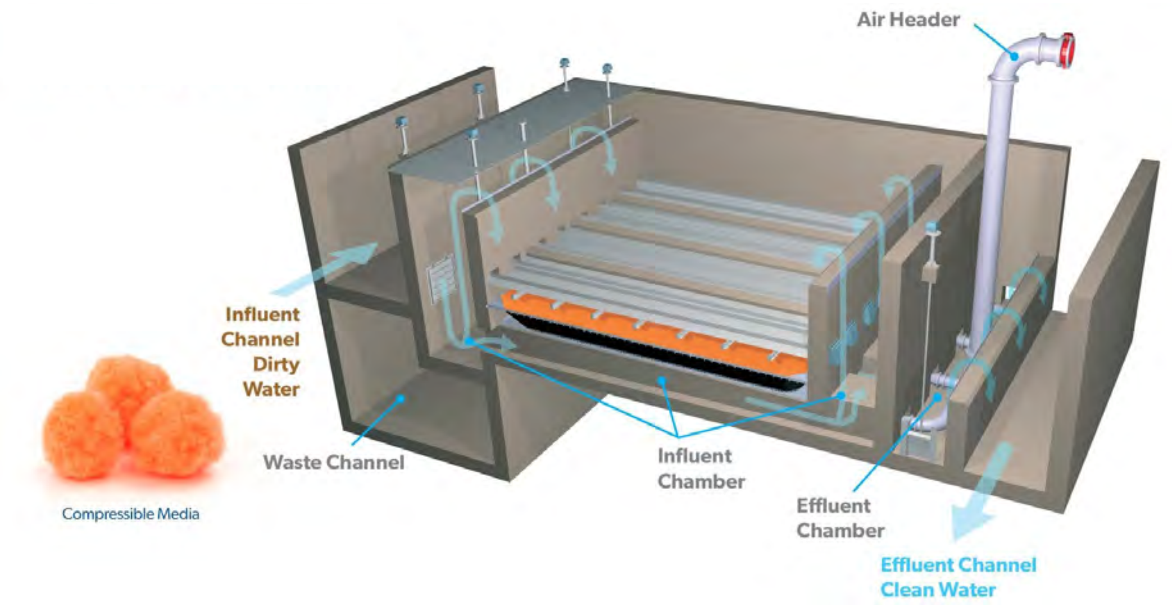


Figure 5-5: Schematic of WesTech FlexFilter

5.5.1.1.3 Deep Bed Sand Filtration

Deep bed sand filtration is a physical process used to remove solids from wastewater which has been successfully used in municipal wastewater tertiary treatment for over 50 years. The granular sand media typically ranges in depth from 4 to 6.5 feet depending on the application, and the sand is placed on top of a gravel support layer approximately 0.5 meters deep. The total depth of the filters, including freeboard, typically ranges from 4.0 to 6.0 meters. Backwash water and air scour blowers are required to clean the filters from solids buildup, and unlike cloth filters a deep bed filter must be taken out of production to perform the backwash procedures. Deep bed filters could be used at the Northglenn WWTP, but it is unlikely they would be preferable to other alternatives since they typically require about double the site footprint and about 1.5 to 2 times the construction cost compared to cloth filters.

5.5.1.1.4 Continuous Upflow Sand Filtration

Continuous backwash sand filters achieve continuous filtration as wastewater is distributed through a counter-flow sand filter. The solids and impurities in the wastewater are trapped in this sand filter material. The effluent filtrate exits the sand bed via an effluent weir, while the sand particles are cleaned and recycled in the filter system.

There are two main manufacturers of continuous upflow sand filters used for phosphorus removal: Nexom (formerly Blue Water Technologies, Inc.) and Parkson. Both vendors manufacture deep bed continuous backwash filters which are similar in design and layout. CDM Smith has had successful projects with the Blue PRO system. This system is described further below.

Blue PRO® - Continuous Upflow Sand Filtration

Blue PRO is a continuous backwash filter process that takes advantage of adsorption as well as some filtration (**Figure 5-6**). Nexom refers to its process as 'reactive filtration'. Iron-based chemicals (Blue PRO can only use iron-based chemicals) are first added to the influent wastewater in the pre-reactor zone (an in-line mixing pipe assembly), which is then pumped through distribution arms in the bottom of the sand bed. As the influent fluidizes the bed, the iron chemicals react with the silica sand and create a hydrous ferric oxide coating. Phosphates adsorb to the coating. Adsorption is thus the primary mechanism for phosphorus removal, while coagulation/filtration offers some additional removal, but to a lesser extent.

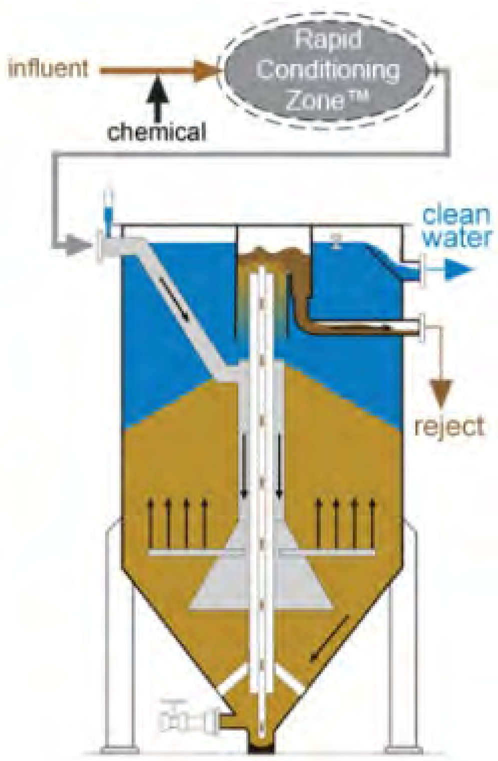


Figure 5-6: Schematic of the Blue PRO Process

The Blue PRO technology uses continuous backwash, deep bed, upflow granular sand filters with a total depth of approximately 19 feet. Each filter module consists of a bottom cone, an airlift pump and inlet, and discharge and backwash piping. The units continuously backwash due to the upflow design and the airlift pump system that returns a sand slurry from the bottom of the cone back to the top of the bed. The airlift pumps are supplied with compressed air by a vendor-provided compressor package, housed in a separate building or enclosure. During the airlift process, iron and phosphorus particles are abraded from the sand and the sand slurry (comprising sand, solids and water) is pushed to the top of the airlift pipe and into a reject compartment. From the reject compartment, the sand falls into the sand washer and is returned to the filter bed, while the lighter rejected solids are carried over the reject weir.

Treated water emerges from the top of the filter and exits the sand bed via an effluent weir and is discharged into an effluent line.

5.5.1.1.5 Membrane Filtration

Microfiltration (MF) and ultrafiltration (UF) are the two processes that are most often associated with the term "membrane filtration" and are alternatives to the media or cloth filters discussed previously. These membranes provide a physical barrier, resulting in more complete rejection of particles greater than a specified size (on the order of 0.1 micrometers (µm) for MF and on the order of 0.01 µm for UF). MF and UF membranes remove particles down to such small sizes that they remove both pathogens and particles that adversely affect the aesthetic appearance of the water. Membrane filtration has been successfully employed in the treatment of secondary effluent, and in recent years, competition among manufacturers has dramatically decreased both the initial and the long-term operating costs of membrane filtration.

Although MF/UF membranes are found in many configurations (hollow fiber, spiral wound, flat sheet, plate and frame), hollow fiber would be recommended for tertiary treatment. These fibers have an inside diameter ranging from 0.4 to 1.0 mm and a wall thickness ranging from 0.07 to 0.6 mm. The physical strength of the fibers allows them to be backwashed. Hollow-fiber membranes are operated in either an inside-out or outside-in mode. During inside-out operation, the feed enters the fiber lumen and passes through the fiber wall to generate filtrate. During outside-in operation, the filtrate is collected in the fiber lumen after the feed is passed through the membrane.

The pressure that is used to drive water through the membrane material is termed transmembrane pressure (TMP). Depending on the way membrane modules are pressurized, they are available in two basic configurations: pressure-vessel systems and submerged systems. For wastewater applications, MF/UF membranes typically operate at pressures between 30 and 620 kPa (approximately 4 to 90 psi).

The primary technical challenge with the use of membranes for wastewater treatment is the high potential for fouling. Membrane fouling can be caused by colloids, soluble organic compounds, and microorganisms that are typically not well removed with conventional activated sludge processes. Fouling increases feed pressure and requires frequent membrane cleaning. This leads to reduced efficiency and a shorter membrane life. Two types of chemical cleaning regimens are typically performed: 1) chemically enhanced backwashes (CEBs) to maintain the day-to-day membrane permeability, and 2) chemical clean-in-place (CIP) to restore the membrane permeability between phases or when the TMP reaches the terminal value. CEBs are preventive cleans performed at specified regular intervals (such as once per day or once every several days), typically using only chlorine. Other chemicals including strong acids may also be used depending on the supplier's membrane chemical compatibility and foulants of concern. CIP cleans are an intensive chemical cleaning used to restore the membrane permeability to pre-fouled conditions and are often recommended to be performed once per month. The chemicals used for recovery cleanings will depend on the severity of the organic or inorganic membrane fouling and can include sodium hypochlorite, sodium hydroxide, and citric acid or mineral acids.

MF filtered water quality is typically very consistent in terms of removal of suspended solids (measured as turbidity). The turbidity will be less than 0.1 NTU during operation, regardless of the quality of source water to the membranes. MF can typically handle a wider range of influent water quality than the granular media or cloth filter technologies.

Figure 5-7 presents a schematic of a hollow-fiber MF/UF membrane. There are a number of different manufacturers of hollow fiber MF/UF membranes, including Evoqua (formerly Siemens), Suez Water Technologies & Solutions (Zenon), Pall Corporation, and Toray.

Membrane microfiltration is not used for phosphorus removal as often as other filtration technologies because it is an expensive technology, it has a high footprint requirement, it produces a much higher volume of return flow than the other processes, requires the use of several more chemicals and more operator attention for cleaning the membranes, and requires more power to pump through the pressure membranes. However, for facilities that desire to produce a high-quality effluent for reclaimed water purposes, this could potentially be a viable option.

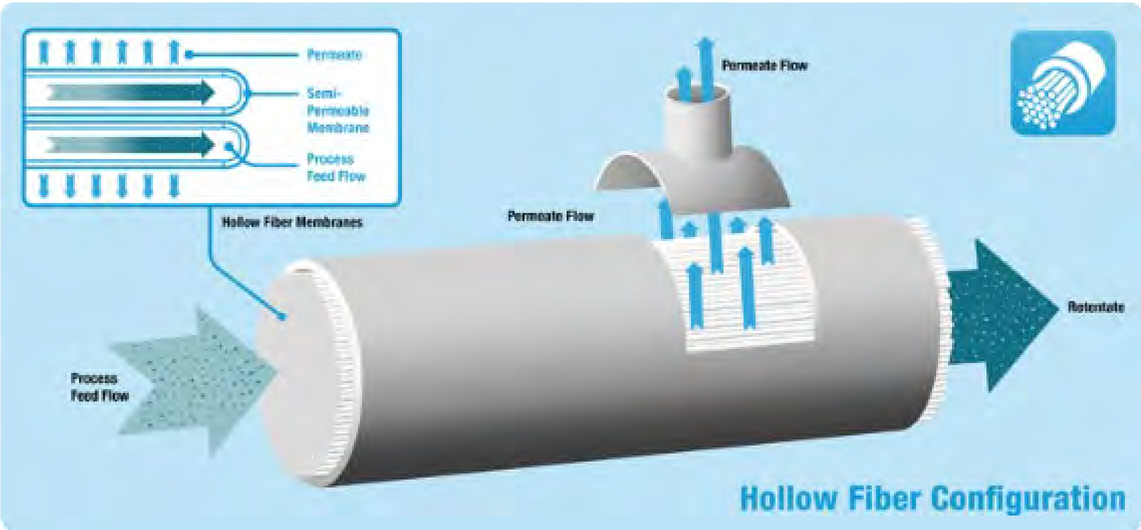


Figure 5-7: Hollow Fiber Microfiltration Membrane Schematic

5.5.1.1.6 Ballasted Flocculation/High Rate Clarification

High rate clarification, also known as ballasted flocculation, involves the rapid dispersion of a metal salt/polymer/ballast mixture, followed by flocculation and settling. The high level of particle removal achieved with ballasted flocculation makes the process suitable for tertiary phosphorus removal down to low effluent concentrations.

There are three main types of high rate filtration processes that could be used for tertiary phosphorus removal: microsand-ballasted flocculation (Actiflo® system by Veolia Water Technologies or RapidSand™ by WesTech), and magnetite-ballasted flocculation (CoMag® system by Evoqua). Each system uses a different ballast, which results in significantly different design criteria, footprints and system layouts.

Actiflo® – Ballasted Flocculation with Microsand

The Actiflo system, as presented in **Figure 5-8**, is based on the use of microsand to enhance flocculation and act as a ballast resulting in a floc with good settling characteristics. In the process, a metal salt is added to the influent wastewater, which is agitated for approximately two minutes in a coagulation tank. Next, the wastewater flows into a maturation (flocculation) tank and polymer and sand are added. The maturation tank plays the role of a flocculation basin, growing the particles formed from the interaction of the remaining solids from secondary treatment, polymer, metal salt, and the ballast. The wastewater enters an upflow tube settler following the maturation tank. The sand adds weight to the newly formed flocs, which allows them to settle rapidly. The tube settlers also minimize the overall footprint by greatly increasing the capacity for particle removal per square foot.

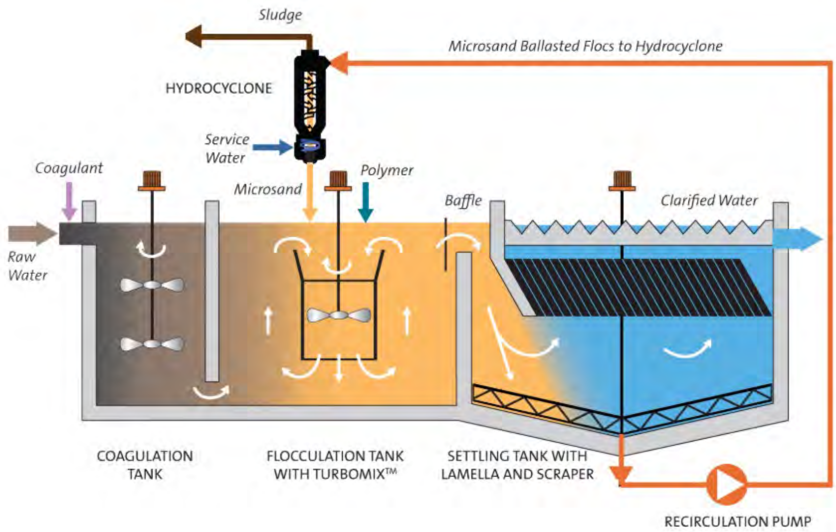


Figure 5-8: Schematic of the Actiflo Process

Sludge formed in this process, which contains the ballast mixture, is collected at the bottom of the tube settler tank. This sludge is typically about 0.1 to 0.5 percent solids and is pumped through a hydrocyclone. The purpose of the hydrocyclone is to separate the sludge from the microsand and to recover the ballast so it can be returned to the process. Veolia estimates that approximately 1 to 3 g of microsand are lost per cubic meter of water treated, and this sand must be replenished over time. **Figure 5-9** is a photograph of two hydrocyclones installed at the 10 mgd (design peak flow) Actiflo facility in Webster, Massachusetts, which was installed to meet a 0.1 mg/L effluent phosphorus limit.

A direct competitor to the Veolia Actiflo process is the RapiSand system by WesTech Engineering Inc. This system is a newer entrant to the market, but it appears to provide an identical process scheme to the Veolia Actiflo process.

Figure 5-9: Actiflo Equipment at the Webster, MA WWTF

CoMag® – Ballasted Flocculation with Magnetite Ballast

The CoMag process by Evoqua (**Figure 5-10**) is a ballasted flocculation process that uses fine magnetite powder as a ballast. A metal salt and anionic polymer are used in the coagulation and flocculation processes along with magnetite to create a dense floc which settles rapidly as a result of the high specific gravity of the magnetite, which is roughly 5.2. Settling can thus occur in a clarifier that has a smaller footprint than a conventional clarifier. The ballast and sludge mixture is pumped through an inline shearing mechanism, called a shear mill. Due to the properties of the magnetite, the ballast is recovered using a magnetic drum to which the magnetite adheres once sheared from the sludge. Recovered magnetite is returned to the CoMag process and sludge is wasted from the process, typically back to the headworks of the plant. Evoqua can provide a guarantee for magnetite recovery efficiency (typically of >99%), and the lost magnetite must be replenished over time. Photographs of the shear mills and magnetic recovery drums at the Marlborough Easterly WWTF in Marlborough, Massachusetts are included in **Figure 5-11**.

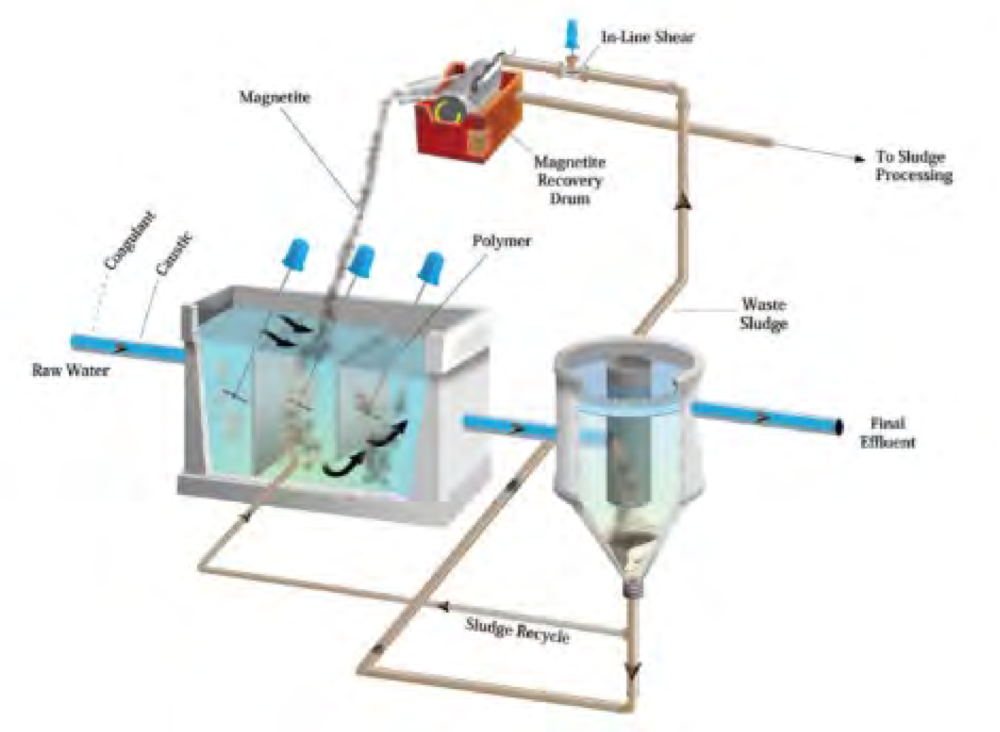


Figure 5-10: Schematic of the CoMag Process


Figure 5-11: Magnetite Recovery Drums over Ballast Mix Tank (left) and Shear Mills (right) at the Marlborough Easterly WWTF

5.5.1.1.7 Dissolved Air Flotation

Solids formed during the coagulation process in a dissolved air flotation (DAF) system are removed by attaching microbubbles to the floc, floating solids to the surface, and skimming by mechanical or hydraulic means. The system has a high clarification rate – typically 12 to 18 gpm/ft^2. DAF is an excellent solids removal solution for water having high levels of algae and other low-density particles that cannot be removed efficiently and effectively with sedimentation.

DAF systems have been installed for tertiary phosphorus removal. However, DAF manufacturers have historically been unwilling to provide performance guarantees for phosphorus concentrations lower than 0.2 mg/L. Thus, this technology would not be suitable for achieving anticipated TP limits.

5.5.1.1.8 Algal Tertiary Treatment

Any wastewater treatment operator is familiar with the nuisance of cleaning algae in secondary clarifiers and disinfection basins exposed to sunlight. The use of algae for nutrient uptake in an engineered fashion is not a new concept, but historically it has been limited to lagoons or raceway ponds, which require a significant amount of land. In recent years, several innovative algae-based nutrient removal technologies have been developed to target low-level phosphorus and nitrogen limits within a smaller footprint. Two algae-based process technologies are described below.

Clearas ABNR™ – Advanced Biological Nutrient Recovery

A leading manufacturer of algae-based tertiary treatment is Clearas Water Recovery, which produces a patented process called Advanced Biological Nutrient Recovery (ABNR™) system. The ABNR system is similar in concept to an activated sludge membrane bioreactor (MBR) system, except algae is used to remove the nutrients rather than microorganisms. Light and carbon dioxide are supplied to the algae population when in contact with the secondary effluent in a photobioreactor that consists of several long, circuitous fences of small-diameter clear PVC or glass pipe located inside a greenhouse structure. The algae uptake the nitrogen and phosphorus in the secondary effluent to grow. Similar to a suspended growth activated sludge process, some of the algae is returned to the influent of the process to maintain a viable population of algae, and a small portion of the algae is harvested from the return algae pipe.

Algae are separated from the process stream through ultrafiltration membrane technology. A schematic of the Clearas ABNR process is shown in **Figure 5-12**. Note that this is a constant-flow process, and thus any peak wet weather flows that exceed the design capacity of an ABNR system would have to be bypassed and blended with the ABNR effluent.

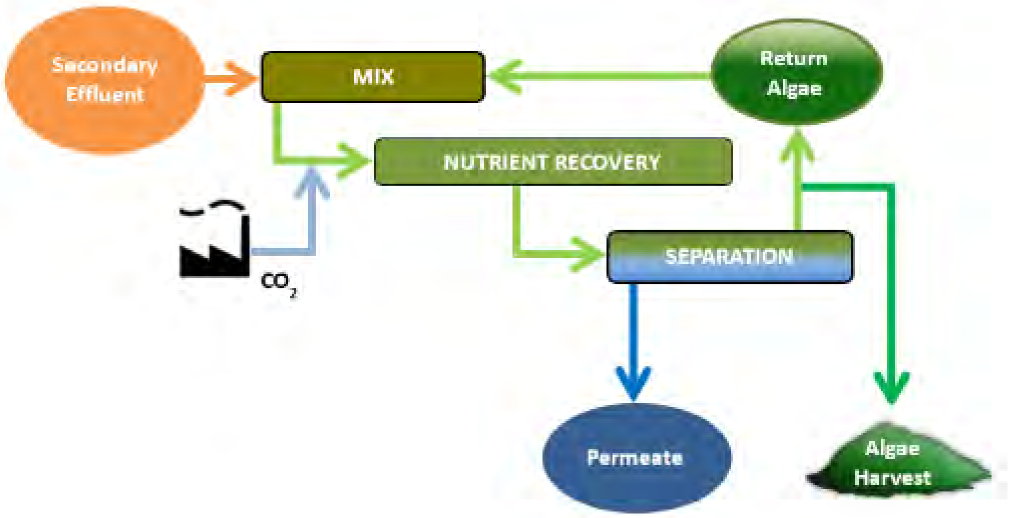


Figure 5-12: Schematic of the Clearas ABNR Process

Potential benefits of this process are that no chemical is required to achieve phosphorus removal to very low concentrations (< 0.05 mg/L); the process recovers nutrients in a biomass that has a variety of beneficial uses, some of which have potential market value; the algae sequester carbon dioxide, a greenhouse gas; the algae release dissolved oxygen into the water; and the process produces high quality effluent with potential for a variety of water reuse applications.

Figure 5-13 includes photographs of the clear photobioreactor piping and the membrane filtration tank installed for an extended pilot of the Clearas ABNR process at the Upper Blackstone Water Pollution Abatement District in Millbury, Massachusetts. The pilot was operated at a constant flow of 11 gpm for approximately 18 months in 2014 to 2015. Although it experienced some performance challenges at the outset, those challenges were ultimately traced to a poor mixing design inside the membrane tank. Once those mixing issues were rectified, Clearas was able to consistently demonstrate ultralow effluent phosphorus (< 0.03 mg/L) for a period of several months in 2015.

Figure 5-13: Clearas ABNR Pilot (11 gpm) (L: Photobioreactor with LED Lights; R: Flat Sheet Membrane Tank)

ABNR is a promising technology with the potential sustainability benefits of generating a biomass product with beneficial reuse value rather than producing a chemical sludge requiring disposal. However, there are concerns related to the feasibility of implementing the technology at this time:

- ABNR requires a substantially larger footprint (over three times) compared to other technologies.
- The capital cost of the ABNR equipment is substantially higher than other technologies, and the economic feasibility would rely on a stable revenue stream from the sale of the algae byproduct, which is an uncertain market at this time.
- The system incurs substantial power costs for normal daily operation of pumping and lights.
- Growing algae in a closed loop photobioreactor requires the bubbling of CO_2 into the system. The cost of purchasing CO_2 can be significant, and the capture of CO_2 from combustion on-site has not yet been proven at a full-scale facility to date.

This technology should remain in consideration as it is further developed and proven at more full-scale facilities.

Gross-Wen Technologies RAB – Revolving Algal Biofilm

An alternative algae tertiary treatment system provider to the Clearas ABNR system is the revolving algal biofilm (RAB) technology by Gross-Wen Technologies (GWT), which was developed by its founders at Iowa State University. The RAB system uses vertically-oriented conveyor belts that provide a surface for algae to grow that is exposed to the atmosphere. **Figure 5-14** provides a schematic explaining how the RAB system works. The RAB system avoids the CO2 transfer and pumping energy requirements of the Clearas ABNR's photobioreactor. An additional advantage of the RAB system compared to the Clearas ABNR process is that the algae is scraped from the belts in a relatively thick biomass, thus reducing solids handling requirements.

Although the RAB system has potential to provide a sustainable nutrient removal option in a smaller footprint than traditional algae treatment solutions, the company has not yet matured the RAB system into a proven technology. GWT received its first order from a municipal entity in 2018 for installation of the RAB system to achieve nitrification of primary effluent upstream of an activated sludge system. However, GWT has not yet installed a full-scale system for tertiary treatment, and it would need to be paired with a solids separation device downstream because the effluent would contain approximately 50 mg/L of TSS as algae. Piloting of different solids separation technologies, including membranes, would be required to demonstrate and validate which technologies could sufficiently polish the effluent to achieve low effluent nutrient limits.

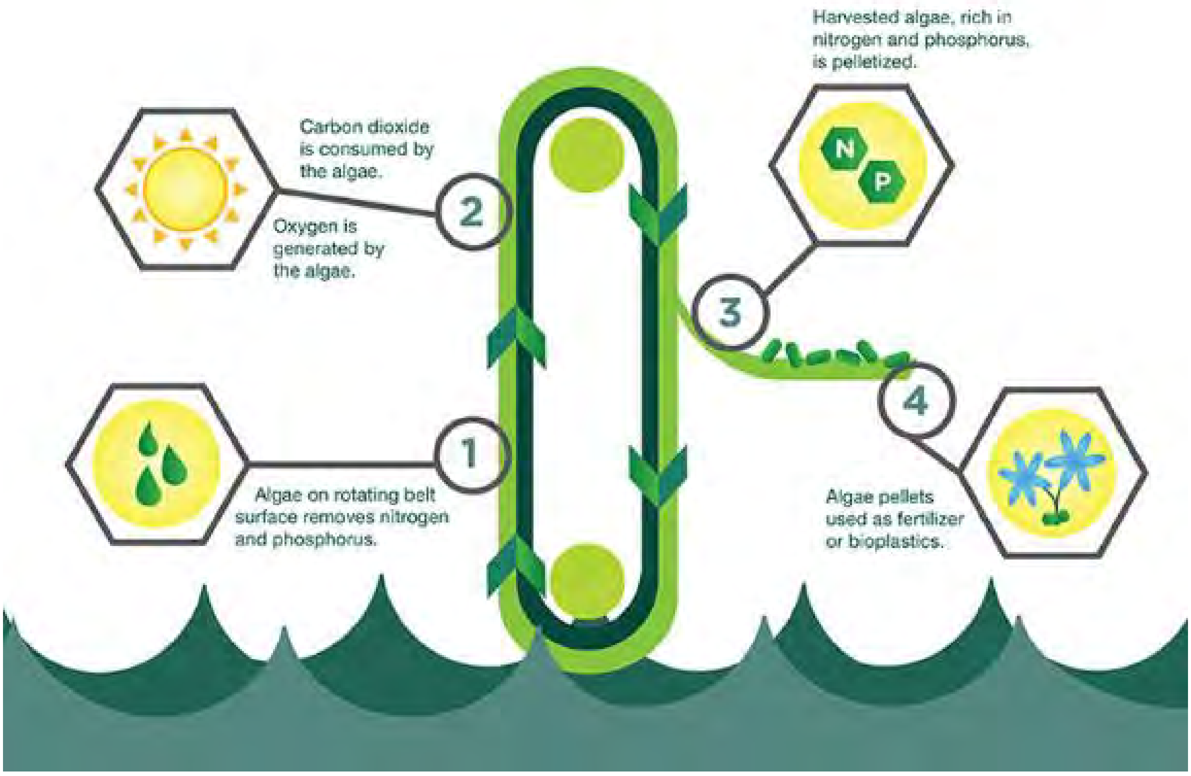


Figure 5-14: GWT Revolving Algal Biofilm System Schematic

5.5.2 Future Total Nitrogen Permit Limit

Complying with a TN permit limit of 2.01 mg/L is extremely challenging and will require implementation of the most efficient treatment technologies available, beyond suspended growth BNR processes or tertiary filtration processes that can achieve a TN limit of 3 mg/L. A TN limit of 3 mg/L is commonly held to be the limit of technology and numerous facilities have demonstrated the capability to meet it. A TN limit considers all forms of nitrogen in the effluent, and complying with a TN limit of 3 mg/L requires the following: 1) complete nitrification, down to an effluent NH_3-N of about 0.2 mg/L to 0.75 mg/L; 2) very efficient denitrification, such that the effluent NO_3- and NO_2-N (or NOx-N) is less than 1 mg/L; 3) a refractory dissolved organic nitrogen (rDON) concentration that is not excessive, typically in the range of 1 mg/L; and 4) a very efficient solids removal system to capture particulate organic nitrogen in the effluent TSS. Particulate organic nitrogen typically comprises 7-8 percent of the effluent TSS, such that for every 1 mg/L effluent TSS there will be 0.07 to 0.08 mg/L of particulate organic N. Therefore, effluent TSS from the plant must be less than 3 mg/L to achieve an effluent TN concentration of 3 mg/L. To reduce the effluent TN to less than 2 mg/L on a consistent basis requires removal of all suspended solids and the removal of dissolved solids as well. Reverse osmosis (RO) is one of the few technologies available that can remove soluble nitrogen, including NO_3, some NH_3, and soluble organic nitrogen from wastewater.

5.5.2.1 Nitrogen Removal Treatment Option

RO, with a nominal pore size of 0.0001 µm or 100 atomic mass units, is one of the few technologies available that can remove TDS from wastewater, which is what is needed to achieve a TN limit of 2 mg/L. RO is often designed with three passes, the first two passes are in series with an additional pass used on the reject from the first pass to maximize water recovery and maximize concentration of the reject. This is shown schematically in **Figure 5-15**. For wastewater applications, RO facilities typically operate at feed pressures between 1,000 and 2,100 kilopascals (kPa) (approximately 150 to 300 psi) and utilize cross-flow membranes wherein the feed water is pumped tangentially to the membrane surface and the differential pressure across the membrane causes a portion of the feed water to pass through the membranes, leaving dissolved constituents in the crossflow. The treated portion of the water, termed permeate, is the final treated effluent from the system.

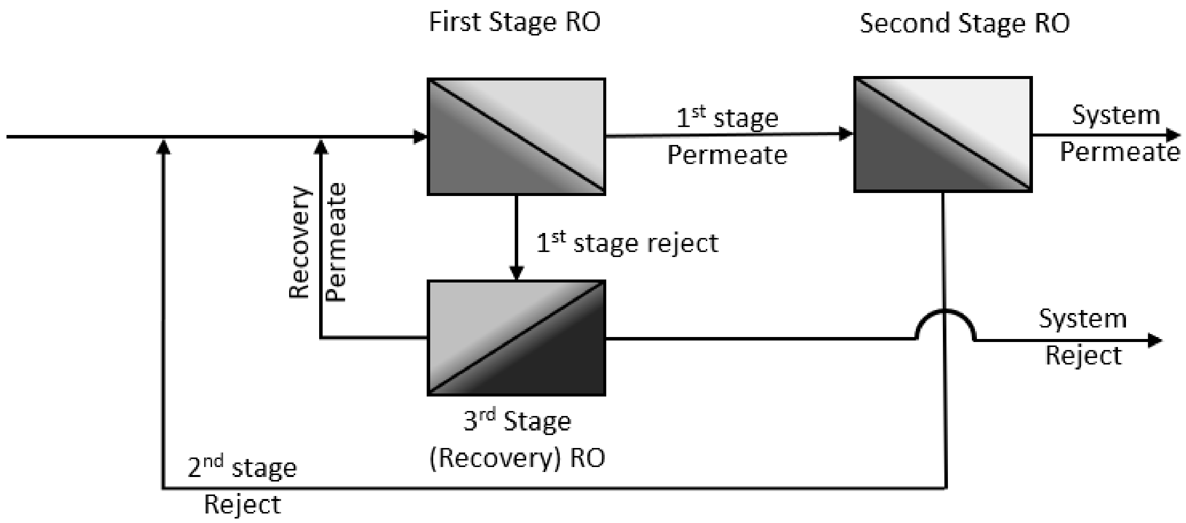


Figure 5-15: Schematic of Three-Stage Reverse Osmosis

In traditional RO, the portion of the water that did not pass through the membrane (the “concentrate” or “reject”) is continuously bled from the RO system. This “one-pass” process results in water recovery on the order of 50%. To improve water recovery, a second RO stage is usually coupled with the first stage so that the reject from the first stage is treated in a second traditional RO system, to increase overall water recovery to 75 to 80% depending on water chemistry. Each pass typically has a dedicated feed tank and feed pump (not shown). Innovations in RO technology has allowed some vendors to offer RO systems that can achieve >95% recovery in a single stage.

A low-pressure membrane system, such as the MF/UF systems described under phosphorus removal and shown in **Figure 5-16**, is also required upstream of the RO system for pretreatment. The pre-treated filtrate tank would serve as the RO feed tank.

Figure 5-16: Typical Pressurized UF Configuration

The RO system uses two chemical cleaning systems to maintain membrane performance: an antiscalant feed system to help prevent scaling of the membranes and a CIP acid (often hydrochloric) feed system to remove any formed scale. Scale is often a mixture of hardness (calcium and magnesium ions) and phosphate or carbonate anions. Chemical addition to the RO permeate may also be required due to low TDS.

RO is an expensive technology, has a large footprint and is both energy and maintenance-intensive. RO also produces a large reject stream (15-25% depending on the characteristics of wastewater being treated) that would also have to be considered. The RO concentrate is high in TDS and can cause issues with accumulation of TDS over time if returned to the head of the WWTP. Discharging the RO concentrate to a receiving water would not be an option due to the high concentration of TDS and also potential toxicity effects of the reject streams. Specialized treatment of the concentrate (brine) would likely be required prior to blending it or returning it to the front of the plant.

Northglenn should continue to monitor the development of any innovations in nitrogen during the planning period of this Plan as well as potential changes to the TN limit. An increase in concentration to 3 mg/L opens the door to numerous other effective treatment technologies, including multiple types of denitrification filters, membrane bioreactors in a 5-stage configuration, and potentially even 5-stage BNR (if sufficient carbon quantities are available to support both nitrogen and phosphorus removal in the activated sludge process). Another approach that could be considered for meeting a 2 mg/L TN limit is achieving 3 mg./L TN with one these mainstream treatment processes, treating only a portion of the forward flow with RO, and blending the two effluent streams. This approach doesn't eliminate the need for RO treatment, but it does reduce the size of the system required and the quantity of RO concentrate produced.

5.5.3 Algae Control in Bull Reservoir

The Bull Reservoir effluent pump station is located on the north end of the reservoir. The pump station has three pumps and was designed with three withdrawal points located 4.5 ft, 16.5 ft, and 24.3 ft above the bottom of the reservoir. The topmost withdrawal point is no longer in use. A broken valve prevents it from being used.

Algae growth in Bull Reservoir increases during the summer months. This creates three potential issues for discharge permit compliance: pH, TSS and future TP limits. As shown in **Figure 5-17**, Algae take in sunlight, nutrients, and dissolved carbon dioxide during the day and convert it into stored sugars. At night, algae chemically burn the sugars created during the day to recover energy. This process removes dissolved oxygen from the water and produces carbon dioxide. The result is higher reservoir pH and DO during the day and lower reservoir pH and DO at night. Longer summer days skew the cycle towards increasing pH and DO. It is common for reservoirs to see pH fluctuations from 6 to 11 over a 24-hour period. The discharge permit limits pH in the final effluent to a range of 6.0 to 9.0 standard units.

Northglenn manages effluent pH in two ways. First, discharges from Bull Reservoir are scheduled to the greatest extent possible to occur when the pH is within acceptable limits. Second, when discharges must be made and the pH is greater than 9.0 standard units, Northglenn has the ability to add sulfuric acid at the reservoir effluent pump station. Sulfuric acid reduces the pH below 9.0 before effluent enters Big Dry Creek.

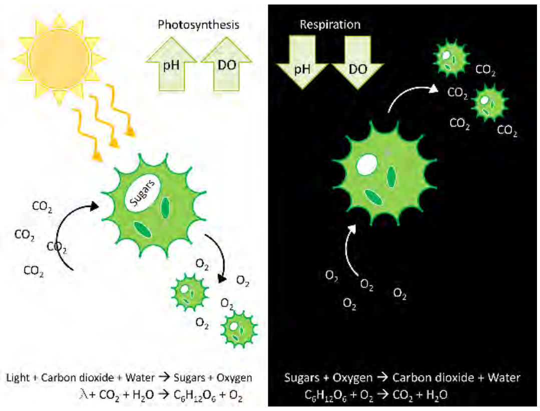


Figure 5-17: Algae Biochemistry

Algae in the final effluent contribute to TSS. Algae prefer to remain in the upper two to three feet of water depth where sunlight penetration is greatest. They are buoyant and can maintain their position in the water column. Most reservoirs and wastewater treatment lagoons minimize discharge of algae simply by withdrawing water from deeper in the water column. Northglenn employs this operational strategy when possible; however, as water levels in Bull Reservoir fall below 10 feet, this is no longer a viable strategy.

The algae in the reservoir may also complicate compliance with future TP limits. The presence of algae can perpetuate or create anoxic conditions and enhance the subsequent release of phosphorus into the water. Several management practices could be implemented to control internal loading of phosphorus. If the practices are not effective, effluent from Bull Reservoir could be directed back through the tertiary treatment system or an additional system to serve Bull Reservoir could be constructed.

Possible alternatives for algae control include:

- **Daphnia:** Daphnia are natural predators of single-celled algae. They may be purchased from aquarium stores and be added directly to the reservoir. They can be effective in controlling algae but must be continuously replenished as they themselves are food for fish. They will not eat blue-green algae, also known as cyanobacteria, which can limit their effectiveness. Daphnia can also contribute to effluent TSS.

- **Copper Sulfate Addition**: Copper sulfate is an algicide that has been successfully used at many facilities to control algae. Copper concentrations between 25 and 40 ug/L have been reported to be effective in controlling certain types of algae in lakes. Application rates between 5.4 and 100 lb/acre have been utilized. It would need to be applied at least four times per year to keep the copper concentration in the upper layer of water in the desired range. The discharge permit does not currently contain a limit for copper; however, CDPHE would add a limit if copper sulfate were added to the treatment process. Based on the water quality analysis conducted by CDPHE, we estimate dissolved copper limits would be 27 ug/L (chronic) and 45 ug/L (acute). Copper can also interfere with whole effluent toxicity testing. Addition of copper sulfate is not recommended.

- **Barley Straw**: Barley straw is an effective algistat but is not an algicide. An aligstat slows or halts the growth of existing algae and can prevent the appearance of new cells. An algicide kills existing algae. Copper sulfate is an algicide. As barley straw breaks down, it releases phenols and other compounds that react with sunlight. Barley straw is typically applied at a rate of 300 pounds per acre. The application rate may need to be higher for Bull Reservoir due to its depth. Barley straw should be applied as soon as the reservoir surface shows open water in the spring. Barley straw may be wrapped in snow fencing or similar material to create booms as shown in **Figure 5-18**. Sealed, empty, plastic milk jugs or similar container should be incorporated into the booms to help keep the barley straw on the surface of the reservoir. New booms should be added before existing booms decay completely. Three to four applications per year are typical.

Figure 5-18: Barley Straw

- **Reducing Phosphorus Entering Bull Reservoir**: Algae blooms typically occur when phosphorus concentrations exceed 0.1 mg/L as P. It may be possible to limit algae growth in Bull Reservoir by further reducing effluent phosphorus concentrations prior to discharge into Bull Reservoir. Phosphorus may be reduced to 0.1 mg/L with a combination of chemical addition, either ferric chloride or alum, and tertiary treatment. Tertiary treatment options are addressed in Section 5.5.1.

Bull Reservoir has been in service since 1982. Solids have accumulated in the bottom of the reservoir. These solids likely contain some amount of phosphorus. It's possible that reducing the amount of phosphorus going into Bull Reservoir won't reduce algae concentrations to the degree the City desires. Rather, phosphorus may be recycled between the solids on the reservoir floor and algae at the top of the reservoir.

- **Alum Addition:** Phosphorus inactivation by aluminum sulfate or alum addition to lakes is the most widely-used technique to control internal phosphorus loading. Iron and calcium compounds can also be used, but alum is the most common metal salt employed. Alum forms a polymer that binds phosphorus and organic matter. The aluminum hydroxide-phosphate complex (commonly called alum floc) is insoluble and settles to the bottom, carrying suspended and colloidal particles with it. Phosphorus in the upper sediment layer is also complexed, reducing phosphorus release from the sediment. This technique is most effective after nutrient loading into the water body is reduced. Aluminum doses between 1 and 5 mg/L have been used to strip phosphorus out of the water column with limited effects on pH or other water quality variables. Mixing with aeration systems can increase treatment efficiency and lower the necessary Alum addition can be toxic to fish and invertebrates and can cause fluctuations in pH. The discharge permit does not currently contain a limit for aluminum; however, the Division would likely add a limit if alum were added to the treatment process.

- **Aeration and/or Mixing**: Algae must remain in the top few feet of water where sunlight penetrates in order to grow and reproduce. Floating mixers can be used to prevent algae from remaining in this zone or the reservoir could be aerated from below. Completely mixing a reservoir requires around 40 horsepower of mixing energy per million gallons. Given the size of Bull Reservoir, this alternative is cost prohibitive and not recommended.

- **Dredging:** Phosphorus release from the sediment is greatest from recently deposited layers. Dredging approximately one meter of recently deposited phosphorus–rich sediment can remove approximately 80 to 90 percent of the internally loaded phosphorus without the addition of potentially toxic compounds to the reservoir. Dredging may also contribute to reductions in internal phosphorus loading by increasing the depth of large portions of the waterbody, reducing the degree of reintroduction of sediments into the water column through physical mixing. However, dredging is typically more costly than other management options.

- **Sonication**: Several manufacturers provide sonic ultrasound devices for controlling algae growth. These devices use low-power ultrasound at high frequencies to disrupt the ability of algae to remain in the upper layers of the reservoir, thereby depriving them of sunlight. Eventually, the algae die and sink to the bottom of the reservoir where they decompose. The frequencies used are beyond human hearing. Manufacturers claim these devices are not harmful to aquatic plants, fish, and other pond life. Individual treatment systems have a range up to 1,600 feet in diameter. Multiple units are typically used with overlapping treatment zones since line of sight is required. Some other considerations related to ultrasonic algae treatment include: it only works on specific types of algae, it requires significant contact time (days), it does not reduce in-lake nutrient concentrations, and the equipment will likely have to be pulled if water will freeze. Most reported examples of successful installations are on smaller-scale applications, but there are some larger installations around the world. This could be a tool used to control nuisance algae, but it would likely have to be used in combination with other techniques and is more expensive than other alternatives.

- **Shade Balls and Floating Covers**: Shade balls or floating covers may be used to limit the amount of sunlight entering Bull Reservoir. Without sunlight, algae cannot grow. A floating cover would be difficult to manage as Bull Reservoir operates with variable water levels. Shade balls are typically 6-inches in diameter and float on the reservoir surface. Los Angeles added 96 million shade balls to one of their drinking water reservoirs in an attempt to slow evaporation rates, but later removed them when they increased evaporation rates by absorbing sunlight and heating the water surface. This effect could impact Northglenn's ability to meet temperature limitations of a future discharge permit.

  Bull Reservoir has a maximum surface area of 140 acres when full. At an estimated cost of $0.40 per shade ball, it would cost approximately $12.4 million to completely cover the reservoir. Due to excessive cost, this option is not recommended.

Section 6

Alternatives Analysis

The treatment process alternatives considered to address capacity limitations and maintain compliance with current and near future regulatory requirements for TIN and TP were evaluated based on the estimated flows and loads presented in Section 5. All alternatives presented herein will achieve an effluent TIN of 10 mg/L, which meets the TIN limit of 14 mg/L N that will take effect in 2024 and meets the treatment goal of 10 mg/L N for the Nutrient Incentives Program. The next permit cycle is also likely to include a TIN limit of 10mg/L based on the Water Supply use designation for Segment 1 of Big Dry Creek. All alternatives considered will also achieve the current TP limit of 1 mg/L that went into effect at the beginning of 2021 with enhanced biological phosphorus removal (EBPR). A combination of biological and chemical phosphorus removal will likely be required to achieve the incentives program goal of 0.5 mg/L when alum residuals from the water treatment plant are no longer sent to the WWTP. A detailed alternatives analysis was not conducted on treatment technologies capable of achieving nutrient limits associated with the adoption of Regulation 31 as these limits should not be imposed within the 20-year planning period associated with this Plan. However, Section 5 presents the treatment options currently available for meeting the stringent nutrient limits associated with adoption of Regulation 31 nutrient criteria.

6.1 Review of Previous Master Plans

6.1.1 Alternatives Evaluated

6.1.1.1 Liquid Stream Alternatives

Between the 2003 and 2012 MPs, a wide variety of liquid stream process upgrades were evaluated to meet permit limits. In the 2003 MP, the following alternatives were evaluated to meet nitrogen and BOD_5 limits: modifications to the WWTP's existing aerated lagoons,3-Stage BNR, 5-Stage BNR, oxidation ditch, Landox oxidation ditch, sequencing batch reactor (SBR), and membrane bioreactor (MBR) processes. From these alternatives, the 3-Stage BNR was recommended and implemented due to its ability to meet strict effluent limits, ability to be modified to adapt to changing influent and effluent conditions, and established history of reliability with similar municipal wastewater applications. The 3-Stage BNR treatment process was originally designed to operate in conjunction with primary clarifiers. Instead of primary clarifiers, ponds that were part of the original treatment facility were repurposed to provide grit removal and primary clarification as described in Section 4.3.1.4.

The 2012 MP concluded that the ponds, referred to as aerated lagoons, located upstream of the aeration basins were inhibiting the BNR treatment process by removing much of the carbon source. Sufficient carbon is necessary for the heterotrophic bacteria to denitrify nitrate in the anoxic zone. Performance data presented in Section 4 demonstrates that sufficient carbon remained post-lagoon treatment to keep effluent nitrate-nitrogen concentrations below 10 mg/L N from January 2015 through mid-2017, when the lagoons were decommissioned in 2017. Personnel changes at the Northglenn WWTP after 2012 resulted in demonstrably increased data quality and improved process control, which may account for the discrepancy. The 2012 MP evaluated alternatives to increase treatment capacity assuming that the aerated lagoons would be decommissioned. These alternatives included: (1)

operating the activated sludge process without aerated lagoons or primary clarification and (2) constructing new primary clarifiers.

In addition, the 2012 MP evaluated modifications to the existing 3-stage BNR treatment process to meet potential future effluent limits. Northglenn anticipated a possible TIN limit of 10 mg/L N being applied if the designated use for Segment 1 of Big Dry Creek or downstream segments were changed to Water Supply. CDPHE has attempted to enforce drinking water standards as end-of-pipe permit limits in the past but has not been successful. To meet a new TIN permit limit of 10 mg/L and increase the capacity of the facility, the 2012 MP evaluated the following alternatives: modifying the anaerobic and anoxic zones to be swing zones to increase aerobic volume, upgrading the existing 3-stage BNR system with more identical treatment trains, converting the aerobic zones to an Integrated Fixed Film Activated Sludge (IFAS) system, and adding a third secondary clarifier. A third secondary clarifier was constructed in 2017.

To meet future phosphorus effluent limits, the 2012 MP evaluated adding effluent filtration, but did not evaluate chemical precipitation of phosphorus. Alternatives evaluated for effluent filtration included: traveling bridge sand media filters, traveling bridge cloth media filters, compressible media filters, continuous backwash sand filters, cloth media disk filters, and deep bed mono-media filters. Tertiary filtration is not evaluated in detail in this Plan but is discussed in combination with other treatment alternatives currently available for meeting the stringent nutrient limits associated with the adoption of Regulation 31 nutrient criteria. **Table 6-1** presents the relevant liquid stream alternatives from previous MPs.

Table 6-1: Liquid Stream Alternatives Evaluated from Previous MPs

Process/Category	Sources	Purpose	Evaluated in this Plan
Biological Nutrient Removal			
3-Stage BNR	2003 MP (page 8-12)	Nitrification/Denitrification, Biological Phosphorus Removal, BOD_5 Removal	Existing Condition
5-Stage BNR	2003 MP (page 8-12)	Nitrification/Denitrification, Biological Phosphorus Removal, BOD_5 Removal	No
Convert Aerobic to IFAS	2012 MP (page 4-62)	Nitrification/Denitrification, BOD_5 Removal	No
Primary Clarifiers	2003 (page 8-31) & 2012 MPs (page 4-50)	Treatment Loading Capacity Upgrades	Yes
Additional Activated Sludge Train	2012 MP (page 4-66)	Treatment Loading Capacity Upgrades	Yes
Secondary Clarifier Upgrades	2003 MP (page 8-32)	Treatment Loading Capacity Upgrades	Yes
Tertiary Filtration			
Traveling Bridge Sand Filter	2012 MP (page 4-68)	Phosphorus Removal	No
Traveling Bridge Cloth Filter	2012 MP (page 4-68)	Phosphorus Removal	No
Compressible Media Filter	2012 MP (page 4-68)	Phosphorus Removal	No
Continuous Backwash Filter	2012 MP	Phosphorus Removal	No

Process/Category	Sources	Purpose	Evaluated in this Plan
	(page 4-68)		
Cloth Media Disk Filter	2012 MP (page 4-68)	Phosphorus Removal	No
Deep Bed Mono Media Filter	2012 MP (page 4-68)	Phosphorus Removal	No

6.1.1.2 Solids Stabilization and Handling Alternatives

The 2003 MP investigated the following alternatives for **sludge thickening** prior to stabilization: gravity thickeners, gravity belt thickeners (GBTs), DAF, and thickening centrifuges. The 2003 MP assumed that both the primary sludge (PS) and WAS flows would be combined prior to thickening. Gravity belt thickeners and centrifuge alternatives were immediately eliminated because they were not ideal technologies for handling primary sludge. GBTs have the potential to produce potent odors when handling PS. Thickening centrifuges are an expensive and operator-intensive technology compared to other thickening technologies that can handle PS. The 2003 MP selected the DAF thickening alternative over the gravity thickener due to its ability to produce higher solids concentrations and because it produces fewer odors.

The 2003 MP investigated the following alternatives for **sludge stabilization** and evaluated their ability to produce class B biosolids: aerobic digestion, aerobic thermophilic digestion (ATAD), and anaerobic digestion. From these alternatives, the anaerobic digestion process was recommended after a qualitative and cost analysis due to its combined process benefits and flexibility to be upgraded to produce class A biosolids.

For **sludge dewatering**, two alternatives were investigated in the 2003 MP: centrifuge and belt filter press. The centrifuge alternative was recommended due to its lower operation costs, its lower potential to produce odors, and because it is cleaner to operate. Centrifuges have lower operating costs because they produce higher percent solids, reducing the volume and weight of cake that needs to be handled, thereby reducing shipping costs.

In the 2012 MP, two general alternatives were evaluated for solids handling. The first, acting as a no action alternative, was to maintain solids handling lagoons for sludge stabilization and land application of biosolids to adjacent land. The second was meant to be in conjunction with construction of primary clarifiers. It was to construct sludge thickening, digestion, and dewatering systems. The second alternative allows for the sludge storage lagoons to be decommissioned, which is beneficial because the lining in the lagoons is in poor condition and because it would eliminate the lagoons as a major odor source. The second alternative looked exclusively at gravity thickeners for PS thickening, rotary drum thickeners (RDTs) for WAS thickening, two-stage mesophilic anaerobic digestion for stabilization, and rotary screw presses for dewatering. The technologies evaluated from the previous MPs for the thickening, digestion, and dewatering processes are summarized in **Table 6-2**.

Table 6-2: Solids Handling Technologies Evaluated from Previous MPs

Process/Category	Sources	Purpose	Evaluated in this Plan
Solids Thickening			
DAF	2003 MP (page 9-25)	Solids Thickening Pre-Digestion	Yes
Gravity Thickener	2003 MP (page 9-25)	Solids Thickening Pre-Digestion	No
Rotary Drum Thickener	2012 MP (page 4-77)	Solids Thickening Pre-Digestion	Yes
Digestion			
Aerobic Digestion	2003 MP (page 9-25)	Stabilization of Solids	Yes
Anaerobic Digestion	2003 MP (page 9-25)	Stabilization of Solids	Yes
Two Stage Anaerobic Digestion	2012 MP (page 4-78)	Stabilization of Solids	No
ATAD	2003 MP (page 9-25)	Stabilization of Solids	No
Dewatering			
Rotary Screw Press	2012 MP (page 4-72)	Reduce Weight and Volume of Solids for Disposal	Yes
Belt Filter Press	2003 MP (page 9-25)	Reduce Weight and Volume of Solids for Disposal	No
Centrifuge	2003 MP (page 9-25)	Reduce Weight and Volume of Solids for Disposal	Yes
Modifications to Existing Solids Handling Processes			
Decommission Aerated Lagoon System	2012 MP (page 4-72)	Remove Failing Infrastructure	Yes

6.1.2 Process Improvements Implemented

Three-Stage BNR was implemented subsequent to the 2003 MP due to its ability to meet strict effluent limits, its ability to be modified to adapt to changing influent and effluent conditions, and its established history of reliability with similar municipal wastewater applications. In addition, two of the original treatment ponds were repurposed to serve as headworks and primary clarifiers as described in Section 4.3.1.4.

The 2012 MP concluded that using the ponds for primary treatment was inhibiting BNR since too much carbon was being removed upstream of the aeration basins. The ponds were decommissioned and filled in 2017 and a third secondary clarifier was constructed to partially account for the higher loads being sent to secondary treatment. Additional upgrades implemented in 2017 include: a new headworks facility with influent screening and grit removal and installation of a secondary generator.

No significant improvements to the solids handling process have been implemented. Waste activated sludge is still directed to the solids handling lagoons for stabilization. Class B solids are produced and land-applied to adjacent land.

6.2 Alternatives Evaluation Approach/Framework

All alternatives considered were evaluated on a non-economic basis. Two overall alternatives were evaluated on both a non-economic and economic basis.

6.2.1 Non-Economic Evaluation Criteria

Each treatment alternative was evaluated based on a series of non-economic criteria. A non-economic evaluation is more subjective in nature than an economic evaluation. Non-cost factors and the relative importance of those factors vary by agency and by project. The non-cost, qualitative criteria considered for this evaluation were developed in conjunction with Northglenn WWTP staff and are listed below.

1. **Feasibility**- Considers the practicality of implementation within the treatment process and construction considering site specific conditions.
2. **Compliance with Future Discharge Limits**- Considers the ability for technology/alternative to reliably comply with future discharge limits and its impact on the overall process related to the facility's effluent limit compliance.
3. **Ease of Operation-** Considers the amount of attention required by the operator as well as the relative complexity of operations and level of training required.
4. **Operating Experience-** Considers how many proven relevant industry applications the technology has and the reliability of those applications**.**
5. **Reliability and Redundancy-** Considers the ability for the technology/alternative to consistently perform under variable operating conditions. Considers the complexity of the process as well as the number of potential fail points and redundancies.
6. **Flexibility-** Considers the ability to adapt the mode of operation of the technology to compensate for unforeseen future conditions. Changes in future conditions may include varied loadings, constitution of feed water, or changes in regulations. This includes the relative ease of expanding the process.
7. **Maintenance-** Considers the level of specialized expertise required to maintain the equipment. Are facility operators capable of performing most repairs, or is a factory representative required? Also includes the availability of spare parts.
8. **Health & Safety-** Considers the relative safety hazards of operating the equipment from factors including but not limited to: high voltage power, operation of heavy equipment, use of hazardous chemicals, exposed moving parts, and air quality risks.
9. **Electrical & Fire Code Impacts-** Considers the National Electrical Code (NEC) which requires use of explosion proof components for the electrical system in anaerobic digestion facilities.

10. **Odor-** Considers the potential for alternatives to produce odors as well as the potential impact of those odors on both staff and surrounding neighbors.

11. **Material Handling & Transportation-** Considers the quantity of solids residuals produced by each technology as well as the solids concentration achievable in the residuals. Also considers the quantity of chemicals (polymers for example) required to condition the residual solids. All these factors have an impact on the ease of handling, transportation, and disposal.

12. **Energy Efficiency-** Considers energy efficiency which is important for mitigating the impact on energy costs as well as environmental air emissions.

Each criterion was assigned a weight value by Northglenn staff according to its relative importance. Each criterion also was assigned either a positive (+), neutral (o), or negative (--) scoring value. If a positive score is awarded, the weight value for the criteria is added to the total score. If a neutral score is awarded, nothing is added to the total score. If a negative score is awarded, the weight value for the criteria is subtracted from the total score.

6.2.2 Economic Evaluation Criteria

The estimated capital expenditure associated with each overall alternative is provided as a planning-level program cost, consisting of both construction and implementation costs. The construction cost is provided as a planning-level opinion of probable construction costs (OPCC) including both direct and indirect costs, as well as a construction contingency and escalation to the assumed mid-point of construction.

Specifically, OPCCs presented in this Plan include:

- Direct costs:
  - Major equipment, equipment skids, and tank purchase costs based on information obtained from selected equipment vendors
  - Materials (e.g. piping, concrete) purchase costs
  - Mobilization, site work, and minor building improvements
  - Allowances for electrical work and instrumentation and controls
  - Labor costs
- Indirect costs:
  - Building permits, bonding, insurance, sales tax, builder’s risk insurance
  - Contractor’s field general conditions, overhead and profit
- Construction contingency: 30% of the sum of direct and indirect costs
- Escalation of the construction total cost (including direct costs, indirect costs, and construction contingency) to January 2025 costs at 3% per year (3.75 years)

Program costs include:

- The OPCC, as described above
- Project contingency: 20% of OPCC

- Allowance for engineering and implementation, including design, construction services, and equipment startup and testing: 20% of the sum of OPCC and project contingency

The costs presented for the different alternatives are considered order-of-magnitude Class 4 estimates, with an anticipated accuracy of -15 to -30 percent and +20-50%, per AACE International. Therefore, alternatives that fall within this range relative to each other are considered equal.

6.2.3 Facility Capacity Limitation Alternatives Considered

Alternatives from previous MPs listed in Table 6-1 were narrowed based on technical feasibility and applicability as well as through discussions in workshops with Northglenn. The facility already produces effluent with nutrient concentrations (TIN and TP) within the permit limits for the 20-year planning horizon and within the targets for the Nutrient Incentives Program. Therefore, the focus of this alternatives analysis is to increase treatment capacity to allow the BNR process to continue to achieve similar effluent quality under increasing loads.

Upgrading the existing BNR tankage from a 3-stage to a 5-stage process would help improve nitrogen and phosphorus removal performance but it would not provide additional BOD_5 removal and nitrification capacity. Upgrading the BNR system to an IFAS process would increase treatment capacity, but implementation would require a complete overhaul of the existing fine bubble diffuser system, replacing them with coarse bubble diffusers, as well as numerous other mechanical and physical upgrades such as media retention screens, air scour system, mixers and any structural modifications to implement IFAS. In general, when the space is available for new trains, constructing new BNR trains is more cost-effective than implementing IFAS in existing trains. IFAS is not considered a preferable alternative for Northglenn since space for new treatment trains is available.

The two alternatives identified for achieving increased treatment capacity include:

- **Process Capacity Alternative #1:** constructing primary clarifiers in addition to new 3-stage BNR trains and secondary clarifiers
- **Process Capacity Alternative #2:** adding 3-stage BNR trains and secondary clarifiers without primary clarifiers. It is assumed that the new BNR trains will have the same configuration and dimensions as the existing trains

Solids handling improvements are also evaluated in this alternatives evaluation. The two alternatives include:

- **Solids Handling Alternative #1:** maintain the existing solids handling lagoons for sludge stabilization
- **Solids Handling Alternative #2:** replace the lagoons with new thickening, stabilization and dewatering processes

The 2003 and 2012 MPs had different assumptions for evaluating solids thickening processes prior to sludge stabilization. The 2003 MP assumed that the primary sludge would be combined with the WAS prior to being thickened. While it is common for facilities to thicken PS by developing a sludge blanket at the bottom of the clarifiers rather than through a mechanical thickening process, the 2003 MP argues that this would potentially produce odors in the primaries. Experience at other WWTPs indicates that odors would be worse by combining PS and WAS upstream of a thickening process than allowing sludge to thicken in the primary clarifiers. In addition, odor mitigation at the primary clarifiers can be implemented by covering the effluent channel and/or weirs where the turbulence of the overflow has the potential to release odors. Using the clarifiers to thicken primary sludge also requires less process tankage and/or equipment since the thickening process only needs to be sized to accommodate the WAS and not the combined flow. The 2012 MP assumes that a separate thickening process would be designed for both PS and WAS. This approach requires the most process tankage and equipment and therefore has the highest capital costs. For this Plan it was assumed that the PS would be thickened in the primaries and that a thickening process would be added for WAS only.

Solids stabilization alternatives from previous MPs were also narrowed for this analysis based on performance, technical feasibility, and discussions in workshops with Northglenn WWTP staff. The ATAD alternative was eliminated due to its potential to produce odors. Aerobic digestion is commonly used at WWTPs with a capacity of less than 5 mgd. It relies on endogenous respiration to stabilize the solids and requires active microorganisms (2003 MP). Aerobic digestion is therefore effective at stabilizing WAS but not PS. Aerobic digestion is the recommended stabilization process for the facility if primaries are not installed. Anaerobic digestion is the preferred stabilization process for WWTPs with flows greater than 5 mgd, although it is sometimes used at smaller facilities due to its process benefits (2003 MP). Examples of smaller facilities with anaerobic digesters in Colorado include the City of Brush at 1.6 mgd and the City of Durango at 2.5 mgd. Anaerobic digestion operates the most efficiently when solely fed PS, however, most facilities feed both PS and WAS to the digesters. Anaerobic digestion is the recommended stabilization process for the facility if primaries are installed. Note that the primary clarifiers could be constructed prior to the anaerobic digesters; however, sending PS to the solids handling pond will increase odor generation and is therefore not recommended. Building anaerobic digesters without primary clarifiers also is not recommended as anaerobic digesters operated with WAS only are unstable and may not be able to meet minimum volatile solids reduction requirements in Regulation 84 and 40 CFR Part 503.

The alternatives analysis considers the two options for increasing liquid treatment capacity in conjunction with multiple options for solids handling improvements. The overall process upgrade alternatives evaluated are summarized below and shown in **Figure 6-1**. Section 6.3 includes a more detailed description of each alternative.

- **Process Capacity Alternative 1: Increase Treatment Capacity including Primary Clarifiers**
  - Construct two primary clarifiers
  - Construct two additional 3-stage BNR process trains
  - Construct one additional secondary clarifier
  - Consider solids handling upgrades
    - *Alternative 1a: keep solids handling lagoons*
    - *Alternative 1b: update solids handling process by replacing lagoons with WAS thickening, anaerobic digestion, and dewatering*

- **Process Capacity Alternative 2: Increase Treatment Capacity without Primary Clarifiers**
  - Do not construct primary clarifiers
  - Construct four additional 3-stage BNR trains
  - Construct one additional secondary clarifier
  - Consider solids handling upgrades
    - *Alternative 2a: keep solids handling lagoons*
    - *Alternative 2b: update solids handling process by replacing lagoons with WAS thickening, aerobic digestion, and dewatering*

Figure 6-2 shows the solids handling alternatives considered under each process capacity alternative. Section 6.3 includes a more detailed description of the solids handling alternatives.

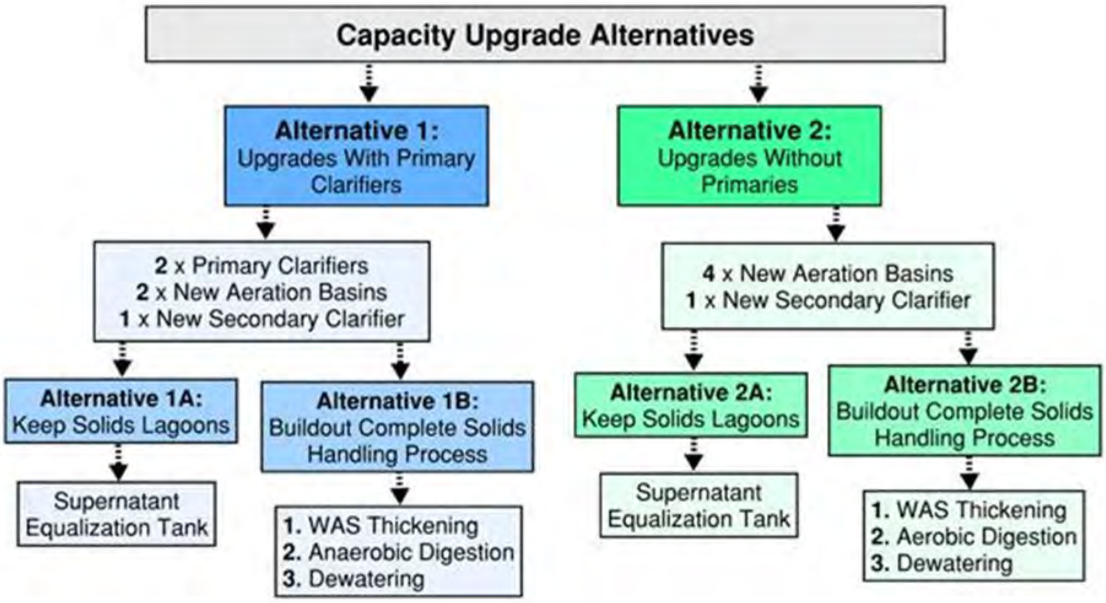


Figure 6-1: Process Capacity Alternatives

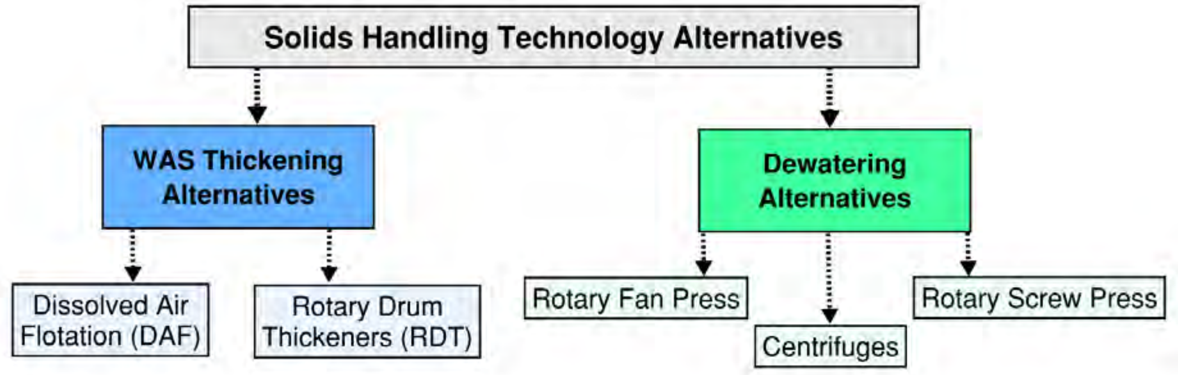


Figure 6-2: Solids Handling Alternatives

6.3 Description of Alternatives

6.3.1 Description of Liquid Process Alternatives to Increase Capacity

6.3.1.1 Alternative 1: Increase Treatment Capacity including Primary Clarifiers

Primary clarification is a sedimentation process designed to reduce the organic and inorganic solids loading to the secondary treatment processes. Adding primary clarifiers would reduce the BOD_5 loading to the BNR process by around 30% and the TSS load by about 50%. Primary clarifiers can only remove settleable solids and organics and cannot remove soluble components such as ammonia and ortho-phosphorus. By reducing the loading to the BNR process, less tankage in the aeration basins is required to meet capacity. The aeration demand in the BNR process would also be reduced, providing significant energy cost savings.

Primary clarifiers are typically paired with anaerobic digesters. Primary sludge sent to an anaerobic digester breaks down to form methane, which is captured and used to heat the digester. Sending primary sludge to an aerobic digester would require the same amount of air to stabilize it as if the influent organic matter were sent directly to the activated sludge process. The benefits of pairing a primary clarifier with an activated sludge process is that grease and scum would mostly be removed in the primary clarifier and fewer inert solids would pass into the activated sludge process. Removing grease in the primaries increases activated sludge process stability and decreases foaming events due to nocardioforms. Primary clarifiers may also be intentionally operated with several feet of primary sludge accumulated in the blanket to generate soluble BOD_5 and volatile fatty acids to enhance biological phosphorus removal and denitrification. These benefits do not typically offset the costs of constructing primary clarifiers and ongoing pumping costs and equipment maintenance.

A desktop analysis was performed to determine the treatment upgrades required under future maximum month flow and loading conditions. **Table 6-3** presents the design criteria for the primary clarifiers, 3-stage BNR trains, aeration system, secondary clarifiers, and RAS and WAS pumps, respectively.

Calculations indicate that two circular 60-ft diameter primary clarifiers would be sufficient to maintain surface overflow rates in an acceptable range. Two additional BNR trains, for a total of five, would be required to maintain existing treatment performance under 2040 flows and loads. This is based on maintaining a design aerobic SRT of 16.8 days at an allowable MLSS of 3,000 mg/L. An MLSS of 3,000 mg/L can be maintained at future maximum day flows if one additional 65-foot secondary clarifier is constructed for four secondary clarifiers total. No modifications to the blowers are required since there is ample aeration capacity to supply the maximum day air demand of approximately 8,000 scfm with the largest unit out of operation. Three additional RAS pumps are proposed to provide a firm pumping capacity of 150% of maximum month flow. The RAS common header will also need to be upgraded to accommodate the increased capacity. No additional WAS pumps are required to handle future estimated WAS flow rates.

Table 6-3: Alternative 1 Design Criteria

Parameter	Unit	Value
Primary Treatment		
Peak Hour (PH) Design Flow	mgd	10.5
Max Month Design Flow	mgd	5.78
Design Surface Overflow Rate at PH	gal/ft2	1870
Design Surface Overflow Rate at MM	gal/ft2	1020
Max Month TSS	ppd	16,300
Max Month BOD5	ppd	11,800
Primary Clarifiers Number Type Diameter Side Water Depth	 # ft ft	 2 Circular 60 10
TSS Removal Rate	%	50
BOD Removal Rate	%	30
Sludge Pumps Number Type Capacity, each	 # gpm	 2 Progressive Cavity 75
Scum Pumps Number Type Capacity, each	 # gpm	 2 Rotary Lobe 20
Secondary Treatment – 3-Stage Biological Nutrient Removal		
Max Month Design Flow	mgd	5.78
Max Month TSS	Ppd	8,150
Max Month BOD_5	Ppd	8,260
Max Month TKN-N	Ppd	2,650
Estimated Net Yield	lb/lb/d	1.08
Design MLSS Concentration	mg/l	3,000
Design Aerobic SRT	days	16.8
Aeration Basins		
Basins		

Parameter	Unit	Value
Number (Existing + Future) Type		3+2 3-Stage BNR
Anaerobic Zone (each Basin) Volume # Mixers Mixer HP each	 mg # HP	 0.127 1 8.3
Anaerobic Zone (each Basin) Volume # Mixers Mixer HP each	 mg # HP	 0.24 2 8.3
Aeration Zone (each Basin) Min DO Concentration Volume	 mg/l mg	 2.0 0.64
IMLR Pumps Number (Existing + Future) Type Pump HP, each Capacity, each Capacity, duty total % of MM Design Flow	# HP gpm mgd MM Flow/Duty IMLR Capacity	3+2 Centrifugal 25 4,200 18.1 500%+
Diffusers Diffuser Type Diffuser Size, diameter Number of diffusers per bank Number of banks per aerobic zone Min/Max airflow per diffuser	 in # # scfm	 Fine bubble 9 300 3 0.67/2.29
Aeration Blowers		
Number (Existing + Future)		3+0 (2 duty, 1 standby)
Type		Integrally-geared centrifugal
Capacity Blowers No. 1 and No. 2	scfm	4,000
Capacity Blower No. 3	scfm	6,000
Total Firm Capacity	scfm	10,000
Motor Blowers No. 1 and No. 2	HP	250
Motor Blower No. 3	HP	350
Secondary Clarifiers		
PH Design Forward Flow	mgd	10.5
MM Design Forward Flow	mgd	5.78
RAS Ratio (RAS Flow/Influent MM Flow)	%	150
Design MLSS	mg/l	3,000
Solids Loading at PHF	ppd	450,000
Design Solids Loading Rate at ADF	lb/day/ft^2	13.33
Design Solids Loading Rate at PHF	lb/day/ft^2	33.8
Design Surface Overflow Rate at ADF	gpd/ft^2	315
Design Surface Overflow Rate at PHF	gpd/ft^2	795
Clarifiers Number (Existing + Future)	 #	 3+1

Parameter	Unit	Value
Type Diameter Side Water Depth	 ft ft	Circular 65 14
Scum Pumps Number (Existing + Future) Type Capacity, each	 # gpm	 2+2 Progressive Cavity 90
RAS Pumps		
Number (Existing+Future)		3+3 (5 duty, 1 standby)
Required Flow Total	gpm (mgd)	6150 (8.86)
Type		Horizontal centrifugal chopper
Capacity (each)	gpm	1400
Capacity (each)	mgd	2.02
Capacity Total Duty Pumps	gpm	7,000
Motor HP	HP	20
Head	ft	30.3
WAS Pumps		
Number (Existing+Future)		2+0 (1 duty, 1 standby)
Required Flow Total	gpm (mgd)	94
Type		Centrifugal
Capacity (each)	gpm	140
Capacity (each)	mgd	0.20
Capacity Total Duty Pumps	gpm	140
Motor HP	HP	5
Head	ft	54

6.3.1.2 Alternative 2: Increase Treatment Capacity without Primary Clarifiers

This alternative evaluates upgrading the secondary treatment process to meet 2040 capacity requirements without adding primary clarifiers. The capacity shortfall would be met with four additional 3-stage BNR treatment trains and an additional secondary clarifier.

A desktop analysis was performed to determine the treatment upgrades required under future max month flow and loading conditions. **Table 6-4** presents the design criteria for the aeration basins, aeration system, secondary clarifiers, and RAS and WAS pumps.

Calculations indicate that four additional BNR trains, for a total of seven, would be required to maintain existing treatment performance under 2040 flows and loads. This is based on maintaining a design aerobic SRT of 16.8 days at an allowable MLSS of 3,000 mg/L. An MLSS of 3,000 mg/L can be maintained at future maximum day flows if one additional 65-foot secondary clarifier is constructed. No modifications to the blowers are required since there is ample aeration capacity to supply the maximum day air demand of approximately 8,000 scfm with the largest unit out of operation. Three additional RAS pumps are proposed to provide a firm pumping capacity of 150% of maximum month flow. The RAS common header will also need to be upgraded to accommodate the increased capacity. One additional WAS pump is required to handle future estimated WAS flow rates with one unit out of service.

Table 6-4: Alternative 2 Design Criteria

Parameter	Unit	Value
Secondary Treatment – 3-Stage Biological Nutrient Removal		
Max Month Design Flow	mgd	5.78
Max Month TSS	ppd	16,270
Max Month BOD_5	ppd	11,800
Max Month TKN-N	ppd	2,650
Estimated Net Yield	lb/lb/d	1.08
Design MLSS Concentration	mg/l	3,000
Design Aerobic SRT	days	16.8
Aeration Basins		
Basins Number (Existing + Future) Type		 3+4 3-Stage BNR
Anaerobic Zone (each Basin) Volume # Mixers Mixer HP each	 mg # HP	 0.127 1 8.3
Anaerobic Zone (each Basin) Volume # Mixers Mixer HP each	 mg # HP	 0.24 2 8.3
Aeration Zone (each Basin) Min DO Concentration Volume	 mg/l mg	 2.0 0.64
IMLR Pumps Number (Existing + Future) Type Pump HP, each Capacity, each Capacity, duty total % of MM Design Flow	# HP gpm mgd MM Flow/Duty IMLR Capacity	3+4 Centrifugal 25 4,200 18.1 730%+
Diffusers Diffuser Type Diffuser Size, diameter Number of diffusers per bank Number of banks per aerobic zone Min/Max airflow per diffuser	 in # # scfm	 Fine bubble 9 300 3 0.67/2.29
Aeration Blowers		
Number (Existing+Future)		3+0 (2 duty, 1 standby)
Type		Integrally-geared centrifugal
Capacity Blowers No. 1 and No. 2	scfm	4,000
Capacity Blower No. 3	scfm	6,000
Total Firm Capacity	scfm	10,000
Motor Blowers No. 1 and No. 2	HP	250
Motor Blower No. 3	HP	350
Secondary Clarifiers		

Parameter	Unit	Value
PH Design Forward Flow	mgd	10.5
MM Design Forward Flow	mgd	5.78
RAS Ratio (RAS Flow/Influent MM Flow)	%	150
Design MLSS	mg/l	3,000
Solids Loading at PHF	ppd	450,000
Design Solids Loading Rate at ADF	$lb/day/ft^2$	13.33
Design Solids Loading Rate at PHF	$lb/day/ft^2$	33.8
Design Surface Overflow Rate at ADF	gpd/ft^2	315
Design Surface Overflow Rate at PHF	gpd/ft^2	795
Clarifiers Number (Existing + Future) Type Diameter Side Water Depth	 # ft ft	 3+1 Circular 65 14
Scum Pumps Number (Existing + Future) Type Capacity, each	 # gpm	 2+2 Progressive Cavity 90
RAS Pumps		
Number (Existing+Future)		3+3 (5 duty, 1 standby)
Required Flow Total	gpm (mgd)	6150 (8.86)
Type		Horizontal centrifugal chopper
Capacity (each)	gpm	1400
Capacity (each)	mgd	2.02
Capacity Total Duty Pumps	gpm	7,000
Motor HP	HP	20
Head	ft	30.3
WAS Pumps		
Number (Existing+Future)		2+1 (2 duty, 1 standby)
Required Flow Total	gpm (mgd)	160
Type		Centrifugal
Capacity (each)	gpm	140
Capacity (each)	mgd	0.20
Capacity Total Duty Pumps	gpm	280
Motor HP	HP	5
Head	ft	54

6.3.2 Description of Solids Handling Alternatives

Two main alternatives are considered for solids handling: 1) keep the existing solids handling lagoons with no thickening or dewatering processes and 2) update the solids handling processes with digestion, a thickening process for WAS, and a dewatering process to produce cake solids.

The need for primary clarifiers and the selection of thickening equipment is dependent on the solids stabilization method selected. **Figure 6-3** graphically demonstrates the interdependence of these technologies. The three potential stabilization methods are solids handling lagoons, aerobic digestion, and anaerobic digestion.

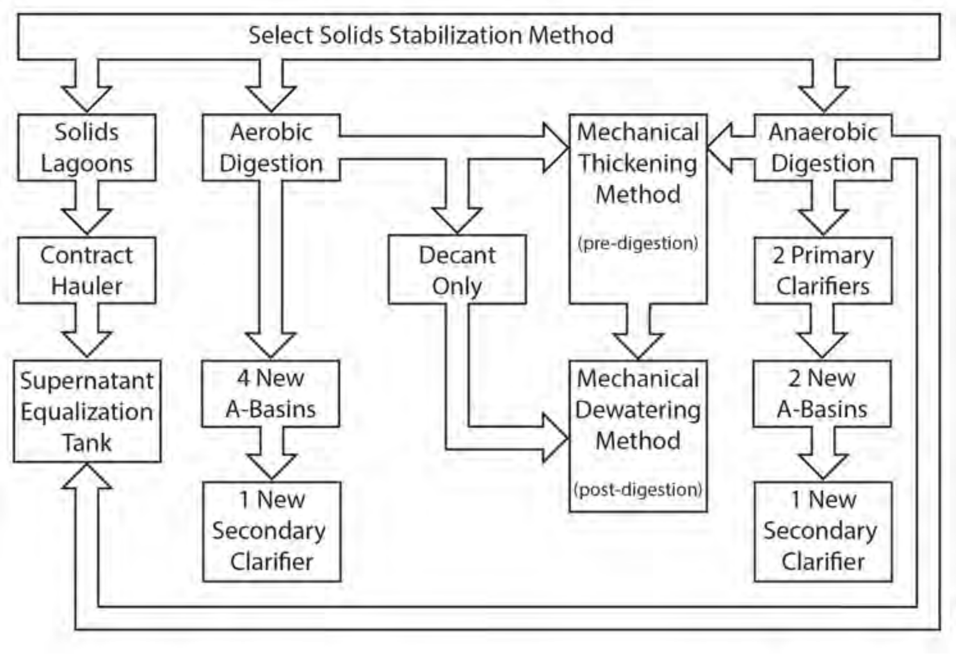


Figure 6-3: Solids Handling and Liquid Treatment Process Interdependencies

As discussed earlier, primary clarifiers are typically paired with anaerobic digesters. If the solids handling lagoons are kept or aerobic digestion is selected, primary clarifiers would not be needed. Adding primary clarifiers reduces the number of activated sludge treatment trains needed to meet future flows and loads.

Thickening is required prior to adding sludge to anaerobic digesters. Pre-thickening minimizes the amount of water sent to the digester, maximizes detention time, helps maintain a constant operating temperature, assists with buffering pH, and reduces the potential for process upsets. Pre-thickening is not required for either the solids handling ponds or aerobic digesters. Aerobic digesters may be manually decanted to remove excess water or sludge may be pre-thickened before it is added to the digester. Decanting has the potential to generate noxious odors. For that reason, this Plan has assumed pre-thickening for both aerobic and anaerobic digesters.

A flow equalization tank is needed with both solids handling lagoons and anaerobic digesters. The breakdown of solids in both these processes generates a high-ammonia side stream. The ammonia-nitrogen concentration in the solids handling pond decant can be several hundred milligrams per liter. Anaerobic digester supernatant can contain over one-thousand milligrams per liter of ammonia-nitrogen. If the ammonia is added back into the activated sludge process too quickly, it can overwhelm the process and result in ammonia bleed-through to the final effluent. This issue can be mitigated by collecting the decant in a flow equalization tank and then metering it into the secondary treatment process. The side stream may be returned over a 24-hour cycle or it can be used to even out the total NH_3 load by returning more of it during periods of low flow and load. Decant holding tanks are typically sized to hold one to two days of anticipated flow.

Aerobic digesters also produce ammonia, but it is converted to nitrite and nitrate during treatment making a decant equalization tank unnecessary.

Dewatering equipment should be selected to meet biosolids disposal/land application requirements. If the biosolids will be land applied to a site that is close to the treatment plant or is taken away by a contact hauler that charges according to dry weight, then achieving a high cake solids concentration is less important. In this situation, dewatering may not even be necessary. If the land application site is further away or if biosolids are taken away by a contract hauler that charges according to volume, Northglenn may benefit from producing the thickest cake possible. Equipment selection should balance the cost of dewatering against the cost of biosolids hauling.

6.3.2.1 Solids Stabilization

6.3.2.1.1 Alternative 1a and 2a: Keep Solids Handling Lagoons

These alternatives consider keeping the existing solids handling lagoons for solids stabilization. For Alternative 1a, both primary sludge from the new primary clarifiers and WAS would be pumped to the solids handling lagoons. For Alternative 2a, only WAS would be pumped to the lagoons like the current practice.

Solids stabilization is provided by the lagoons through long term storage and aerobic stabilization. This stabilization method comfortably achieves class B biosolids. The lagoons are typically dredged annually. With the projected loads through 2040, the lagoons may have to be dredged twice a year. The supernatant from the solids lagoons is recycled upstream of the secondary treatment process.

The lining in the solids lagoons is nearing the end of its useful life and will need to be repaired. In addition, the supernatant recycle flow creates slug ammonia loads into the secondary process. The BNR process does not have the capacity to accommodate the slug load; therefore, ammonia spikes can occasionally be seen in the plant's effluent. A supernatant equalization tank will need to be installed in order to allow consistent supernatant flows to be added to the BNR process throughout the day.

The concentration of the solids dredged from the lagoons is low, between 3 and 4%, since there are no thickening or dewatering processes on the finished biosolids. The city currently contracts with Veris Environmental for solids removal, and they charge on a dry ton basis. The finished biosolids are land applied to a combination of city-owned property adjacent to the WWTP and property owned by or contracted though Veris Environmental. Northglenn is evaluating the possibility of selling portions of Section 36 and disposing of all solids externally. It is common for disposal companies to charge for transport on a net weight basis as opposed to a dry ton basis. If future disposal rates are based on a net weight basis, shipping low concentration sludge may be very costly.

The solids handling lagoons produce substantial odors with just WAS being sent to them, particularly during dewatering and seasonal turnover events. The odor potential will increase if both primary sludge and WAS are sent to the lagoons, as proposed in Alternative 1a.

6.3.2.1.2 Alternative 1b: Anaerobic Digestion

If primary clarifiers are constructed in conjunction with new solids handling processes, anaerobic digestion is the recommended solids stabilization process. Primary sludge and thickened WAS would be combined upstream of the anaerobic digesters.

Mesophilic anaerobic digestion occurs in a gas tight, mixed reactor in the absence of oxygen at a constant temperature of around 95 degrees Fahrenheit. The process comprises three steps. First, organic compounds in the suspended solids are broken down into soluble compounds by extracellular bacterial enzymes. Next, the complex soluble compounds are decomposed into simple organic acids. Finally, the simple organic acids are broken down into methane and carbon dioxide. In conventional digesters, the entire process occurs in a single reactor. To optimize the process, two in-line reactors can be built: the first dedicated to forming the acids and the second to forming the gas. This can reduce the reaction time required for each step, increase the volatile solids destruction, increase the gas production, and reduce foam production.

The digester gas produced typically consists of approximately 60% methane (CH_4), 35% carbon dioxide (CO_2), 1% hydrogen (H_2), and 0.15% hydrogen sulfide (H_2S). Digester gas production for municipal WWTPs ranges from 12 to 18 cubic feet per pound of volatile solids destroyed. Digester gas produced by the anaerobic digestion process has an upper heating value of 600 btu/scf. The digester gas would be collected and used as fuel to heat the incoming sludge to 95 degrees Fahrenheit and replace heat losses to the environment.

Digester gas typically contains approximately 1,500 ppm of H_2S. Hydrogen sulfide is detrimental to the operation of gas burning equipment including boilers and engines. A sold scavenging reactant such as SulfaTreat or Iron Sponge would be used to remove H_2S from the gas prior to combustion. Surplus digester gas could be used to heat buildings, fuel engine driven equipment or generate electrical power. However, the capital cost of equipment needed to use the surplus gas as fuel usually exceeds the value of the gas for small treatment plants, like the Northglenn WWTP. It is assumed therefore, that excess gas will be burned in a waste gas flare. (2003 MP)

Table 6-5 presents the design criteria for the anaerobic digesters estimated from the desktop analysis. The digesters were sized based on max month loading conditions.

Table 6-5: Anaerobic Digesters Design Criteria

Parameter	Units	Anaerobic Digesters (Alt 1b)
Digester Tanks	#	2
Diameter	ft	80
Digester Depth	ft	20
Total Volume	mg	1.5
Operating Temperature	deg F	95
SRT	days	15
Volatile Solids Destruction	%	45
Feed Pumps	#	2
Feed Pump HP	HP	10
Digester Gas Production	scf/d	82,400
Gas Compressors	#	2
Gas Compressors HP each	HP	25
Digested Sludge Pumps	#	3
Digested Sludge Pumps HP each	HP	30
Heat Exchangers	#	2
Gas Flares	#	1

6.3.2.1.3 Alternative 2b: Aerobic Digestion

If primary clarifiers are not added at the facility but new solids handling processes are constructed, aerobic digestion is the recommended stabilization process. Waste activated sludge would be thickened upstream of the aerobic digesters.

The aerobic digestion process consists of an aerated mixed vessel with a solids retention time of 60 days at a minimum temperature of 60 degrees Fahrenheit. Volatile solids reduction in aerobic digesters ranges from 35 to 50%.

Aerobic digestion stabilizes waste sludge by endogenous respiration. The microorganisms in the sludge deplete the available food sources and consume their own protoplasm to provide energy for cellular maintenance. Because the process relies on endogenous respiration, it is best suited to treat WAS.

The aerobic digestion process is unheated, but temperatures below 60 °F can inhibit the process, therefore the digester should be covered to mitigate the effects of temperature during Colorado winters. Blowers and digested sludge pumps would be housed in an equipment building adjacent to the digesters. (2003 MP)

Table 6-6 presents the design criteria for the aerobic digesters estimated from the desktop analysis. The digesters were sized based on max month loading conditions. Aerobic digesters may be round, square, or rectangular. The design criteria included in Table 6-6 assume circular digesters; however, square or rectangular digesters of equivalent volume could also be constructed. The overall footprint of the digesters may be reduced by increasing liquid depth. Centrifugal blowers are typically limited to water depths of about 12 feet; however, positive displacement blowers are compatible with liquid depths over 20 feet.

Table 6-6: Aerobic Digesters Design Criteria

Parameter	Units	Aerobic Digesters (Alt 2b)
Digester Tanks	#	2
Diameter	ft	160
Digester Depth	ft	12
Influent Solids Concentration	%	0.8
Total Volume	mg	6.0
Operating Temperature	deg F	68
SRT	days	60
Volatile Solids Destruction	%	40
Feed Pumps	#	2
Feed Pump HP	HP	10
Blowers[1]	#	3
Blowers HP each	HP	300
Digested Sludge Pumps	#	3
Digested Sludge Pumps HP each	HP	

[1]Blower sizing is based on 2003 master plan.

6.3.2.2 WAS Thickening Technologies

WAS would be thickened upstream of stabilization for both alternatives employing digestion for its stabilization process: *Alternatives 1b and 2b*. Two WAS thickening options were considered: dissolved air flotation thickeners and rotary drum thickeners. As noted previously, it is assumed that primary sludge will be thickened in the bottom of the primary clarifiers, so the thickening technologies are sized based on WAS production only.

6.3.2.2.1 Dissolved Air Flotation

A DAF separates and removes solids by injecting extremely fine air bubbles which adhere to the solids and suspend them at the top of the tank (**Figure 6-4**). The solids are then either skimmed or they overflow to a hopper where they are removed. To create the fine bubbles, air is pulled by a vacuum into a pressure chamber operating between 40-100 psi. The water becomes saturated with air under the high pressure. As the water enters the bottom of the DAF main tank, it depressurizes causing the air to transfer back to a gaseous state and the fine bubbles are formed. Polymer is typically added to promote the formation of floc to improve solids removal efficiency.

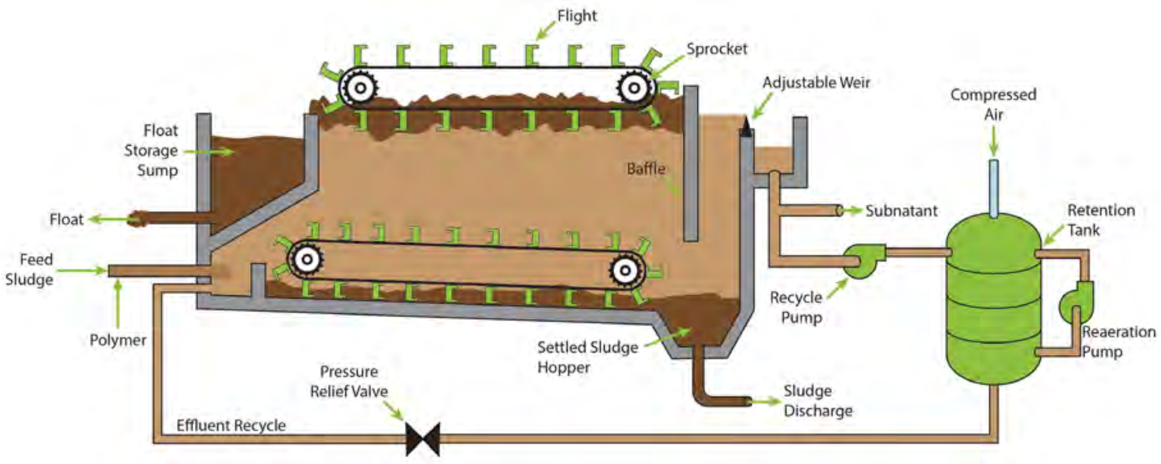


Figure 6-4: Schematic of a Dissolved Air Floatation Thickener

The DAF process runs continuously. Changing hydraulic loading has a minimal effect on DAF performance, therefore, wasting rates can be changed without effecting the thickening process. The DAF process is relatively simple and has few points of failure. If polymer supply is unavailable, then the process can continue to operate with slightly reduced capture efficiency. The primary point of failure in a DAF system is if the air saturation tank vacuum pump is offline. Then air bubbles cannot be formed in the system and the solids will settle instead of being suspended.

Design loading information from the desktop analysis was given to the manufacturer Suez to provide preliminary sizing details. **Table 6-7** includes the design criteria for the DAF for both Alternatives 1b and 2b.

Table 6-7: DAF Design Criteria

Parameter	Units	Alternative 1b	Alternative 2b
Design Flow	gpm	94	172
Operation Time		continuous	continuous
Influent Solids Concentration	%	0.8	0.8
Number of Units	#	2	2
Unit Width	ft	13.0	13.8
Unit Length	ft	25.4	39.3
Unit Height	ft	13.8	13.8
Air Requirements	scfm @ 70 psi	3.5	6.5
Number of Recirculation Pumps	#	1	1
Recirculation Pump HP	HP	40	75
Sludge Thickener per Unit	#	2	3
Sludge Thickener Motor	HP	0.5	0.5
Sludge Scraper per Unit	#	1	1
Sludge Scraper Motor	HP	1.5	1.5
Outlet Solids Concentration	%	4-6	4-6
Polymer Usage	lbs/dry ton sludge		

6.3.2.2.2 Rotary Drum Thickeners

Rotary drum thickeners thicken solids by compressing the sludge against a rotating drum screen to remove excess water (**Figure 6-5**). A flocculation tank is positioned upstream of the RDT where polymer is added. Sludge is fed internally into the drum where it is conveyed via an integral screw or conveying flights. The drum rotates causing the sludge to move up the side of the drum until it falls. Water is released as the sludge is tumbled inside the drum. The screw/flights convey the thickened sludge out of the drum and into a chute where it is pumped (WEF 2018). RDTs typically have internal spray wash systems to intermittently clean the drum. RDTs often perform well thickening WAS streams that do not have the best settling characteristics.

Figure 6-5: Side View of Drum Screen Inside a Rotary Drum Thickener

Rotary drum thickeners typically operate intermittently and are assumed to be operated during staffed hours. Rotary drum thickeners are also a relatively simple processes with only a couple of fail points. These include: if the feed pump is offline or fails, the polymer feed pump clogs or fails, polymer or sludge injection lines plus, or if the drum motor fails.

Design loading information from the desktop analysis was given to the manufacturer Parkson to provide preliminary sizing details. **Table 6-8** includes the design criteria for the RDTs for both alternatives 1b and 2b.

Table 6-8: RDT Design Criteria

Parameter	Units	Alternative 1b	Alternative 2b
Design Flow	gpm	395	672
Operation Time		8hrs per day 5 days per week	8hrs per day 5 days per week
Flow Capacity Per Unit	gpm	400	400
Influent Solids Concentration	%	0.8	0.8
Number of Units	#	2 (1 Duty, 1 Standby)	3 (2 Duty, 1 Standby)
Unit Width	ft	6	6
Unit Length	ft	23	23
Unit Height	ft	7	7
Number of Booster Pumps per Unit	#	1	1
Booster Pump HP	HP	5	5
Drum Gearmotor HP	HP	3	3
# of Floc Tanks per unit	#	1	1
Floc Tank Mixer HP	HP	0.5	0.5
Outlet Solids Concentration	%	4-6	4-6
Polymer Usage	lbs/dry ton sludge	7-10	7-10

6.3.2.3 Dewatering Technologies

Digested sludge would be dewatered to produce cake solids for offsite land application for both alternatives employing digestion for its stabilization process: *Alternatives 1b and 2b*. Three options were considered for dewatering: centrifuges, rotary screw presses and rotary fan presses.

6.3.2.3.1 Centrifuges

Solid bowl centrifuges dewater wastewater sludge by applying centrifugal force to the suspension. Solids mixed with a polymer would be pumped into the main centrifuge bowl (**Figures 6-6** and **6-7**). The bowl spins at high speeds, generating centrifugal force exceeding 1,000 times normal gravitational force. The solids accumulate on the interior surface of the rotation bowl. An internal rotating helical screw conveys the concentrated solids along the interior surface of the bowl to the discharge port. Dewatered cake discharges into a sump and is pumped or conveyed to truck for disposal. The liquid fraction (centrate) is recycled to the liquid treatment stream. High torque centrifuges can achieve a solids capture efficiency of 95% and produce dewatered cake with a concentration of 25% solids.

CDM Smith

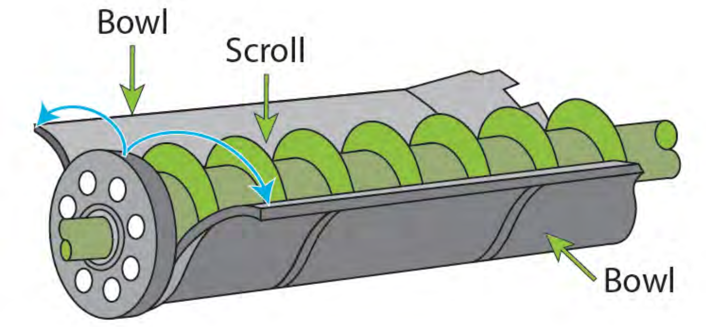


Figure 6-6: Centrifuge Scroll and Bowl Cutaway View

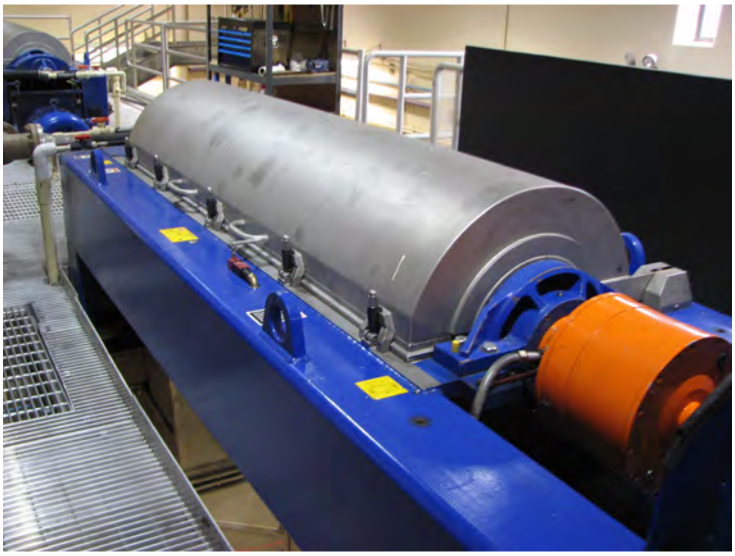


Figure 6-7: External View of Centrifuge

The solid bowl and the internal helical screw each rotate at slightly different speeds and are driven by two independent motors. The solid bowl rotates at a constant speed while the rotational speed of the internal helical screw can be varied. The difference in the rotational speed between the bowl and the internal helical screw determines the solids residence time in the machine and affects the concentration of the dewatered cake. Reducing the differential speed increases the cake solids concentration and solids capture efficiency but decreases the hydraulic and solids loading throughput of the machine.

The presence of abrasive particles in the sludge causes wear of the bowl and internal scroll. The edge of the internal scroll is protected with welded metal hard-facing or sintered tungsten carbide tiles. The scroll must be removed periodically to rebuild or repair the equipment. The scroll may operate for 15,000 hours or more between major overhauls. However, some WWTPs rebuild the centrifuges annually to prevent unexpected equipment breakdown (Integra 2003).

Centrifuges generally require small amounts of space per loading capacity and they produce cake with high solids concentration, minimizing the weight of cake for disposal. They have relatively low up-front equipment costs for the amount of capacity they can accommodate but more structural considerations. Odors are well contained in a centrifuge system. Centrifuges are difficult to maintain, requiring skilled personal or factory support. Wear on the scroll can potentially be a high maintenance problem (WEF 2018). They also require the most operator attention compared to other alternatives analyzed. Start-up and shutdown sequences for centrifuges can take about 30 minutes and the centrate quality should be consistently checked during operation. Centrifuges can be noisy and hearing protection is recommended when working in the same room for a prolonged period of time with an operating centrifuge. Centrifuges also consume the most energy per capacity compared to other alternatives analyzed.

Design loading information from the desktop analysis was given to the manufacturer Centrysis to provide preliminary sizing details. Note that the units were sized to accommodate max month loadings assuming an operating time of 8 hours per day and 3 days per week. Two units will need to be operated to accommodate the loads given the assumed operating period. If a unit is offline for maintenance during max month loading conditions, the operating time can be increased while there is only a single unit online. With a single unit online under Alternative 1b, the centrifuge will need to run for an estimated 27 hours per week or approximately 5.5 hours per day. With a single unit online under Alternative 2b, the centrifuge will need to run for an estimated 22 hours per week or approximately 4.5 hours per day. These daily operating periods provide sufficient time for startup and shut down of the units within a single shift so a third unit was not considered. **Table 6-9** presents the design criteria for the centrifuges for both Alternative 1b and Alternative 2b.

Table 6-9: Dewatering Centrifuges Design Criteria

Parameter	Units	Alternative 1b	Alternative 2b
Design Flow	gpm	600	480
Design Solids Loading Rate (MM)	lb/min/unit	92	73
Operation Time		8hrs per day 3 days per week	8hrs per day 3 days per week
Flow Capacity Per Unit	gpm	400	400
Solids Loading Capacity Per Unit	Lb/min/unit	89	89
Number of Units	#	2	2
Maximum Bowl Speed	RPM	2850	2850
Unit Width	ft	4.6	4.6
Unit Length	ft	18.3	18.3
Unit Height	ft	5.4	5.4
Main Motor HP	HP	125	125
Back Drive Motor HP	HP	40	40
Cake Solids Concentration	%	25-30	25-30
Polymer Usage	lbs/dry ton sludge	23	23
Solids Capture Rate	%	98	98

6.3.2.3.2 Screw Presses

Screw presses have mainly been used in the agricultural and industrial applications since the 1960s. Only recently have screw presses been applied in the municipal market in the United States, with most installations dating after the year 2000 (WEF 2018).

Screw presses come in two major configurations: horizontal and inclined. Inclined screw presses are positioned at angles from 10 to 20 degrees from horizontal. Sludge is fed into the screw press and is initially dewatered by gravity drainage. The sludge is conveyed via a screw which compresses the sludge, removing water through perforated plate or wire screens surrounding the screw. The screens retain the solids. The screw widens towards the discharge end, gradually reducing the cross-sectional area between the screw and the perforated wall. The dewatered solids are dropped into a hopper at the end of the press. Screw presses use automatic spray wash cleaning systems that spray water onto the press screen to remove built-up solids (WEF 2018).

Screw presses generally require relatively low maintenance and produce little noise. They require relatively minimal amounts of energy to operate. Odors are also well contained in the screw press process. Screw presses are also easy to operate requiring minimal attention. The downside to screw presses is that they produce lower cake concentrations compared to other dewatering technologies. They also have lower solids capture rates (WEF 2018).

Design loading information from the desktop analysis was given to the manufacturer PWTech to provide preliminary sizing details. **Table 6-10** includes the design criteria for the dewatering screw presses

Table 6-10: Dewatering Screw Presses Design Criteria

Parameter	Units	Alternative 1b	Alternative 2b
Design Flow	gpm	140	110
Design Solids Loading Rate (MM)	lb/min/unit	21	16
Operation Time		16 hrs per day 5 days per week	16 hrs per day 5 days per week
Flow Capacity Per Unit	gpm	240-280	240-280
Solids Loading Capacity Per Unit	lb/min/unit	44	44
Number of Units	#	2 (1 duty, 1 standby)	2 (1 duty, 1 standby)
Unit Type		Inclined	Inclined
Polymer Mixing Chamber	#	1	1
Polymer Mixer Motor	HP	0.5	0.5
Polymer Metering Pump	HP	0.5	0.5
Unit Length	ft	18.4	18.4
Unit Width	ft	10.25	10.25
Unit Height	ft	7.4	7.4
Dewatering Drum Motor	HP	2.2	2.2
Flash Mixing Tank Mixer Motor	HP	1	1
Flocculation Tank Mixer Motor	HP	3	3
Cake Solids Concentration[1]	%	15-20%	15-20%
Polymer Usage[2]	lbs/dry ton sludge	7.5-12.5	7.5-12.5
Solids Capture Rate	%	<98	<98

[1,2]From WEF 2018 Table 22.14 listing performance characteristics of screw press installations.

6.3.2.3.3 Rotary Fan Presses

A rotary fan press (**Figure 6-8**) uses friction and pressure to remove water from solids. Sludge enters the bottom of the system and moves slowly through a tapered channel which turns vertically at 180 degrees. Two rotating filter screens induce friction on the biosolids. As friction is added, filtrate is removed and drained through the screens. The cake then exits through the top of the system. Rotary fan presses have a self-cleaning system which uses wash water to flush the lines and equipment. This must run for at least 5 minutes at the end of each operating period (WEF 2018).

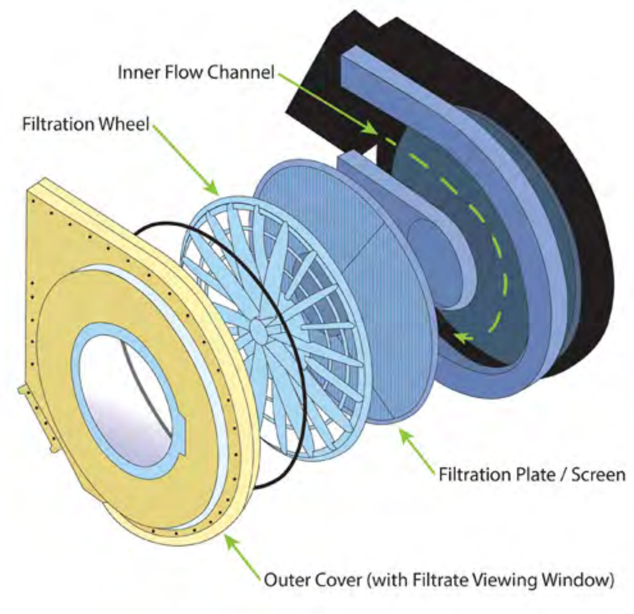


Figure 6-8: Rotary Fan Press Schematic

Rotary fan presses generally have small footprints per capacity and require less energy than centrifuges. Like the other alternatives presented, odors are also well contained in a fan press system. Fan presses are mechanically simplistic, having few moving parts. They have minimal start-up and shutdown times and produce minimal noise (WEF 2018).

A pilot study was performed by PWTech on their Volute rotary fan press at the Northglenn drinking water treatment plant (PWTech 2019). Water entering the facility is treated with aluminum coagulant before entering a clarifier. The flocs and other solids are settled in the clarifier and are pumped to a sludge lagoon. The clarified water is filtered and disinfected before being stored for distribution. The backwash from the filters is also stored in the sludge lagoon. The Volute press dewaters settled sludge in the lagoon. Generally, the influent solids concentrations into the press during the pilot ranged from 0.5-2.0%. The pilot achieved an average cake concentration of 26% and achieved a max concentration of 34% solids. The ideal polymer dosing rate was found to be 14.6 lbs/ton of solids loading yielding a solids capture rate of around 98%. The solids dewatered in this study have different characteristics than the biosolids produced at the WWTP. The solids at the drinking water facility are primarily inorganic in nature while the WWTP biosolids are organic. As a result, the performance at the WWTP may vary from what was shown in the pilot.

Design loading information from the desktop analysis was given to the manufacturer Prime Solutions to provide preliminary sizing details. **Table 6-11** includes the design criteria for the dewatering rotary fan presses.

Table 6-11: Dewatering Rotary Fan Presses Design Criteria

Parameter	Units	Alternative 1b	Alternative 2b
Design Flow	gpm	140	110
Design Solids Loading Rate (MM)	lb/min/unit	21	16
Operation Time		16 hrs per day 5 days per week	16 hrs per day 5 days per week
Flow Capacity Per Unit[1]	gpm	220	136
Number of Units	#	2 (1 duty, 1 standby)	2 (1 duty, 1 standby)
Polymer Feed Blend System	#	1	1
Fan Press Motor	HP	20	20
Polymer Metering Pump	HP	0.5	0.5
Sludge Feed Pump	#	2	2
Unit Length	ft	9.5	9.5
Unit Width	ft	10	10
Unit Height	ft	8.5	8.5
Cake Solids Concentration[2]	%	18-24%	18-24%
Polymer Usage[3]	lbs/dry ton sludge	13-15	13-15
Solids Capture Rate[4]	%	98	98

[1] This flow rate is based on the design solids loading rate.

[2] Cake solids concentration was taken from the manufacturer website and can vary depending on sludge characteristics: https://psirotary.com/equipment/rotary-fan-press/.

[3,4] The polymer usage and capture rates are based off a pilot study performed at the Northglenn drinking water treatment plant in 2019 (PWTech 2019). This pilot was performed on an alum sludge solids stream which has different characteristics than this application, therefore these figures may vary.

6.4 Evaluation of Alternatives

An evaluation of solids handling alternatives was conducted first to determine the preferred approach for solids handling. The preferred solids handling approach is then considered in conjunction with the two main process capacity alternatives to recommend improvements for addressing the capacity limitations of the existing process at design 2040 flow and loads.

6.4.1 Solids Handling Alternatives Evaluation

6.4.1.1 Comparison of Stabilization Alternatives

Table 6-12 presents the non-cost scoring for each of the stabilization alternatives.

Table 6-12: Non-Cost Criteria Scoring for Stabilization Alternatives

Evaluation Criteria	Weight (1-5)	Anaerobic Digestion (Alt 1b)	Aerobic Digestion (Alt 2b)	Keep Lagoons (Alts 1a & 2a)
Feasibility	3	+	+	+
Compliance with Future Discharge Limits	3	+	+	+
Ease of Operation	5	o	+	+
Operating Experience	1	+	+	+
Reliability and Redundancy	5	+	+	--
Flexibility	1	+	--	--
Maintenance	3	o	+	+
Health & Safety	1	--	+	+

Evaluation Criteria	Weight (1-5)	Anaerobic Digestion (Alt 1b)	Aerobic Digestion (Alt 2b)	Keep Lagoons (Alts 1a & 2a)
Electrical & Fire Code Impacts	1	--	o	o
Odor	5	o	--	--
Material Handling & Transportation	4	+	o	--
Energy Efficiency	3	+	--	+
Total Score		18	12	4
Relative Rank		1.00	0.67	0.22

Anaerobic digestion scored the highest in the non-cost evaluation. All alternatives scored the same for multiple criteria such as feasibility, compliance with future discharge limits and ease of operation. The criteria listed below are the differentiators in the non-cost evaluation.

Ease of Operation – Anaerobic digesters are more complex to operate and have more associated equipment (boiler, heat exchanger, recirculation pumps, and gas handling system) than aerobic digesters. Incoming sludge feed rates must be carefully monitored to prevent process upsets. Anaerobic digester supernatant contains high concentrations of ammonia as will the sidestreams from dewatering anaerobically digested solids. Supernatant, centrate, and filtrate return to the secondary treatment process must be managed to prevent ammonia bleed-though to the final effluent. Aerobic digester operation is similar to that for activated sludge processes. Aerobic digesters should be operated with anoxic and aerobic cycles to mitigate pH drop. Operations staff will need to monitor these cycles to prevent odor events. Aerobic digesters are prone to foaming and can seed filamentous bacteria back into the activated sludge process through decanting and dewatering processes.

Reliability and Redundancy – The liner for the solids handling lagoons is nearing the end of its useful life and will have to be repaired or replaced in the near future. Both aerobic and anaerobic digestion processes are proven, reliable treatment processes. Both digestion types are used throughout Colorado.

Flexibility- Neither aerobic digestion nor the solids handling lagoons are capable of meeting class A biosolids requirements, whereas anaerobic digesters can be upgraded to meet class A requirements. In addition, the location of the solids handling lagoons adjacent to Bull Reservoir limits the ability to use the reservoir for drinking water storage in the future.

Health & Safety, Electrical & Fire Code Impacts- Anaerobic digestion produces flammable methane gas requiring extra worker safety and design precautions.

Odor- The solids handling lagoons produce uncontained odors. Aerobic digesters may produce odors during decanting and if air on/off cycles are improperly managed. Aerobic digesters can accumulate surface foam, which can also produce noxious odors. Operations staff may need to wash down interior walls routinely to prevent foam and dried solids from accumulating and generating odors. Anaerobic digesters typically produce few odors.

Material Handling & Transportation- Anaerobic digestion produces digested sludge which is easier to dewater than aerobically digested sludge so less polymer would be required to achieve the same cake solids concentration. The solids handling lagoons produce solids with a concentration between 3 and 4%, much lower than dewatered cake. This could result in significantly higher disposal costs if the method for calculating fees switches from a dry ton basis to a wet ton basis.

Energy Efficiency- Aerobic digesters require a significant amount of energy due to aeration and mixing requirements. The solids handling ponds and anaerobic digesters are not aerated. This saves approximately two pounds of oxygen for every pound of solids stabilized. Surplus gas from anaerobic digestion will be used to heat the digester and has the potential to be used to heat buildings, fuel engine driven equipment or generate electrical power, although those options are not being considered for Northglenn.

Keeping the solids handling lagoons received the lowest score of the solids stabilization alternatives. The lagoons scored poorly in terms of flexibility since the lagoons can only produce Class B biosolids and limit the ability to use Bull Reservoir for drinking water storage. The lagoons also scored poorly on reliability because they are nearing the end of their useful life and they generate noxious odors due to the uncontained nature of the lagoons. Finally, the lagoons produce solids with low concentrations, especially compared to dewatered digester sludge. Northglenn's current biosolids disposal costs are based on a per dry ton basis, therefore the concentration of the biosolids does not directly impact the disposal costs. It is common, however, for disposal costs to be based on a wet ton basis. The costs of disposal for the solids handling lagoons process could be quadruple what it would be for a new solids handling process, including dewatering, if the solids disposal costing methodology changes in the future. **Table 6-13** presents the projected annual biosolids produced on a wet ton basis for solids handling lagoons (Alternatives 1a and 2a) and dewatered digested biosolids (Alternatives 1b and 2b).

Table 6-13: Comparison of Projected Net Weight of Solids to be Disposed

Solids Handling Process	% Solids Disposed	Wet Ton/yr Alternative 1	Wet Ton/yr Alternative 2
Solids Handling Lagoons	4%	35,000	30,000
WAS Thickening + Digestion + Dewatering	20%	7,100	6,100

Keeping the solids handling lagoons, Alternatives 1a and 2a, were eliminated from consideration based on the results of the non-cost evaluation.

6.4.1.2 Comparison of Thickening Alternatives

Table 6-14 presents the non-cost criteria scoring for the thickening alternatives.

Table 6-14: Non-Cost Criteria Scoring for Thickening Alternatives

Evaluation Criteria	Weight (1-5)	DAF	RDT
Feasibility	3	+	+
Compliance with Future Discharge Limits	3	+	+
Ease of Operation	5	+	+
Operating Experience	1	+	+

Evaluation Criteria	Weight (1-5)	DAF	RDT
Reliability and Redundancy	5	+	+
Flexibility	1	+	o
Maintenance	3	+	o
Health & Safety	1	+	+
Electrical & Fire Code Impacts	1	o	o
Odor	5	+	+
Material Handling & Transportation	4	o	o
Energy Efficiency	3	o	o
Total Score		27	23
Relative Rank		1	0.85

Both technologies are commonly used for thickening WAS and scored similarly positive on the critical ease of operation and odor criteria. Both also scored the same on material handling, since they would produce similar thickened WAS concentrations with comparable polymer addition, and most other criteria. The criteria listed below are the differentiators in the non-cost evaluation.

Flexibility- DAFs are minimally affected by changes in hydraulic loading which allows for changes to wasting rates without worrying about the impact on thickening and allows for continuous wasting. Utilizing DAFs for thickening also presents the opportunity for wasting MLSS directly from the aeration basins rather than wasting solids from the bottom of the clarifiers. One benefit of "hydraulic wasting" is that determining the waste rate to maintain an SRT is simplified since the desired waste rate is determined by dividing the aerobic volume by the desired SRT. For example, Northglenn would waste 1/10th of the aerobic volume in a day to maintain a 10-day SRT. Eliminating solids analysis from the waste rate calculation also allows a stable SRT to be maintained which is beneficial to the biological process. The stability also benefits the competing biological activity occurring in the BNR trains. Operators can feel more comfortable dialing into a lower SRT that maintains the nitrifier population but also benefits biological phosphorus removal.

Maintenance- Neither alternative is maintenance intensive, but the RDTs require periodic cleaning of the drum with a power washer. The DAFs will also run continuously while the RDTs will require the routine maintenance associated with start-up and shutdown of the equipment whenever it is run.

Overall, DAFs scored higher than RDTs for the non-cost analysis. The main differentiator for the DAF is the level of flexibility it offers in terms of hydraulic loading rates. Having the ability to hydraulically waste is beneficial to a 3-stage BNR process for which nitrification, denitrification and biological phosphorus removal are occurring within the same reactor.

6.4.1.3 Comparison of Dewatering Alternatives

Table 6-15 presents the non-costs scoring for each of the dewatering alternatives.

Table 6-15: Non-Cost Criteria Scoring for Dewatering Alternatives

Evaluation Criteria	Weight (1-5)	Centrifuge	Rotary Screw Press	Rotary Fan Press
Feasibility	3	+	+	+
Compliance with Future Discharge Limits	3	+	+	+
Ease of Operation	5	o	+	+
Operating Experience	1	+	o	o
Reliability and Redundancy	5	+	+	+
Flexibility	1	+	+	+
Maintenance	3	o	+	+
Health & Safety	1	o	+	+
Electrical & Fire Code Impacts	1	o	o	o
Odor	5	+	+	+
Material Handling & Transportation	4	+	--	o
Energy Efficiency	3	--	o	o
Total Score		19	22	26
Relative Rank		0.73	0.85	1.00

All three technologies have been successfully implemented for dewatering sludge and scored similarly positive on the critical reliability and redundancy and odor criteria. All three also scored the same on feasibility, compliance with future discharge limits and flexibility. The criteria listed below are the differentiators in the non-cost evaluation.

Ease of Operation- Centrifuges are typically only operated during staffed hours, require consistent monitoring for performance, and have relatively time-consuming startup and shutdown sequences. Many larger treatment facilities, such as Denver Metro and Broomfield, require an operator to remain with the centrifuge the entire time it is operating. A change in feed rate can result in a plugged centrifuge and/or damage to the equipment. Smaller facilities, such as the Frisco WWTP, routinely operate their centrifuges unattended. Other alternatives require less attention and can even be run after startup without attendance.

Operating Experience- Centrifuges are widely applied in the municipal industry unlike the other alternatives. Both rotary screw presses and rotary fan presses are used, but they are not as common.

Maintenance – Centrifuges are more complex mechanically and typically cannot be maintained in-house. The scroll must be removed periodically to rebuild or repair the equipment. This can be spread out between 15,000 hours of operation or can be done on a more regular basis to prevent unexpected equipment breakdown.

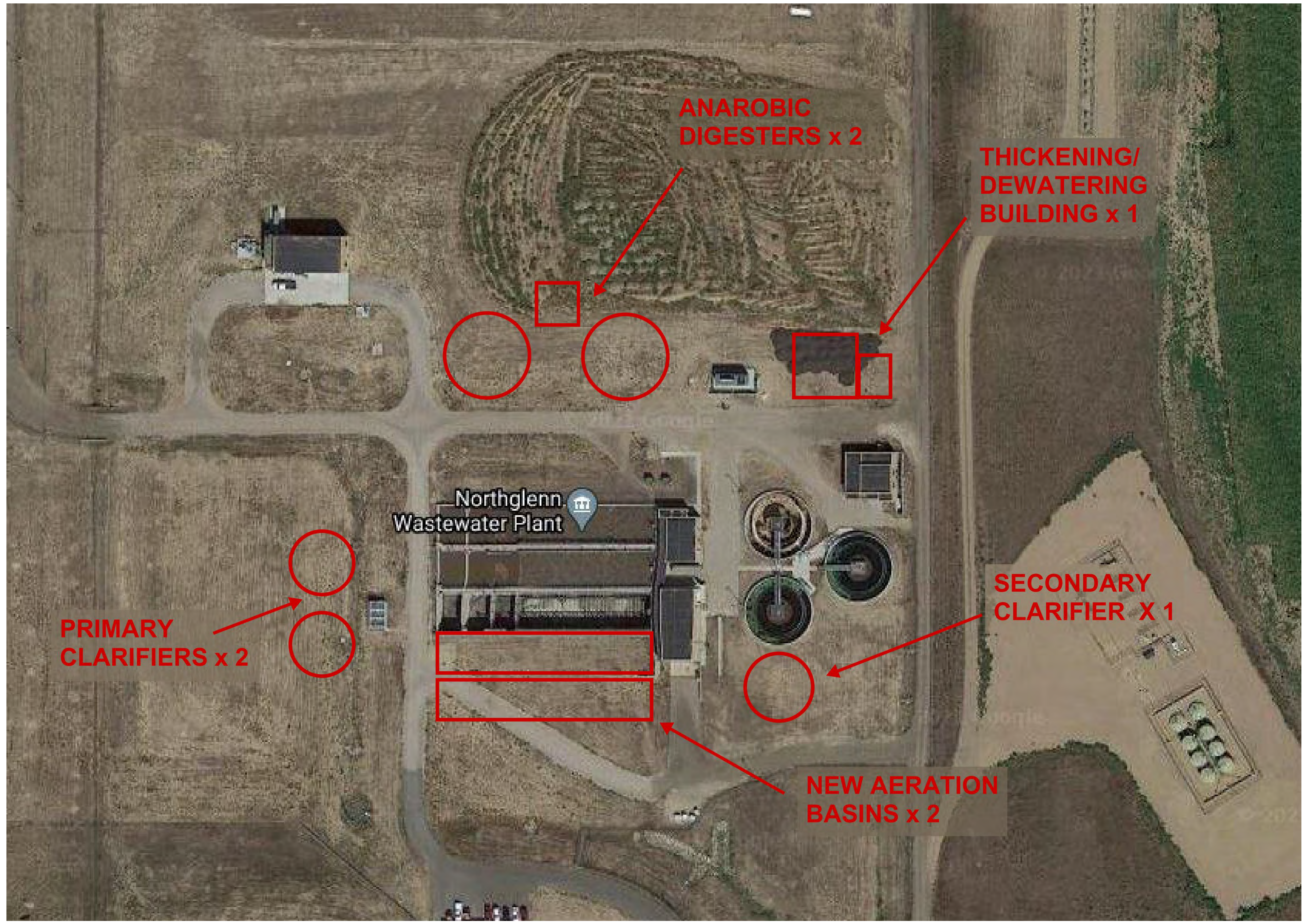
Material Handling & Transportation- Centrifuges can produce the highest percentage of cake solids of the three alternatives at 25 to 30%. The fan press pilot study found that with influent solids of similar concentration to this application, they were able to achieve cake with an average concentration of 26% solids; however, the solids dewatered in the pilot are different than this application and so these results may not be applicable. Performance at other WWTPs indicate cake concentrations of between 18 and 24% are expected for the fan press. The screw press produces the lowest cake concentration of between 15 and 20%.

Energy Efficiency – The power required to operate a centrifuge is significantly higher than the rotary screw press and the rotary fan press.

Overall, the rotary fan press scored higher than the other alternatives in the non-cost analysis. The main differentiator for the rotary fan press compared to the rotary screw press is the higher cake solids produced by the fan press, which would result in lower disposal costs. The rotary fan press is preferred over centrifuges due to ease of operation and maintenance and the lower energy costs associated rotary fan press operation.

6.4.2 Overall Alternatives Evaluation

Process capacity Alternatives 1b and 2b were compared on a non-cost and cost basis. **Figure 6-9** shows a proposed layout for Alternative 1b and **Figure 6-10** shows a proposed layout for Alternative 2b. Alternatives 1a and 2a were not considered further since maintaining the existing solids handling lagoons scored so poorly in the non-cost evaluation of solids stabilization options. DAF and rotary fan presses were assumed for WAS thickening and digested sludge watering, respectively, in the overall evaluation since they scored highest in the non-cost evaluations.



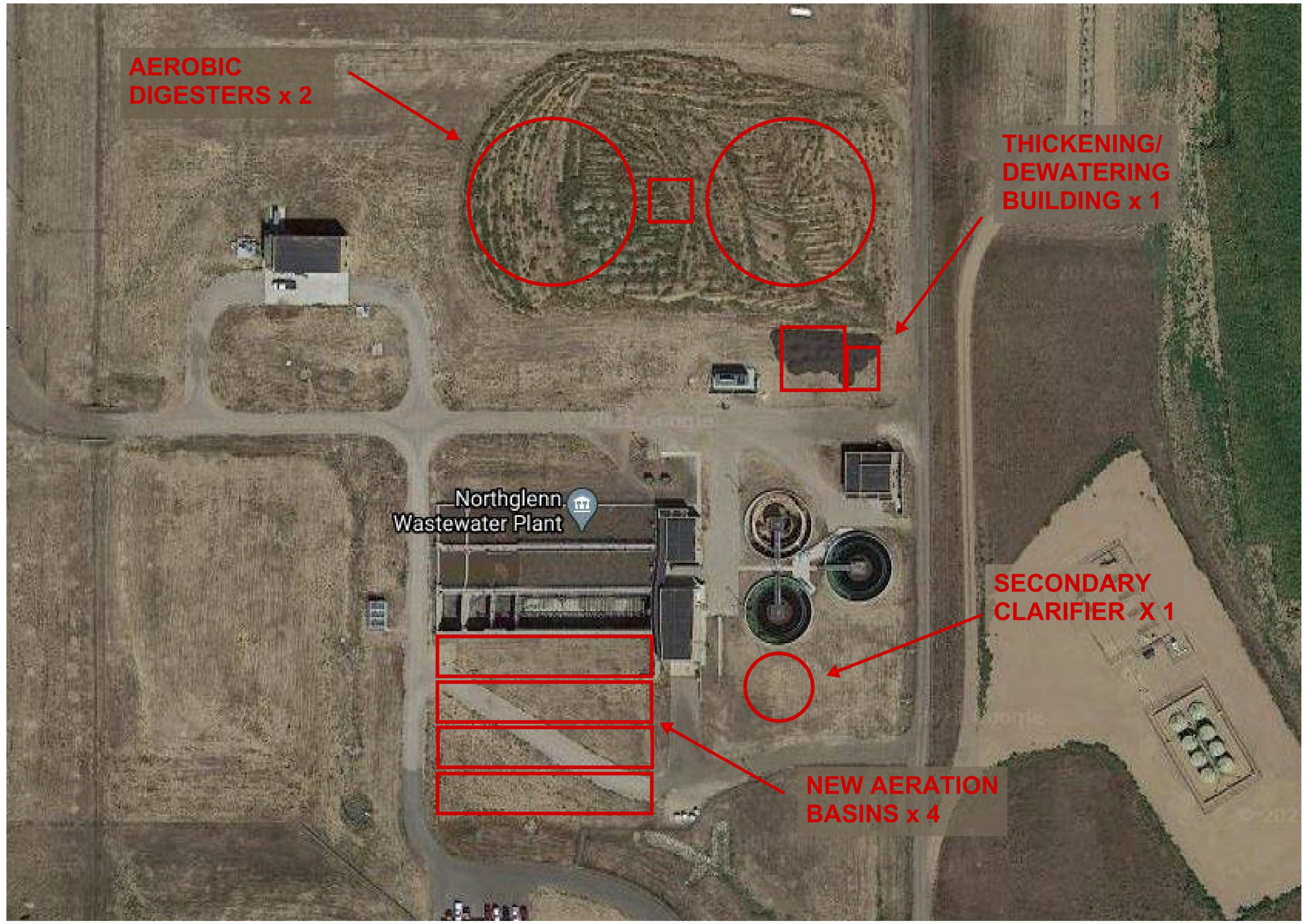
ALT1 SITE

PLAN

SCALE: NTS

CDM Smith

Figure 6-9
NORTHGLENN WWTP
Proposed Layout for Alternative 1b



ALT2 SITE
PLAN
SCALE: NTS

CDM Smith

Figure 6-10
NORTHGLENN WWTP
Proposed Layout for Alternative 2b

6.4.2.1 Non-Economic Evaluation

Table 6-16 summarizes the non-cost criteria scoring for remaining two process capacity alternatives.

Table 6-16: Non-Cost Criteria Scoring for Process Capacity Upgrades Alternatives

Evaluation Criteria	Weight (1-5)	Alt 1b: Process Capacity Upgrades with Primary Clarifiers and Solids Handling	Alt 2b: Process Capacity Upgrades, No Primaries, with Solids Handling
Feasibility	3	+	+
Compliance with Future Discharge Limits	3	+	+
Ease of Operation	5	+	+
Operating Experience	1	+	+
Reliability and Redundancy	5	+	+
Flexibility	1	+	--
Maintenance	3	o	--
Health & Safety	1	--	+
Electrical & Fire Code Impacts	1	--	o
Odor	5	o	o
Material Handling & Transportation	4	o	o
Energy Efficiency	3	+	--
Total Score		19	11
Relative Rank		1.00	0.58

Both overall treatment alternatives are widely implemented in WWTPs and Northglenn would be able to achieve nutrient limits with or without primary treatment. Both also scored similarly on critical ease of operation, reliability and redundancy and odor criteria. They also scored the same on material handling and transportation since the slight increase in biosolids production associated with primary clarifiers is offset by anaerobically digested sludge being easier to dewater than aerobically digested sludge. Implementing Alternative 1b would take up less space on the site, but there is adequate space available for both alternatives. The criteria listed below are the differentiators in the non-cost evaluation.

- **Flexibility-** Primary treatment reduces the overall loading to the BNR process and provides another tool to manage the variability inherent in wastewater flow and loading to the plant. Primary treatment reduces grease and scum going to the activated sludge process and may reduce foaming due to Nocardioforms. Finally, primary clarifiers may be operated to generate volatile fatty acids (VFAs), which may improve phosphorus removal. Aerobic digestion is not capable of meeting class A biosolids requirements, whereas anaerobic digesters can be upgraded to meet class A requirements.

- **Maintenance** – Including primary clarifiers and anaerobic digesters adds to the pieces of equipment that need to be maintained, although primary clarifiers will remove additional grit and heavy solids than can protect and ease maintenance on downstream equipment. Without primary clarifiers, more activated sludge basins will be needed along with their associated equipment and valves. Diffuser replacement, in particular, is an extremely labor-intensive maintenance activity that must be completed every few years. Overall, the presence of primary clarifiers should decrease total maintenance time and expense.

- **Health & Safety, Electrical & Fire Code Impacts**- Anaerobic digestion produces flammable methane gas requiring extra worker safety and design precautions.
- **Energy Efficiency**- Adding primary clarifiers reduces the BOD_5 loading to the BNR treatment process and TKN loading to a lesser extent, thereby reducing the aeration demand for secondary treatment. Aeration is the highest energy consuming process for treatment facilities. Aerobic digestion requires additional aeration consuming more energy. Surplus gas from anaerobic digestion has the potential to be used to heat buildings, fuel engine driven equipment or generate electrical power, although those options are not included in these recommendations for Northglenn.

Overall, process capacity Alternative 1b scored higher than Alternative 2b in the non-cost analysis. The main differentiators for including primary clarifiers in the liquid treatment train include reduced overall maintenance time and expense, improved flexibility for the operations staff, and the ability to reduce the overall loading on the BNR process. Primary clarifiers also can relieve some of the stress that variability in flow and load places on the BNR process. Load reduction into the secondary process also decreases energy consumption of the aeration process.

6.4.2.2 Economic Evaluation

Table 6-17 presents the opinions of probable construction cost and overall program costs for process upgrades Alternatives 1b and 2b. Alternative 1b has the lower OPCC due to the overall smaller footprint associated with the alternative. The OPCC is about 4% lower for Alternative 1b, however this cost difference is well within the range of cost accuracy expected for this order-of-magnitude estimate.

Table 6-17: Costs Comparison for Overall Alternatives

Process Improvements		Alternative 1b	Alternative 2b
Primary Clarifiers and Splitter Box		$5,120,000	$-
BNR Aeration Basins		$6,200,000	$11,500,000
Secondary Clarifiers		$2,900,000	$2,900,000
WAS and RAS Pumps[1]		$310,000	$380,000
Digesters		$5,100,000	$8,200,000
Solids Handling Building[2]		$7,300,000	$7,300,000
Equalization Tank[3]		$400,000	$-
Yard Piping		$3,765,000	$3,400,000
Electrical and Controls		$5,700,000	$5,700,000
Subtotal of Process Improvements		$36,795,000	$39,380,000
Indirect Costs (Permits, Bonding and Insurance)	5%	$1,839,750	$1,969,000
Subtotal		$38,634,750	$41,349,000
Contractor's Field General Conditions, Overhead and Profit	10%	$3,863,475	$4,134,900
Subtotal with OH&P		$42,498,225	$45,483,900
Construction Contingencies	30%	$12,749,468	$13,645,170
Total Construction Costs		$55,247,693	$59,129,070
Construction Escalation to Mid-Point of Construction	11.25%	$6,215,365	$6,652,000
Total Opinion of Probable Construction Cost (Rounded)		$61,000,000	$66,000,000
Project Contingency	20%	$12,200,000	$13,200,000
Subtotal		$73,200,000	$79,200,000
Engineering and Implementation	20%	$14,640,000	$13,200,000
Total Program Cost		$87,840,000	$92,400,000
Total Program Cost (Rounded)		$88,000,000	**$92,000,000**

[1] RAS pump costs per unit were based on the 2003 MP and were escalated to 2020 dollars.
[2] Cost of solids handling building is based off the 2012 MP. This cost includes RDTs for thickening and rotary screw presses for dewatering. Also includes a truck bay for cake offloading and assumes that truck trailers will be available for cake storage until shipping.
[3] Equalization tank assumes two days of storage under max month loads.

Annual biosolids disposal rates from 2015 to 2020 for both onsite disposal and offsite disposal were used to project future disposal rates. **Table 6-18** lists the onsite and offsite disposal rates for Northglenn from 2015 to 2020 as well as the projected rates in 2040.

Table 6-18: Historical Solids Disposal Rates

Year	Onsite Rates	Offsite Rates	Onsite/Offsite
2015	NA	$294.00	NA
2016	$204.00	$386.00	0.53
2017	$275.00	NA	NA
2018	NA	$392.80	NA
2019	$338.00	$405.00	0.84
2020	$342.00	$410.00	0.83
2040[1]	$445.00	$533.00	0.83

[1] 2040 offsite biosolids disposal rates were projected according to the increase in offsite rates from 2016-2020. Onsite rates for 2040 were assumed to be 83% of offsite rates, representative of the rates from 2019-2020.

The projected rates for solids disposal for 2040 were used to estimate and compare costs for Alternatives 1b and 2b for onsite and offsite disposal. Disposal costs are shown for both 2020 rates and projected 2040 rates in **Table 6-19**. The annual disposal costs are higher for Alternative 1b since more sludge is produced with that alternative.

Table 6-19 Projected Annual Biosolids Disposal Costs

Price Year	Onsite Rates $/Dry Ton	Offsite Rates $/Dry Ton	Alternative 1b			Alternative 2b		
			Dry Ton/yr Produced	Onsite Disposal Cost $/yr	Offsite Disposal Cost $/yr	Dry Ton/yr Produced	Onsite Disposal Cost $/yr	Offsite Disposal Cost $/yr
2020	$342.0	$410.0	1400	$480,000	$570,000	1200	$410,000	$490,000
2040	$445.0	$534.0	1400	$620,000	$750,000	1200	$530,000	$640,000

6.5 Recommended Improvements

Alternative 1b is the recommended treatment process alternative to address capacity limitations at the WWTP and to maintain compliance with current and near future regulatory requirements for TIN and TP. Alternative 1b includes the following improvements:

- Two 60-ft diameter primary clarifiers, a clarifier splitter box and primary sludge pumping station. The pumping station would house both primary sludge and scum pumps.
- Two additional 3-Stage BNR trains, for a total of five, configured the same as the existing trains and with the same dimensions. The new trains include all associated mechanical equipment such as anoxic mixers, fine bubble diffusers, IMLR pumps and all required process piping. No additional blower capacity is required.
- One additional 65-ft diameter secondary clarifier, for a total of four, three new RAS pumps and modifications to the RAS pump discharge header. No additional WAS pumps are required.
- Two 80-ft diameter anaerobic digesters (or equivalent volume) and all accompanying equipment such as boiler, heat exchanger, recirculation pumps, gas handling system

and flare. The associated equipment would be located in a building adjacent to the digesters.

- One flow equalization tank so dewatering side streams can be returned over a 24-hour cycle or be used to even out the total NH_3–N load to the activated sludge process by returning more of it during periods of low flow and load.
- Two DAF thickeners to thicken WAS upstream of the anaerobic digesters. The DAFs would be located within a new Solids Handling Building.
- Two (1 duty/1 standby) rotary fan press dewatering units installed within the new Solids Handling Building. The rotary fan presses would dewater the thickened sludge.
- A Solids Handling Building that would house the DAFs and rotary fan presses and all associated equipment such as sludge transfer pumps, polymer feed systems, mix tanks, cake pumps and truck loading bays.

Alternative 1b and 2b received similar rankings in the non-cost evaluation and are considered equal in cost for this planning level evaluation. Alternative 1b is recommended due to the improved energy efficiency, reduced maintenance time and expense. and additional flexibility and it offers the operators in both the liquid and solids trains. Primary treatment reduces the overall loading to the BNR process and provides another tool to manage the variability inherent in wastewater flow and loading to the plant. Primary treatment reduces the number of activated sludge basins required and the associated maintenance, such as diffuser replacement. It also removes grease, scum and heavy solids which can protect and lessen maintenance on downstream equipment. Load reduction into the secondary process decreases energy consumption of the aeration process. Anerobic digestion can be upgraded to meet class A requirements and also had the ability to produce energy rather than consume it.

Section 7

Capital Improvements Plan

The Capital Improvements Plan (CIP) for the Northglenn WWTP is presented in this section. The CIP incorporates the results of the conditions assessment, the evaluation of the existing treatment facility, and the alternatives analysis to address capacity limitations as the city continues to expand. **Section 4.3.5** presented the conditions assessment of the existing facility which identified equipment which urgently needs replacement. **Section 4.3.6** recommended improvements to the existing treatment system to benefit operations and maintain the facility's ability to meet effluent limits. Finally, **Section 6.5** summarized the recommended upgrades to the facility that should be undertaken in the near future to address capacity concerns. This section presents the recommended timeline for each project as well as the cost of each project and potential funding opportunities.

7.1 Capital Improvement Projects

Two projects are recommended for implementation over the 20-year planning cycle in this Plan. Project 1 includes replacement of existing equipment in poor condition and improvements to the existing facility addressing specific concerns from staff related to plant O&M and process performance. Project 2 includes facility upgrades to increase capacity to accommodate current and future flows and loads. Project 2 is further broken down into two phases for implementation.

7.1.1 Project 1: Condition Assessment Equipment Replacement and Improvements to Existing Treatment System

Project 1 involves repairs, replacements, and facility improvements that are recommended for implementation immediately to ensure operational reliability and continued compliance with effluent limits. Project 1 would also include any additional improvements resulting from the analyses recommended at the end of **Section 4.3.6**.

Working with facility staff, the following pieces of equipment were identified as needing urgent replacement. Items needing urgent replacement were identified based on whether they meet one of the following characteristics:

- Asset failed or failure is imminent
- Excessive maintenance is required
- No further service life expectancy
- Significant health and safety hazard

The equipment was broken up by location within the facility. See **Section 4.3.5** for details on the condition assessment and what was included in the OPCC. No program costs were included for the conditions assessment since these items will be replaced in-kind and the scope is well-defined.

- Aeration Basins

  - Replace Mixers: $ 260,000
  - Replace Gates: $ 970,000
  - Replace Valves: $ 60,000
  - Subtotal: $ 1,300,000
- Process Basement - Secondary Treatment
  - Replace Valves: $ 600,000
  - Subtotal: $ 600,000
- Secondary Clarifiers
  - Replace Clarifier Mechanism: $ 2,000,000
  - Subtotal: $ 2,000,000
- UV Building Subtotal
  - Replace Slide Gates: $ 150,000
  - Subtotal: $ 150,000
- Equipment Replacements Total
  - Total OPCC: $ 4,100,000

Upgrades to the existing facility were recommended to address specific concerns brought up by staff. These upgrades will improve facility O&M and will likely improve overall performance of the WWTP. The recommended improvements are listed below along with the total projected program costs. Program costs include the OPCC, additional contingency to account for project development, engineering, and implementation. Refer to **Section 4.3.6** for details on how program costs were developed.

- Demolition of Heat Exchange Equipment
- Relocation of Existing Secondary Clarifier Launders
- Installation of Additional Process Control Analyzers
  - New DO probe in each aeration basin
  - New NO_3 analyzer in each aeration basin
- Installation of H_2S sensors and transmitters in headworks building for operational safety
- **Recommended Improvements to Existing Treatment System**
  - Total Program Cost: $ 1,400,000

The total program costs and recommended target completion date for **Project 1** are:

- Total Program Cost: **$ 5,500,000**
- Target Completion Date: **March 2024**

7.1.2 Project 2: Recommended Improvements for Capacity Upgrades

Section 6 evaluated alternatives for meeting capacity requirements and nutrient limits through the year 2040. Over the last 10 years, the facility has exceeded 80% of the current permitted capacity of 4.2 mgd for 22 months and has exceeded 95% of the current permitted capacity for 6 additional months. According to Northglenn's discharge permit, the city is required to initiate financial planning for the expansion of the WWTP when the 30-day average flows exceed 80% of the treatment capacity. When the 30-day average flows reach 95% of the capacity, construction of facility upgrades must commence or issuance of building permits within the municipality which will contribute to the increase of flow to the facility must cease until construction of upgrades has commenced (CDPHE 2019). In addition, the facility exceeded 80% influent BOD_5 loading capacity 32 months over the last 10 years and exceeded 95% during November of 2019. It is recommended that the construction of the upgrades described in **Section 6.5** begin as soon as possible to allow development within the service area to continue. The following upgrades are recommended to meet projected future flows and loads within the 20-year planning period:

- Primary Clarifiers
  - Construction of two primary clarifiers with a splitter box and primary sludge pumping station
- Secondary Process Upgrades
  - Construction of two additional 3-stage BNR trains
  - Construction of one additional secondary clarifier
- Solids Handling Process Upgrades
  - Construction of two anaerobic digesters with sludge heating and gas handling equipment
  - Construction of two DAF thickeners for WAS thickening upstream of anaerobic digestion
  - Construction of two rotary fan press dewatering units
  - Construction of a solids handling building that would house the DAFs and rotary fan presses and all associated equipment such as sludge transfer pumps, polymer feed systems, mixing tanks, cake pumps and solids truck loading bays
  - Construction of a side stream flow equalization tank

If these upgrades were constructed in a single phase, the total program costs would be around $88 million, as presented in **Table 6-17**. This cost assumes a midpoint of construction in January 2025. Refer to **Section 6.2.2** for additional details on how costs were developed.

All upgrades listed above are needed to meet the flows and loads projected in **Section 5** for the 20-year planning horizon; however, several scenarios for the phased implementation of upgrades were evaluated to spread out the capital cost burden over more time. These scenarios, along with notes on scenario feasibility, include:

1. Construct primary clarifiers prior to solids handling and secondary process upgrades.
   a. Not feasible: primary solids would need to be sent to the sludge lagoons which would generate significant odors.
2. Construct solids handling process upgrades prior to primary clarifiers and secondary process upgrades.
   a. Not feasible: anaerobic digesters do not operate efficiently without primary solids and may not be able to meet minimum volatile solids reduction requirements in Regulation 64 and 40 CFR Part 503.
3. Construct primary clarifiers and solids handling process upgrades prior to secondary process upgrades.
   a. Feasible: Phase 1 would consist of constructing primary clarifiers with solids handling and Phase 2 would consist of constructing the secondary treatment upgrades. Building the primary clarifiers with the solids handling process upgrades would restore some of the lost flow and BOD_5 loading capacity from the decommissioning of the primary ponds, and this would allow Northglenn to push the Phase 2 upgrades into the future. Upgrading the solids handling process is critical for Northglenn since the solids handling lagoons are nearing the end of their useful life and are an odor source due to their uncontained nature. Since upgrading solids handling is a priority and the primary clarifiers are required to implement anaerobic digestion, construction of primary clarifiers and solids handling is recommended prior to secondary process upgrades rather than constructing the secondary process upgrades first.

The third phasing approach is recommended for implementation of capacity upgrades. Construction of Phase 1 would need to begin as soon as possible since the WWTP is already receiving flows and BOD_5 loads over 95% capacity. After constructing Phase 1 upgrades, the facility would be rated for a maximum month influent BOD_5 load between 8,700 ppd and 10,300 ppd depending upon the design criteria assumptions utilized. The flow rating of the modified WWTP would have to be determined with a hydraulic analysis. The timeline for the construction of Phase 2 is dependent on the rate of planned development in Northglenn and how quickly influent flows and loads will exceed the 95% thresholds of the new WWTP rating.

Two Project 2 implementation schedules were developed. The first schedule considers full development of the SSA and the development of Section 36 in the NSA. The second schedule considers the full development of the SSA but assumes no development of Section 36 of the NSA.

7.1.2.1 Project 2 Development Plan 1: Full buildout of the SSA; Section 36 Development in NSA

Both Phase 1 and Phase 2 upgrades are required to meet the projected flows and loads associated with full buildout of the SSA and the development of Section 36 within the NSA. As noted previously, construction of Phase 1 should begin as soon as possible so that development in the service area is not restricted. It is assumed that construction would be completed in five years. Phase 2 would need to be constructed prior to the development of Section 36. There is uncertainty as to when Section 36 might be developed. For the purposes of this Plan, it was assumed that construction of Phase 2 will need to begin in eight years to prepare for the development of Section 36.

Table 7-1 presents the program costs for Phase 1 and reflects a mid-point of construction of January 2025, similar to the mid-point used for the alternatives analysis in **Section 6**. **Table 7-2** presents the program costs for Phase 2 and reflects a mid-point of construction of January 2030. The total program cost of both phases is $93 million, which is approximately 6% higher than if both phases were constructed by 2026 (refer to **Section 6.4.2.2**). The higher cost is due to pushing Phase 2 further into the future.

Table 7-1: Costs for Phase 1 of Project 2 (Development Plan 1 and 2)

Process Improvements		Costs
Primary Clarifiers and Splitter Box		$5,120,000
Digesters		$5,100,000
Solids Handling Building		$7,300,000
Equalization Tank		$400,000
Yard Piping		$2,000,000
Electrical and Controls		$3,200,000
Subtotal of Process Improvements		$23,130,000
Indirect Costs (Permits, Bonding and Insurance)	5%	$1,156,500
Subtotal		$24,286,500
Contractor's Field General Conditions, Overhead and Profit	10%	$2,428,650
Subtotal with OH&P		$26,715,150
Construction Contingencies	30%	$8,014,545
Total Construction Costs		$34,729,695
Construction Escalation to Mid-Point of Construction	11.25%	$3,907,090
Total Opinion of Probable Construction Cost (Rounded)		$38,636,785
Project Contingency	20%	$7,727,357
Subtotal		$46,364,142
Engineering and Implementation	20%	$9,272,828
Total Program Cost		$55,636,971
Total Program Cost (Rounded)		**$56,000,000**

Table 7-2: Costs for Phase 2 of Project 2 Development Plan 1

Process Improvements		Costs
Aeration Basins		$6,200,000
Secondary Clarifiers		$2,900,000
WAS and RAS Pumps		$310,000
Yard Piping		$1,765,000
Electrical and Controls		$2,490,000
Subtotal of Process Improvements		$13,665,000
Indirect Costs (Permits, Bonding and Insurance)	5%	$683,250
Subtotal		$14,348,250
Contractor's Field General Conditions, Overhead and Profit	10%	$1,434,825
Subtotal with OH&P		$15,783,075
Construction Contingencies	30%	$4,734,923
Total Construction Costs		$20,517,998
Construction Escalation to Mid-Point of Construction	26.25%	$5,385,974
Total Opinion of Probable Construction Cost (Rounded)		$26,000,000
Project Contingency	20%	$5,200,000
Subtotal		$31,200,000
Engineering and Implementation	20%	$6,240,000
Total Program Cost		$37,440,000
Total Program Cost (Rounded)		**$37,000,000**

The two phases for Development Plan 1 are summarized below:

- Phase 1: Construct Primary Clarifiers and Solids Handling Process
  - Total Program Cost: $ 56,000,000
  - Begin Design: 2022
  - Begin Construction: 2024
  - Project Completion: 2026
  - Facility Rated Capacity after Implementation
    - *Flow: 4.7 mgd*
    - *BOD_5: 11,300 ppd*
- Phase 2: Construct Aeration Basins and Secondary Clarifier
  - Total Program Cost: $37,000,000
  - Begin Design: 2028
  - Begin Construction: 2029
  - Project Completion: 2031

- Facility Rated Capacity after Implementation
  - *Flow 5.8 mgd*
  - *BOD_5: 11,800 ppd*

7.1.2.2 Project 2 Development Plan 2: Full buildout of the SSA; No Development in Section 36

It may be possible to meet projected flows and loads for the full development of the SSA without the construction of Phase 2. The BOD_5 loading capacity of the facility would exceed the projected maximum month loading of 8,630 ppd for the full development of the SSA. The projected maximum month flow for the full development of the SSA is 5.13 mgd. This flow rate may exceed the expected flow rating after Phase 1 is implemented, but the projected flows are likely skewed due to abnormal wet weather events in 2015. If Section 36 is not developed, it is recommended that a study be conducted to further develop the basis for projected flows to determine if and/or when Phase 2 needs to be constructed. The program costs for Phase 1 remain the same as shown in **Table 7-1** for development Plan 1. Phase 2 costs would change depending on when the additional flow analysis determines Phase 2 needs to be constructed.

The two phases for Development Plan 2 are summarized below.

- Phase 1: Construct Primary Clarifiers and Solids Handling Process
  - Total Program Cost: $ 56,000,000
  - Begin Design: 2022
  - Begin Construction: 2023
  - Target Completion: 2025
  - Facility Rated Capacity after Implementation
    - *Flow: 4.7 mgd*
    - *BOD_5: 11,300 ppd*
- Phase 2: Construct Aeration Basins and Secondary Clarifier
  - A study is recommended to further develop the basis for projected flows according to the planned rate of city development in the SSA to determine if Phase 2 is necessary and, if so, when construction should begin.
  - Total Program Cost: $37,000,000 +/-; this cost would change depending on when the mid-point of construction occurs.

7.1.3 Capital Improvements Summary

Table 7-3 summarizes the recommended capital improvements by year for Development Plan 1. **Table 7-4** summarizes the recommended capital improvements by year for Development Plan 2.

Table 7-3: Capital Improvements Summary, Development Plan 1

Project Description	Estimated Cost	Year										
		2021	2022	2023	2024	2025	2026	2027	2028	2029	2030	2031
Project 1												
Existing Facility Equipment Replacements	$4,100,000	Start			Completion							
Existing Treatment Facility Improvements	$1,400,000	Start			Completion							
Project 2												
Facility Capacity Upgrades Phase 1	$56,000,000		Begin Design	Finish Design	Begin Construction		Completion					
Facility Capacity Upgrades Phase 2	$37,000,000								Begin Design	Begin Construction		Completion

Table 7-4: Capital Improvements Summary, Development Plan 2

Project Description	Estimated Cost	Year										
		2021	2022	2023	2024	2025	2026	2027	2028	2029	2030	2031
Project 1												
Existing Facility Equipment Replacements	$4,100,000	Start			Completion							
Existing Treatment Facility Improvements	$1,400,000	Start			Completion							
Project 2												
Facility Capacity Upgrades Phase 1	$56,000,000		Begin Design	Finish Design	Begin Construction		Completion					
Facility Capacity Upgrades Phase 2	$37,000,000*	*Begin construction if facility maximum month influent flows reach 4.46 mgd or influent BOD5 loading reaches 10,700 ppd.*										

*Phase 2 costs depend on the timing of the project. Cost assumes escalation to midpoint of construction in 2032.

7.2 Funding Options

Described below are options for funding sources including federal and local grants and loans. It is recommended that a rate study be performed as soon as possible to plan for the projects described in the CIP.

7.2.1 Federal and State Funding Sources

Northglenn can apply for grant and loan funds available to public entities for wastewater utility system projects. The State of Colorado has revolving funds available for grants and low-interest loans for water and wastewater facility projects which are funded through the U.S. EPA and the Federal Clean Water Act. This is the primary funding source available for Northglenn's capital improvement plan. **Table 7-5** provides a summary of the agencies and their contacts for the Colorado Water Pollution Control Revolving Funds (WPCRF). This source rarely provides full funding of a construction project. Northglenn must supplement these funds with matching funds to meet eligibility criteria and to ensure that implementation of the recommended capital improvement projects can occur.

Table 7-5: Colorado Water Pollution Control Revolving Fund Agencies and Contacts

Agency	Address	Phone	Internet
Water Quality Control Division Grants and Loans Unit	4300 Cherry Creek Drive South, B-2 Denver, CO 80246	303.692.2053	cdphe_grantsandloans @state.co.us
Colorado Water Resources & Power Development Authority	1580 Logan Street, Suite 620 Denver, CO 80203	Keith McLaughlin 303.830.1550	www.cwrpda.com
Department of Local Government Affairs (DOLA)	1313 Sherman Street, Room 521 Denver, CO 80203	Barry Cress 303.866.2352	www.cwrpda.com barry.cress@state.co.us

7.2.1.1 Colorado Water Pollution Control Revolving Fund and State Domestic Wastewater Treatment Plant Grant Program

Colorado Senate Bill 50 amended Title 37 of Colorado Revised Statutes (CRS). Title 37 of Article 95 established the WPCRF as an on-going funding mechanism for water quality projects. This was authorized and completed under the guidelines of the federal Clean Water Act. The purpose of these funds is to improve or benefit water quality in the state. The program is administered through a partnership between CDPHE through the Division, the DOLA, and the Colorado Water Resources and Power Development Authority (Authority).

The Authority was created by the General Assembly to aide other state agencies in managing the funds for various funding programs for water and wastewater capital improvements. The Authority assists governmental entities such as cities and special districts by issuing revenue bonds and loaning the proceeds to the governmental entity with substantial savings in costs of issuance and interest rates. Eligible projects include wastewater treatment plants, storage reservoirs, water distribution systems, water wells and pumping stations. Eligible costs include design, engineering, costs of issuance, financing reserves, interest during construction, site acquisition, planning, environmental documentations, construction and mitigation costs. There are two loans types applicable to Northglenn. One is a direct loan, if under $3 million, and the other is a leveraged loan, which is any loan in an amount over $3 million. Loans of less than $3 million were given at an interest rate of 2.25% during 2021.

In order to receive a loan through the WPCRF, you must first make sure that your project is on the WPCRF Eligibility List. If your project isn't on the list, your community can apply to get on the list by filling out the eligibility survey. **Table 7-6** shows the application deadlines.

Table 7-6: Application Deadlines for WPCRF

Application Deadlines	Loan Type	Authority Board Meeting
January 15	Direct Loans (Bond Issue Spring)	March
February 15	Direct Loans	April
April 15	Direct Loans	June
June 15	Direct Loans (Bonds Issue Fall)	August
August 15	Direct Loans	October
October 15	Direct Loans	December
November 15	Direct Loans	January

7.2.1.2 Colorado Water Resources and Power Development Authority Interim Loan Policy

Interim loans can be made to entities for their water and wastewater infrastructure projects for up to two years, or until closing of the next bond issue for leveraged loans. The types of projects funded are the same as those funded by the Authority. The project does not necessarily need to be on the eligibility list but must be eligible to be on the list. The interim loans are generally only available for projects that are assumed to receive a loan under the WPCRF program but have financial obligations prior to the receipt of the bond proceeds.

The interest rate is higher than the previously mentioned loans. If not paid within the two-year time frame, the interest rate defaults to the prime rate plus two percent. This program is intended to fund projects until they can receive other loan or financing mechanisms. The time frame for applying is any time outside the funding cycle for the programs listed above.

7.2.1.3 State and Tribal Assistance Grants (STAG Grants)

These funds are generally available through congressionally earmarked funds through the State's Congressional delegation. Funds come through the Department of Interior Appropriations Bill, within the EPA's budget categorized as "Congressionally Requested Project" STAG grants. There are approximately $400 million in STAG grants appropriated each year. These are grants of up to 55 percent of the project cost, with the rest matched by the entity or other non-federal funding source (revolving funds qualify as match). Funds can also be requested retroactively, for projects already constructed, especially if the financing of that project resulted in utility rates that create a financial hardship.

The best way to begin this process is to call the local Congressional representative's/Senator's office and talk to their chief of staff about the project, its merits, why it is important to your community. This helps you to determine if the Congressional member will be supportive of the project and take the request forward.

7.2.1.4 Community Development Block Grants

This federal program is funded by the U.S. Department of Housing and Urban Development (HUD) and administered through DOLA. The grant program is often used for small wastewater capital improvements. Colorado's Community Development Block Grant (CDBG) Program is a federally funded competitive grant program designed to help communities with their most critical community development needs. By agreement, DOLA administers the CDBG Program for local governments who do not receive funding on an “entitlement” basis directly from HUD.

Under federal law, all CDBG projects must principally benefit low- and moderate-income persons. In public facility projects, this is accomplished by making improvements to public facilities that serve communities or neighborhoods that are mostly low- or moderate-income families.

7.2.2 Local Sources of Funds for Wastewater Capital Improvements

Local funding of capital improvements relies on private financing through bond sales or cash-based financing through use of system development fees, capital reserves, and rates.

7.2.2.1 Revenue Bonds

Revenue bonds are long-term municipal bonds guaranteed solely by the dedication of project income or sewer funds (user fees) rather than by a general tax.

7.2.2.2 General Obligation Bonds

General Obligation Bonds are long-term municipal bonds that are backed by the full faith and credit of the local government. This means that Northglenn would pledge to use all of its taxing and other revenue-raising powers to repay bond holders. General obligation bonds require voter approval through a local bond election.

7.2.2.3 Capital Reserves or Cash Basis

Capital improvements may be funded through cash reserves developed through system development fees and user fees. The advantages of this approach are the avoidance of debt and interest payments.

7.2.2.4 Rate Funded Capital

Northglenn has not traditionally planned for a specific amount of project costs to be borne by current ratepayers through capital improvements funded from rates. A general rule of thumb for funding utility renewal and replacement projects is to fund a minimum level of annual depreciation expense. This “source of revenue” should be planned into Northglenn’s financial strategy to ensure that current ratepayers are helping to fund the deterioration on the infrastructure they use while receiving service. This planning philosophy has the added benefit of improving debt service coverage requirements that are required by bonding agencies if revenue bond funding is used to pay for the project costs.

7.3 Summary

Two projects are recommended for implementation over the 20-year planning cycle in this Plan. Project 1 includes replacement of existing equipment in poor condition and improvements to the existing facility addressing specific concerns from staff related to plant O&M and process performance. Costs and targeted completion date for Project 1 are as follows:

- Project 1: Condition Assessment Equipment Replacement and Improvements to Existing Treatment System
  - Total Program Cost **$ 5,500,000**
  - Target Completion Date: **March 2024**

Project 2 includes facility upgrades to increase capacity to accommodate current and future flows and loads. Project 2 was broken down into two phases for implementation. Phase 1 consists of installing primary clarifiers and new solids handling processes to increase treatment capacity of the WWTP. Construction of Phase 1 upgrades is recommended as soon as possible to accommodate current flows and loads since the plant is already operating near capacity. Phase 1 upgrades would also account for any additional loads associated with development in the SSA but not necessarily the full flow increase. Phase 2 upgrades are required to meet the projected flows and loads associated with the development of Section 36 within the NSA in addition to the development that will occur in the SSA. If it is determined that Section 36 will not be developed within the next 20 years, a more detailed flow study should be conducted to determine if the Phase 2 upgrades are required. The capacity increase associated with the Phase 1 upgrades is greater than the projected 2040 influent loads for the SSA but likely not the current projected 2040 flows for the SSA. Assuming that Section 36 will be developed, the program costs and targeted completion dates for Phases 1 and 2 are listed below:

- Phase 1: Construct Primary Clarifiers and Solids Handling Processes
  - Total Program Cost: **$ 56,000,000**
  - Target Completion Date: **2026**
- Phase 2: Construct Aeration Basins and Secondary Clarifier
  - Total Program Cost: **$37,000,000**
  - Target Completion Date: **2031**

Utilizing a phased implementation rather that constructing all upgrades at once increases the Project 2 costs by approximately 6%. However, the recommended phasing provides Northglenn the opportunity to reassess development plans in the service areas, evaluate the performance of the upgraded WWTP after Phase 1, and spread the capital cost burden over more time.

It is recommended that a rate study be conducted as soon as possible to plan for the projects included. Additional funding opportunities were described in **Section 7.2**.

Section 8

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