Wastewater Treatment Plant Evaluation

City of West Plains, Missouri

Wastewater Facility Plan
Project No. 126220

Revision 0
7/29/2021

Wastewater Treatment Plant Evaluation

prepared for

City of West Plains, Missouri
Wastewater Facility Plan

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Revision 0
7/29/2021

prepared by

Burns & McDonnell Engineering Company, Inc.
St. Louis, Missouri

INDEX AND CERTIFICATION

City of West Plains, Missouri
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Certification

I hereby certify, as a Professional Engineer in the state of Missouri, that the information in this document was assembled under my direct personal charge. This report is not intended or represented to be suitable for reuse by the City of West Plains, Missouri or others without specific verification or adaptation by the Engineer.

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LIST OF ABBREVIATIONS

Abbreviation	**Term/Phrase/Name**
BMcD	Burns & McDonnell
BNR	Biological nutrient removal
BOD	Biochemical oxygen demand
City	City of West Plains
COD	Chemical oxygen demand
DO	Dissolved Oxygen
EBPR	Enhanced biological phosphorus removal
EPA	Environmental Protection Agency
F/M	Food to microorganism ratio
gpd	Gallons per day
gpm	Gallons per minute
hp	Horsepower
HRT	Hydraulic residence time
lb/hr	Pounds per hour
MGD	Million gallons per day
MLSS	Mixed liquor suspended solids
MLVSS	Mixed liquor volatile suspended solids
NPDES	National Pollutant Discharge Elimination System
O&M	Operations and maintenance
OIT	Operator interface terminal
PAO	Polyphosphate accumulating organisms

Abbreviation	**Term/Phrase/Name**
RAS	Return activated sludge
scfm	Standard cubic feet per minute
SRT	Solids retention time
TKN	Total Kjeldahl nitrogen
TN	Total nitrogen
TP	Total phosphorus
TSS	Total suspended solids
UV	Ultraviolet
VFD	Variable frequency drive
WAS	Waste activated sludge
WWTP	Wastewater treatment plant

1.0 INTRODUCTION

1.1 Scope of Assessment

The City of West Plains (City) retained Burns & McDonnell (BMcD) to conduct an evaluation of the existing wastewater treatment plant (WWTP) to support the development of a Facility Plan. The Facility Plan identifies required improvements to the WWTP through the planning year of 2040. Systems evaluated include influent screening, influent pumping, peak flow handling, secondary treatment, clarification, tertiary filtration, disinfection, solids handling, and ancillary facilities. Implementation of future processes or process modifications to accommodate National Pollution Discharge Elimination System (NPDES) permit changes, including biological nutrient removal (BNR), were also considered. The drivers for the improvements identified in this memorandum are based on anticipated regulations, growth projections, and capacity and performance-related issues.

1.2 Service Area Description

The City of West Plains is located in central Howell County, Missouri, near the southern border of the state. The West Plains WWTP treats wastewater generated by residential, commercial, and industrial customers. The WWTP is currently rated to treat an average day flow up to 3.0 million gallons per day (MGD). The WWTP was designed for a peak flow capacity of 7.0 MGD, but actual throughput is limited to approximately 3.5 MGD.

2.0 CONDITION AND CAPACITY EVALUATION

2.1 Introduction

The existing WWTP was constructed in 1979 as an extended aeration activated sludge process; it underwent significant improvement projects in 1998 and 2002. The WWTP is rated for an average day flow of 3.0 MGD and a peak flow of 7.0 MGD. The evaluation of existing facilities is divided into the following categories: headworks, secondary treatment, tertiary treatment, disinfection, and solids handling.

The headworks includes a mechanical screen, influent pumps, peak flow pumps, a peak flow clarifier, and grit removal. The liquid stream treatment process includes anaerobic selector basins, an activated sludge oxidation ditch, final clarifiers, traveling bridge sand filters, and ultraviolet (UV) disinfection. The maximum capacity of the influent pump station is 6.25 MGD. During peak events, 6.25 MGD is conveyed to the liquid stream treatment process via plant influent pumping, and up to 10 MGD is conveyed to the peak flow system via the peak flow pump station. Influent flows over 3.5 MGD threaten compliance with effluent E. coli limits and start to cause structures to backup due to undersized yard piping. Refer to Figure 2-1 for a process flow schematic.

The solids handling process provides stabilization and storage of solids wasted from the activated sludge treatment system. The solids handling process includes waste activated sludge (WAS) pumping and six (6) aerobic digesters. The aerobic digestion system is designed to meet the Environmental Protection Agency (EPA) Part 503 regulations for Class B biosolids; following stabilization, biosolids from the digesters are land applied on local agricultural sites.

2.2 Headworks

The headworks includes one (1) mechanical bar screen, two (2) plant influent pumps, three (3) peak flow pumps, and grit removal. The ¼-inch mechanical bar screen removes solids from the influent wastewater prior to being pumped to the grit removal system. Hydraulic capacity through the screen is 13 MGD and 6.25 MGD through each influent pump. Plant influent pumps convey up to 6.25 MGD to the aerated grit and grease removal process (rated for 6.25 MGD), and peak flow pumps convey up to 10 MGD to the peak flow clarifier.

	A*	B	C	D	E	F	G	H	I	J
Existing Design Capacity	10 MGD	6.25 MGD (firm) 12.5 MGD (total)	7.0 MGD (firm) 10.5 MGD (total)	7.0 MGD	6.25 MGD	3.0 MGD (ADF) 6.25 MGD (Peak)	2.5 MGD (ADF) 6.25 MGD (Peak)	2.08 MGD (per Clarifier) 6.25 MGD (total)	3.0 MGD (per filter) 9.0 MGD (total)	3.5 MGD (Peak)

* Based on O&M manual for newly purchased Duperon screen

FIGURE 2-1
LIQUID STREAM PROCESS FLOW DIAGRAM

2.2.1 Influent Screening

Influent screening consists of a mechanical bar screen and washer/compactor (Figure 2-2). Wastewater is conveyed to the WWTP via two 24-inch gravity interceptors and flows through a ¼-inch mechanical bar screen, which was installed in 2020. Debris collects on the surface of the screen and are lifted out of the channel into a washer/compactor. The washer/compactor removes organics and dewaters the screenings prior to disposal. Dewatered screenings are collected in a dumpster, adjacent to the compactor and hauled to a landfill for disposal. Wash water containing organics drains back to the influent channel. The mechanical screen is sized for a peak flow of 10 MGD, however, the downstream hydraulics limit the hydraulic throughput. The screen channel depth is not sufficient, causing the baseline water level within the channel to be too high. A bypass pipe was constructed following the 2005 improvements project that conveys flow around influent screening to the alleviate hydraulic bottleneck resulting from insufficient channel depth. Operations staff lack the capability of isolating the screening bypass. Thus, during peak wet weather events, a significant portion of the influent flow is unscreened prior to secondary treatment.

Figure 2-2: Influent Screening

The screening equipment is installed outdoors and subject to freezing during the winter months. Operations staff indicated that drive shafts have sheared in the past due to freezing. Enclosing influent screening is recommended to protect the integrity of equipment.

Below is a summary of the mechanical screen characteristics:

- Manufacturer: Duperon
- Quantity: 1 Mechanical Screen
- Bar Spacing: ¼-inch
- Channel Width: 6 feet
- Channel Depth: 8 feet, 6 inches
- Horsepower: ½ hp (screen) and ¾ hp (washer/compactor)
- Hydraulic Capacity: 10 MGD

2.2.2 Influent Pumping

Two influent screw pumps were installed in the original 1979 construction (refer to Figure 2-3). The pumps are each rated for 6.25 MGD and 18 feet; thus, firm capacity for influent pumping is 6.25 MGD. During this evaluation, one of the screw pumps was out of service, and the WWTP was operating on a single pump. The screw pump structure is equipped with a high-level float that will alarm and notify operators when the water level upstream of the pumps exceeds the high-level elevation.

At the time of the site investigation, operations staff indicated that the operational pump requires a replacement flight. The existing flight is in poor condition, resulting in excess wastewater fallback and a reduction in pumping efficiency. The influent pumps have exceeded their designed useful life and require replacement. During original construction, the concrete channels were not formed properly, which significantly impacts pumping capacity. Since construction, the City has added grout to reform the left channel; however, concrete work is still needed to render the pumps operable and efficient.

Below is a summary of the influent pump characteristics:

- Manufacturer: Passavant
- Quantity: 2 Pumps
- Screw Size: 48-inch diameter
- Capacity: 6.25 MGD each
- Head: 18 feet
- Horsepower: 30 hp
- Speed: 45 rpm

Figure 2-3: Influent Pump Station

2.2.3 Peak Flow Pump Station and Clarifier

The peak flow system consists of a peak flow pump station and a peak flow clarifier. The system was originally constructed as part of the 1998 improvements project. The pump station is equipped with three 40-hp pumps, each rated for 2,500 gpm and 45 feet of head, and the pump station has a firm capacity of 7 MGD. Excess flow is diverted via an overflow weir within the flow splitter structure directly downstream of the screening channel. This flow is conveyed to the peak flow treatment system via a 24-inch pipe that discharges into the wet well of the peak flow pump station.

Figure 2-4: Peak Flow Pump Station

Flow is discharged from the peak flow pump station via a 12-inch header, which conveys wastewater to the peak flow clarifier. The plant currently has one peak flow clarifier, which is 96-foot in diameter and designed to treat up to 7.0 MGD. Clarified wastewater is gravity discharged back into the treatment system downstream of the influent pump station. Discharges from the peak flow clarifier were previously permitted via Outfall 002; the WWTP’s current Missouri State Operating Permit (MO-0096610) no longer includes Outfall 002 as a permitted discharge. Prior to the 2005 improvement project, the peak flow clarifier effluent was routed to the effluent mixing structure, downstream of UV disinfection. The Peak Flow Clarifier effluent piping was later rerouted to the filter splitter structure, to facilitate blending of clarified peak flow wastewater with treated effluent. However, operations staff indicated that blending did not function as designed due to hydraulics.

The peak flow clarifier has not been active for approximately six years, due to difficulty of operation. As part of the 2005 improvement project, a blower was added to provide air to the peak flow clarifier for odor control. Sludge collected in the clarifier has historically caused odor issues, due to holding time in the clarifier and inability to waste from the peak flow clarifier during wet weather events. The water surface elevation in the filter splitter is too high, relative to the peak flow clarifier, to allow gravity discharge from the peak flow clarifier during rain events. Further, the piping between the peak flow clarifier and the filter splitter structure is undersized. Piping improvements are required to restore full performance of the peak flow system.

Below is a summary of the peak flow treatment system characteristics:

- Peak Flow Pump Station
  - Manufacturer: Flygt
  - Quantity: 3 Pumps
  - Capacity: 3.5 MGD each
  - Head: 45 feet
  - Horsepower: 40 hp
- Peak Flow Clarifier
  - Manufacturer: Environmental Equipment & Systems, Inc.
  - Mechanism Type: Spiral-blade
  - Quantity: 1 Clarifier
  - Capacity: 7.0 MGD
  - Horsepower: 0.75 hp

2.2.4 Grit Removal

The grit removal system was originally constructed as a vortex grit removal process. However, the vortex mechanical components were abandoned as part of the 1996 improvements project, and the system was converted to a passive grit removal system (Figure 2-5). A grit chamber with a plug-flow regime was constructed with a sloped bottom. This grit system relies on maintaining a sufficient velocity and holding time within the chamber to allow grit particles to settle. To remove grit from the chamber, operators open an aluminum slide gate on the side of the chamber. Water is drained to the influent pump station, and the grit is manually loaded into a truck to haul off-site.

Figure 2-5: Existing Grit Chamber

The grit handling system was in operation at the time of the site investigation; however, the performance of the system is unknown. Operations staff suspect there is significant grit accumulation in the aeration basin. A Parshall flume is installed downstream of the grit removal system for influent flow measurement and is currently programmed for a maximum reading of 7.0 MGD. Operations staff suspect there have been numerous wet weather events when flow has exceeded 7.0 MGD but has been improperly recorded by the influent flume. If possible, the level transmitter should be reprogrammed to allow flows higher than 7.0 MGD to be recorded.

2.2.5 Summary of Existing Headworks

Table 2-1 provides a summary of the existing headworks equipment and operational concerns.

Table 2-1: Summary of Existing Headworks

Facility	Hydraulic Capacity	Notable Issues
Influent Screening	10 MGD (rated)	Acts as hydraulic bottleneck during peak wet weather events.
Influent Pump Station	6.25 MGD	Screw pump structure requires concrete work to reform channels Screw pumps require replacement due to age and condition
Peak Flow System	7.0 MGD (pumped) 7.0 MGD (treatment)	Undersized piping and water level elevation in the peak flow clarifier compared to the filter splitter inhibit full functionality of the system and ability to blend. This results in long holding times in the peak flow clarifier, leading to odors.
Grit Removal	7.0 MGD	Performance is unknown

2.3 Secondary Treatment

The liquid stream treatment process includes three anaerobic selector basins, one activated sludge oxidation ditch, and three final clarifiers. Screened and de-gritted influent wastewater and return activated sludge (RAS) from the final clarifiers combine in the anaerobic selector basins, which select for phosphorus accumulating organisms (PAO) and reduce filamentous growth in the oxidation ditch. The selector basins discharge into the oxidation ditch, which was designed as an extended aeration activated sludge process. The oxidation ditch includes brush and surface aerators to transfer oxygen to the process and facilitate biochemical oxygen demand (BOD) removal and nitrification (conversion of ammonia to nitrite/nitrate). Mixed liquor is conveyed by gravity to the final clarifiers for solids separation. A portion of the settled mixed liquor in the final clarifiers (sludge) is returned to the selector basins, and a portion is wasted to the solids handling process. Clarified effluent flows by gravity to the filtration process.

2.3.1 Anaerobic Selectors

The anaerobic selector basin consists of three cells, each with a volume of approximately 95,000 gallons. The selector basin was constructed in the 2005 expansion to facilitate phosphorus removal. Under anaerobic conditions, the PAOs present in the MLSS can outcompete other microorganisms for the readily biodegradable components in the influent flow and store this material as food. Under aerobic conditions (in the aeration basin) the PAOs utilize the stored food and accumulate polyphosphate at a higher rate than typical activated sludge bacteria in a process called luxury uptake. Phosphorus is removed from the system during sludge wasting.

RAS is pumped to the first cell of the selector basin, where it flows over a weir to Cell 2. In Cell 2, the RAS is combined with influent wastewater from the grit removal system. From Cell 2, the mixture flows over a weir to cell 3, where it is then conveyed to the aeration basin. The selector basins were designed for an average day flow of 3.0 MGD and maximum treatment capacity of 3.5 MGD. Each of the three cells are equipped with a submersible mixer to maintain solids suspension. The selector basins have sufficient capacity for biological phosphorus removal under current design flows and loadings.

Figure 2-6: Anaerobic Selector Cells 2 & 3

Operations staff indicated that the selector basins flooded shortly after construction due to insufficient hydraulic capacity in the piping from cell 3 to the aeration basin. To provide relief for the hydraulic bottleneck, a channel was constructed to convey flow to the aeration basin (Figure 2-7).

The selector basin and its equipment were in operable condition at the time of the site investigation. Below is a summary of the selector basin characteristics:

- Total Volume: 285,000 gallons
- Solids Retention Time: 1 day
- Hydraulic Residence Time: 2.3 hours at average day flow; 1 hour at peak day flow
- Mixer Manufacturer: Wilo
- Mixer Quantity: 3 (one in each cell)
- Horsepower: 2.7 hp

Figure 2-7: Anaerobic Selector Overflow Channel

2.3.2 Aeration Basin

The secondary treatment process consists of one activated sludge oxidation ditch, which was constructed in 1979. The oxidation ditch was designed as an extended aeration process, which uses a longer solids retention time (SRT) to achieve BOD removal and nitrification. The process was originally designed for an average day flow of 2.5 MGD and a maximum flow of 6.25 MGD. The average day treatment capacity was subsequently increased to 3.0 MGD with the construction of an additional clarifier in the 2005 improvements. However, during peak wet weather events, the aeration basin overtops due to the basins weir elevation and undersized piping downstream of the aeration process.

Passavant brush rotors provide aeration and mixing in the aeration basin. The rotors are dual-purpose: they provide aeration and mixing. An in-channel velocity of one foot per second is targeted to maintain solids in suspension. Oxygen transfer efficiency of brush rotors is directly related to rotor submergence – as submergence increases, transfer efficiency increases. The basin's effluent box is equipped with level control weirs that can be adjusted manually to control rotor submergence. The rotors are constant speed and lack the ability for dissolved oxygen (DO) control.

The system was designed to operate based on one of two parameters: (1) food to microorganism ratio (F/M) or (2) solids retention time (SRT). The F/M ratio is the ratio of influent BOD loading to total mass of mixed liquor volatile suspended solids (MLVSS) in the basins. The aeration basin was designed to operate in the 0.07 to 0.11 range, as the process is an extended aeration process. Further, the basin was designed for an operating mixed liquor suspended solids (MLSS) concentration between 2,000 mg/L and 3,000 mg/L, depending on the influent organic loading. The SRT is the approximate age of biomass contained in the activated sludge system. The oxidation ditch system was originally designed for an SRT of 26 days. Operations staff indicated during the site investigation that current operation and wasting is controlled by MLSS concentration; MLSS is maintained around 3,400 mg/L to optimize nitrification performance.

During the site investigation, operations staff indicated several issues related to the aeration basin and its equipment:

- The brush rotor gear boxes frequently fail due to elevated water surface elevations during peak flow events, which floods the platforms housing the gear boxes (Figure 2-8). Hydraulic bottlenecks downstream of the oxidation ditch process cause the flooding. Two floating aerators were added to the oxidation ditch to provide supplemental air. During the site investigation, all brush rotors were operable and one floating aerator was in use to supplement air supply.

- The brush rotors generate a significant amount of splashing at the water surface. When operation staff are servicing the rotors or are walking the bridges across the basin, they are at risk of direct exposure to airborne wastewater (refer to Figure 2-9).

- The walkway bridges are sagging in the center due to deflection caused by ice formation in the winter months (refer to Figure 2-10). The walkway thickness does not meet code recommended thicknesses for control of deflection.

- The secondary treatment process has a single oxidation ditch, and plant staff are unable to take the basin offline while maintaining compliance with effluent permit limits. It is suspected that a significant amount of grit may have accumulated at the bottom of the tank, which may reduce treatment volume.

Below is a summary of the aeration basin design characteristics:

- Manufacturer: Kruger
- Trains: 1 Oxidation Ditch
- Basin Volume: 2.35 MG
- Capacity: 3.0 MGD average day
  6.25 MGD peak day

- Quantity of Aerators: 4 brush rotors
- Aerator Horsepower: 50 hp
- Oxygen Requirement: (SOR): 415 lb/hr

Figure 2-8: Aeration Basin Flooded Gearbox Platform

Figure 2-9: Brush Aerator Splashing

Figure 2-10: Aeration Basin Sagging Bridge

2.3.3 Final Clarification

The three final clarifiers are 60-ft in diameter; Clarifiers 1 and 2 were constructed in 1979 with the original WWTP, and third clarifier was constructed in 2005 (Figure 2-11). Mixed liquor from the aeration basin is split between the clarifiers by three weirs in the mixed liquor flow splitter box. Each clarifier was designed for an average day flow (with recycle) of 1.5 MGD and a maximum flow (with recycle) of 3 MGD. During the site visit, all three clarifiers were operational and appeared to be in adequate condition. Recoating the clarifier mechanisms is recommended to extend the useful life of the equipment.

Figure 2-11: Final Clarification

Scum is skimmed from the top of each clarifier and directed to a scum box, attached to each clarifier. Scum is wasted from the box into the clarifier’s sludge line via a telescoping valve. Constant-speed submersible pumps are installed in a wet well adjacent to Aerobic Digester 2. Return sludge from

Clarifier 3 discharges directly into the pump station wet well. Sludge from clarifiers 1 and 2 discharges into the old sludge pump station wet well (west of the new pump station), which then overflows via a 24-inch pipe into the active sludge pump station. Return activated sludge (RAS) versus WAS pumping is controlled by a manually operated valve on the RAS discharge line. Consideration should be given to adding variable frequency drives (VFDs) to the RAS/WAS pumps and an electric actuator to the valve that controls the wasting rate to enable tighter process control.

The RAS/WAS pumps were functional during the site investigation, and the operators indicated they have not had issues with the pumps. A summary of the final clarifier equipment and capacities is provided below.

- Manufacturer: DBS Manufacturing
- Quantity: 3
- Diameter: 60-ft
- Solids Loading Rate (each): 40 lb/day/ft^2 (at 4,000 mg/L MLSS, 7.0 MGD influent, 3.0 MGD RAS)
- Surface Overflow Rate (each): 825 gpd/ft^2 (at 7.0 MGD influent)
- Horsepower: 0.5 hp (each)

Below is a summary of the return and waste pump characteristics:

- Manufacturer: Wilo
- Quantity: 3 (2 duty / 1 standby)
- Capacity: 1562 gpm each
- Head: 23 feet
- Horsepower: 14.8 hp

2.3.4 Summary of Existing Secondary Treatment

Table 2-2 provides a summary of the existing secondary treatment processes and operational concerns.

Table 2-2: Summary of Existing Secondary Treatment Process

Facility	Capacity	Notable Issues
Anaerobic Selector	3.0 MGD	Overflow channel constructed from Cell 3 to aeration basin due to hydraulics.
Aeration Basin	2.5 MGD	Frequently floods during wet weather events. Insufficient volume for design BOD and ammonia loading. Rotors frequently require repair and are inefficient in terms of oxygen transfer. Significant airborne wastewater near rotors, which is a safety concern. Basin bridges require rehabilitation, as structural integrity has deteriorated.
Final Clarification	7 MGD	Clarifier mechanisms require re-coating
RAS/WAS Pumping	4.5 MGD	Consider adding actuated valve to control sludge wasting

2.4 Tertiary Filtration

Due to the effluent BOD, total suspended solids (TSS) and total phosphorus (TP) limits, tertiary filtration is required. Effluent from the final clarifiers is conveyed to the filters for additional suspended solids removal. The filter system consists of three channels, each rated for 3.0 MGD at a hydraulic loading rate of 5 gpm/ft^2. The filter system is designed to meet average day flow conditions with one channel in service and peak flow conditions with all three channels in service. The filter consists of a sand media bed and traveling bridge backwash system. The backwash system uses submersible pumps to transfer filtered effluent through the media bed to wash organics, scum, and floatables that have accumulated in the filter into a waste trough. The waste trough is piped to the plant sewer, which ultimately discharges to the influent pump station.

Operations staff indicated filtration acts as a hydraulic bottleneck during peak events, primarily at the filter splitter structure. The filter influent piping is undersized and limits the hydraulic throughput of the process. The traveling bridge and underdrains on Filter 1 and Filter 2 have exceeded their useful life and are nearing the end of their useful life for Filter 3. Further, many sand filtration installations are being replaced with cloth filters, due to superior performance and smaller footprint. Cloth media filters have approximately double the hydraulic throughput while maintaining historical removal efficiency of solids.

Figure 2-12: Tertiary Sand Filters 1 and 2

A summary of the filter equipment and capacities is provided below.

- Quantity: 3 Filters
- Hydraulic Loading Rate: 5 gpm/ft^2 (at 3.0 MGD per filter)
- Hydraulic Capacity: 9.0 MGD (total)
- Horsepower: Traveling Bridge: 0.75 hp (each), Backwash Pump: 1.5 hp (each), Water Wash Pump: 1.5 hp (each)

2.5 Disinfection

UV disinfection was installed to replace the chlorine disinfection system as part of the 2005 improvement project. UV light is emitted from lamps and results in photolysis, which inactivates bacteria and viruses and makes them unable to reproduce. The existing system is a low-pressure, high-output, horizontal configuration that consists of two UV banks in a single channel (Figure 2-13). The lamps are flow paced with the effluent flow, which is measured by a Parshall flume downstream of the disinfection system. The current system was designed to achieve an effluent *E. coli* of 126 CFU per 100 mL at a peak flow of 3.5 MGD.

The UV system is not providing the level of disinfection required for peak conditions and does not have sufficient hydraulic throughput. Effluent *E. coli* concentrations exceeded the effluent permit limit seven

(7) times over the analysis period of January 2016 through August 2020. The system is a hydraulic bottleneck for the plant during peak wet weather events. Further, the UV structure (Figure 2-13) was constructed in the floodplain, below the 100-year flood elevation. The structure has flooded multiple times since construction.

Figure 2-13: Open Channel UV Disinfection System

A summary of the UV equipment and capacities is provided below.

- Manufacturer: Trojan
- Quantity: 2 banks each with 6 modules
- Design Peak Flow: 3.5 MGD
- System Type: Low pressure, high output
- Design Effluent: 126 CFU /100 mL

2.6 Solids Handling

As discussed in Section 2.3.3, sludge drawn off the bottom of the clarifiers flows by gravity to the sludge pump station. Wasting is controlled by a manually operated valve on the RAS discharge header. The valve position is adjusted to maintain the return and wasting rates set by the operator. Operators have historically wasted an average of 25,000 gpd to maintain an MLSS of 3,400 mg/L in the aeration basin. WAS is pumped to the sludge control valve vault, where it is directed to one of six aerobic digesters.

Each digester is 35-foot square with a sidewater depth of 10 feet (Figure 2-14). The digesters serve to store, stabilize, and thicken sludge prior to land application. The digesters are operated in parallel and were designed to achieve an SRT of 60 days, which is required to qualify as a Process to Significantly Reduce Pathogens (PSRP) under the EPA 503 Class B requirements. However, the existing volume and a 60-day SRT can accommodate an influent flow rate of 1.5 MGD and maximum month BOD concentration of 250 mg/L. The digesters have insufficient volume for the plant's design flow of 3.0 MGD, and in order to continue meeting Class B biosolids requirements, additional digester volume is needed.

Air is provided to each digester through dedicated fine bubble diffused aeration grids. Each digester aeration grid is equipped with 28 one-meter flexible tube diffusers. The system is designed to deliver 420 scfm per digester. Three positive displacement blowers provide air to the six digesters. Each blower is sized for 420 scfm; they operate with two duty blowers and one standby.

Each digester is equipped with three shear gates and decant lines to enable decanting and thickening of the sludge. The first shear gate is located 2.75 feet below the top of wall with the next two spaced at 1.5 ft intervals. The digesters thicken the sludge to approximately two percent, and the digester supernatant is conveyed by gravity to the sludge pump station. Process equipment appeared to be in good working condition at the time of the site investigation; however, the blower control panel has significant damage, and the operator interface terminal (OIT) was not working. Replacement of these is recommended. Further, operations staff indicated that land availability for biosolids application has been an issue in the past due to wet soil conditions.

Figure 2-14: Aerobic Digester with Diffused Aeration

A summary of the solids handling equipment and capacities is provided below. Consideration should be given to incorporating a solids dewatering process to reduce the volume of biosolids prior to hauling.

- Digestion Design SRT: 60 days
- Design Solids Concentration: 2%
- Design DO Concentration: 2 mg/L
- Manufacturer: EDI (Diffusers)
  United Blower (Blowers)
- Quantity: Six digesters and three blowers
- Design Airflow (per blower) 420 scfm
- Blower Horsepower: 50 hp (each)

2.7 Hydraulics and Yard Piping

Piping throughout the WWTP is undersized and causes hydraulic bottlenecks throughout the treatment system. The WWTP has two 24-inch mains that convey influent from the collection system to influent screening. Due to the poor downstream hydraulics and insufficient channel depth in the screen channels, the influent mains backup into the collection system and act as storage under normal operating conditions.

During peak events, the manhole downstream of influent screening (MH-1) frequently overflows during peak wet weather events. The existing influent pumping structure also does not have sufficient depth to allow enough throughput; the wet well level is too high relative to the downstream water level in the screen channel. Increasing the depth of the structure and reducing friction loss through the influent piping would reduce backups at MH-1.

The influent line to the aeration basin is 18 inches and provides insufficient hydraulic throughput. Operations staff indicated that shortly after startup of the anaerobic basin, it overtopped, and construction of a channel within the drive between the anaerobic basin and the aeration basin was constructed to alleviate the bottleneck. Similarly, the aeration basin frequently overtops during peak wet weather events. Effluent leaves the aeration basin via a 14-inch pipe that increases to an 18-inch. The pipe should be increased to a 30-inch in order to provide sufficient hydraulic capacity during peak events.

With the 2005 improvements, a line was installed to route effluent from the peak flow clarifier to the filter splitter structure to facilitate blending. However, operations staff indicated blending is not feasible, as they are unable to flow by gravity between the peak flow clarifier and the filters. The 16-inch pipe provides insufficient capacity and requires replacement to provide the operational flexibility of blending clarified peak flow wastewater with fully treated wastewater prior to discharge.

A hydraulic profile of the existing WWTP was developed to identify the piping segments and/or structures that require modification to eliminate hydraulic bottlenecks. Table 2-3 summarizes the recommended piping improvements to improve hydraulics throughout the treatment process.

Table 2-3: Piping Improvements Required to Alleviate Hydraulic Bottlenecks

Upstream Structure	Downstream Structure	Existing Pipe Size	Recommended Pipe Size
Influent Pump Station	Anaerobic Basin	18-inch	**24-inch**
Anaerobic Basin	Aeration Basin	18-inch (plus channel)	**30-inch**
Aeration Basin	Final Clarifier Splitter	Part 14-inch, Part 18-inch	**30-inch**
Final Clarifier Splitter	Final Clarifiers	Part 14-inch, Part 18-inch	**20-inch**
Final Clarifier Effluent	Filter Splitter	18-inch	**30-inch**
Filter Splitter	Filter 1, 2	14-inch	**20-inch**
Filter Effluent (combined)	UV Disinfection	20-inch	**30-inch**
UV Disinfection	Effluent Mixing Structure	20-inch	**30-inch**
Peak Flow Clarifier	Filter Splitter	16-inch	**24-inch**

3.0 WATER QUALITY AND DISCHARGE PERMIT ASSESSMENT

The following sections describe current and anticipated plant loadings and regulatory requirements. This section also discusses the design basis for the WWTP improvements.

3.1 Current Flows and Loadings

Data from 2016 through August 2020 was used to develop the current wastewater quality characterization for the West Plains WWTP. Daily flow data was analyzed to develop average day, maximum month, maximum day, and peak flow rates, and pollutant data was analyzed to develop average day, maximum month, and maximum day loading rates. Pollutants of interest included total suspended solids (TSS), five-day biochemical oxygen demand (BOD), ammonia-nitrogen, total Kjeldahl nitrogen (TKN), and total phosphorus (TP). At the WWTP, flow is automatically logged daily, while TSS and BOD are measured once per week. Influent ammonia, TKN, and TP are not currently monitored.

For all analyses, the median (50 percentile) of the data set was used to report the average day flow or loading, as this approach tends to reduce the impact of extreme values when compared to the use of mean values. The maximum month condition was determined by developing a continuous set of data consisting of running 30-day average values, and then identifying the largest value of that group. Maximum day loading for all analyses was determined using the 99.7 percentile (equivalent to 364/365) of the data set. Because the data set did not include hourly data, hydraulic modeling of the collection system was conducted to estimate the maximum flow to the WWTP. The statistical analyses used for determining flow and loading parameters are summarized in Table 3-1.

Table 3-1: Statistical Analyses Used for Flow and Loading Conditions

Parameter	Flow	Pollutant Loadings[a]
Average Day	Median (50 Percentile)	Median (50 Percentile)
Maximum Month	30-d Running Average	30-d Running Average
Maximum Day	99.7 Percentile	Maximum Value
Peak	Hydraulic Modeling	-

[a] Pollutant loading analyses conducted after data trimming as described in this memorandum

Operations data from January 2016 through August 2020 was analyzed, including flow, BOD, and TSS. Historical influent flow and BOD and TSS loadings are included in Figure 3-1, Figure 3-2, and Figure 3-3, respectively. This data was supplemented with targeted sampling, collected from September 14, 2020 through September 25, 2020, which comprised BOD, chemical oxygen demand (COD), TSS, volatile suspended solids (VSS), alkalinity, total phosphorus (TP), ortho-phosphate, total Kjeldahl nitrogen

(TKN), and ammonia (NH_3) to support modeling efforts of the secondary treatment process. Table 3-2 summarizes the remaining results of the additional sampling requested.

Figure 3-1: Influent Flow from January 2016 Through August 2020

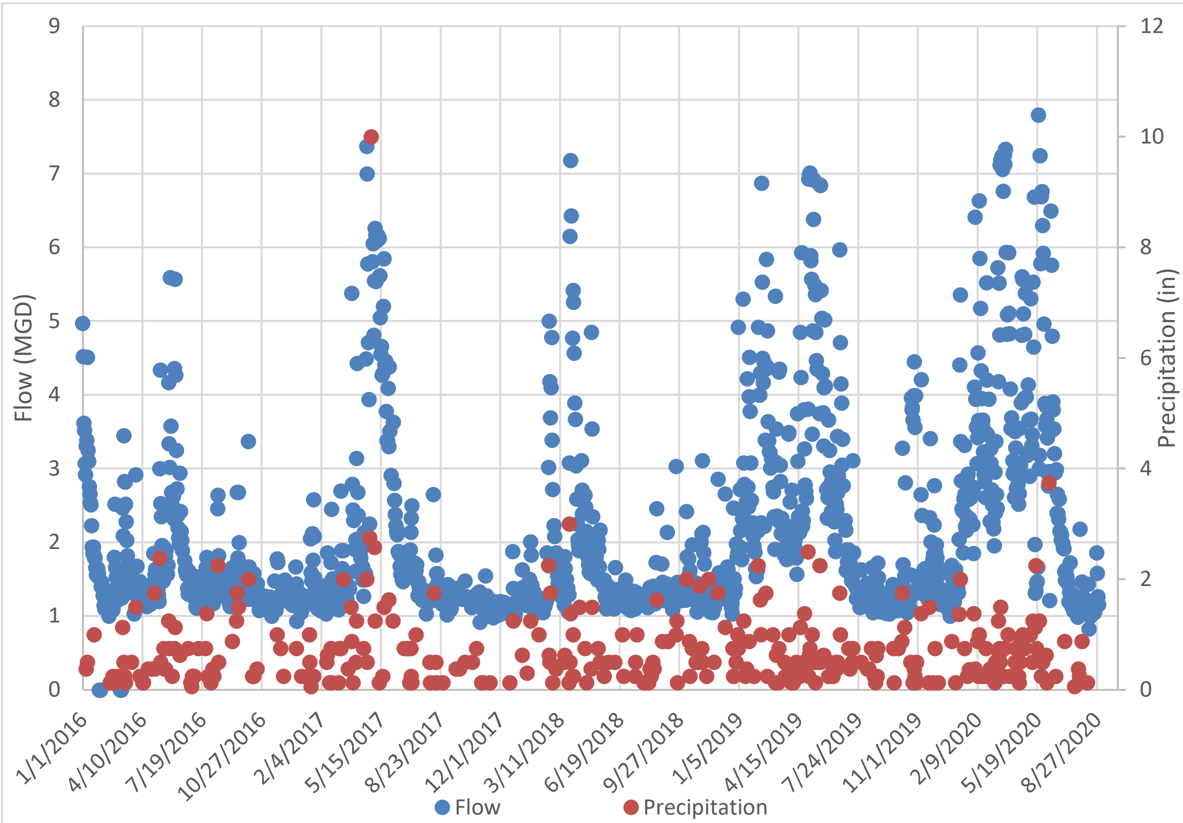


Figure 3-2: Influent BOD from January 2016 Through August 2020

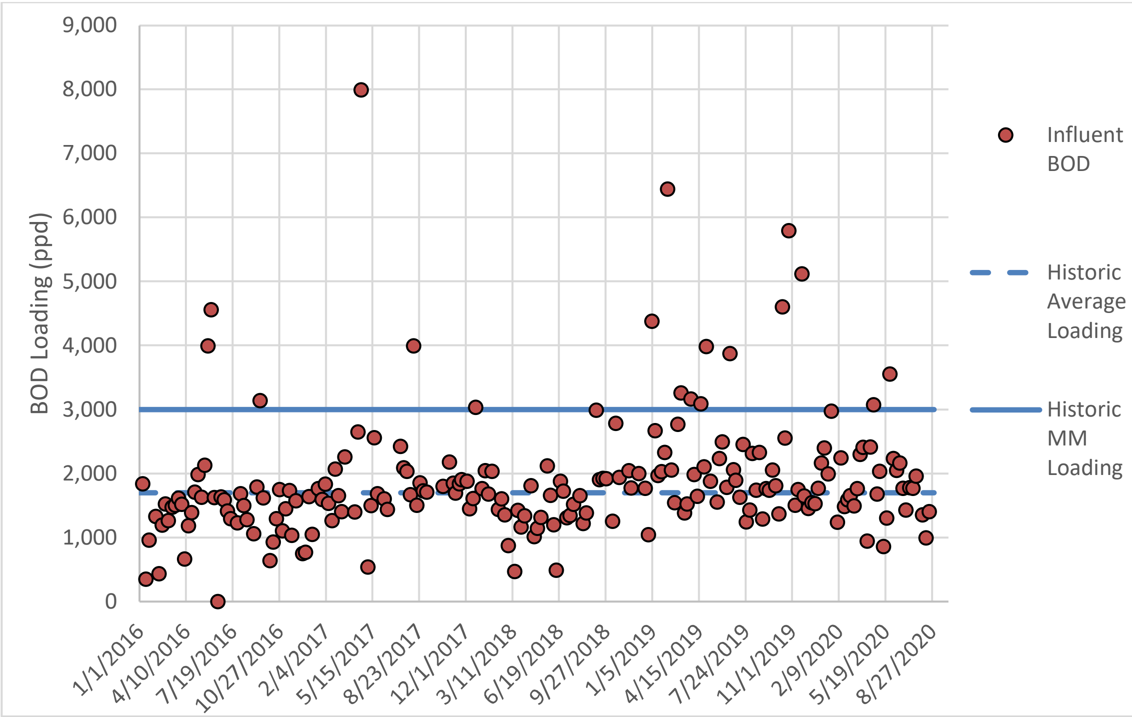


Figure 3-3: Influent TSS from January 2016 Through August 2020

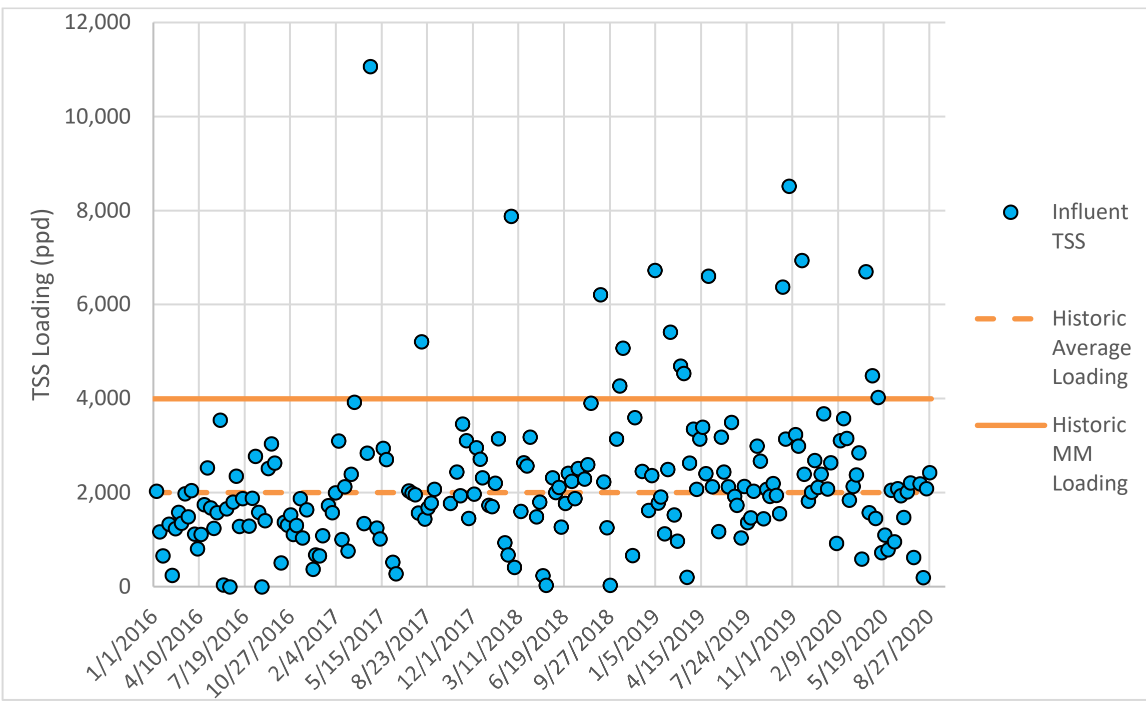


Table 3-2:Summary of Supplementary Sampling Data

Constituent	Average	
	mg/L	lb/day
COD	430	4,700
TKN	48	530
NH_3	13	145
TP	5.1	55
Ortho-Phosphate	3.9	33
Alkalinity	330	-

3.2 Discharge Permit Requirements

The WWTP currently operates under Missouri State Operating Permit MO-0096610, which became effective October 1, 2018 and expires March 31, 2023 (Appendix A). The permit allows discharge from Outfall #001 to Howell Creek, which is classified as a losing stream. The permit contains final effluent limits and monitoring requirements for BOD, TSS, *E. coli*, ammonia, and oil & grease. Table 3-3 shows a summary of effluent limitations and monitoring requirements for the WWTP.

Table 3-3: Summary of Effluent Limitations and Monitoring Requirements

Constituent	Daily Maximum	Weekly Average	Monthly Average
BOD (mg/L)	-	15	10
TSS (mg/L)	-	20	15
E. Coli (#/100 mL)	126	-	Monitor
Ammonia as N (mg/L) April 1 – Sept 30	5.9	-	1.2
Ammonia as N (mg/L) Oct 1 – Mar 31	10.9	-	2.1
Oil & Grease (mg/L)	15	-	10
Bis (2-ethylhexyl) phthalate (µg/L)	Monitor	-	Monitor
Cyanide, Amenable to Chlorination (µg/L)	Monitor	-	Monitor
Total Phosphorus	Monitor	-	Monitor
Total Nitrogen	Monitor	-	Monitor

3.3 Anticipated Discharge Limits

In the future, permit limits will become progressively more restrictive as additional constituents are included and effluent limits are reduced. The following sections discuss potential ammonia and nutrient removal requirements.

3.3.1 Ammonia

On Aug. 22, 2013, the U.S. Environmental Protection Agency (EPA) finalized new water quality criteria for ammonia based on findings regarding the negative effect of elevated levels of ammonia on mussel species. Under the current water quality standard, ammonia effluent limitations for a facility discharging to a stream are 3.6 mg/L daily maximum and 1.4 mg/L monthly average during the summer months, and 7.5 mg/L daily maximum and 2.9 mg/L monthly average during the winter months. Future criteria mandated by the EPA may be reduced to 1.7 mg/L daily maximum and 0.6 mg/L monthly average during the summer months, and 5.6 mg/L daily maximum and 2.1 mg/L monthly average during the winter months[1] (Table 3-4). These new ammonia effluent criteria apply to facilities that discharge to receiving streams where mussels are present or expected to be present and no dilution is provided.

Table 3-4: EPA Ammonia Criteria

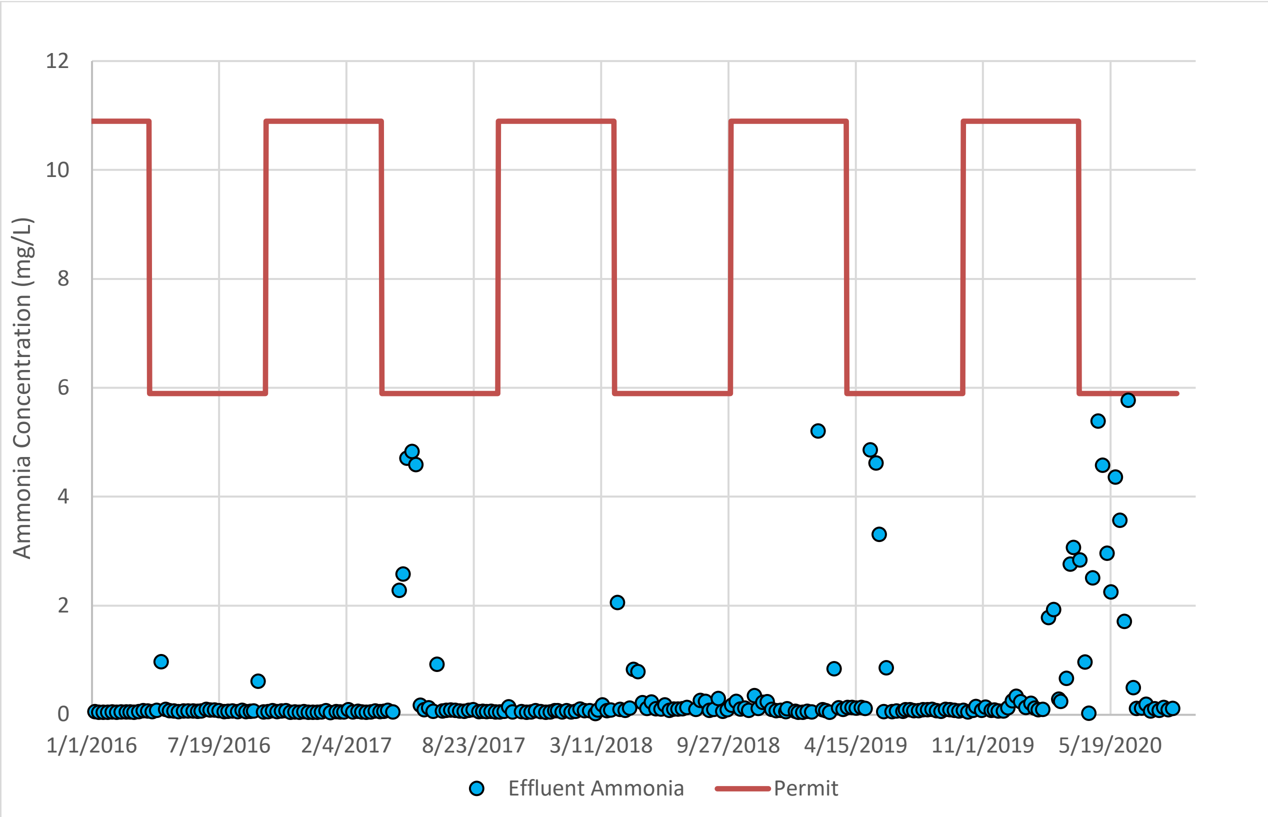
	Current Standards		Future Standards	
	Daily Max	Monthly Avg.	Daily Max	Monthly Avg.
Summer	3.6 mg/L	1.4 mg/L	1.7 mg/L	0.6 mg/L
Winter	7.5 mg/L	2.9 mg/L	5.6 mg/L	2.1 mg/L

The process of incorporating new ammonia criteria into Missouri regulations has begun but will take some time. There will likely be consideration for how long it may take to meet more stringent requirements through a legally binding mechanism, such as an NPDES permit.

Based on the 2016 through 2020 data collected for effluent ammonia at the WWTP, new criteria in Table 3-4 appear achievable based on modeling of the improvements described in Section 4.0. Most samples from 2016 to 2020 reported an ammonia discharge from the plant less than 0.10 mg/L (Figure 3-4). The instances where a daily discharge was observed to have exceeded the potential future limitations, a large peak wet weather event occurred and upset the biology in the secondary treatment system.

[1] Ammonia Criteria: New EPA Recommended Criteria. (2014, February). Retrieved December 13, 2017, from https://dnr.mo.gov/pubs/pub2481.htm

Figure 3-4: Historic Effluent Ammonia Compliance From January 2016 Through August 2020



3.3.2 Nutrient Removal Regulatory Requirements

Regulatory agencies are implementing more stringent effluent nutrient limitations (total nitrogen and total phosphorus) to protect water quality. In 2004, the EPA mandated all states develop nutrient water quality criteria. In response to this mandate, the MDNR developed a plan for developing nutrient criteria in 2005 for total nitrogen and total phosphorus. MDNR is currently amending 10 CSR 20-7.031 to include a blanket 0.5 mg/L total phosphorus limit facilities contributory to lakes and reservoirs; after these criteria have been developed, the focus will shift to facilities contributory to rivers and streams. Criteria will be based on a statistical review of stream data, a USGS study of algae response to nutrients in the Ozarks, and analyses of the effect on the macro-invertebrates and chlorophyll-a populations[2]. When the department completes this analysis, stakeholder meetings will be held for briefings on the criteria development process. On April 13, 2018, MDNR issued an updated version of the Missouri Water Quality Standards to the EPA and was approved on December 14, 2018. The revision includes updates to

[2] Nutrient Criteria for Water Quality. (n.d.). Retrieved December 13, 2017, from https://dnr.mo.gov/env/wpp/wqstandards/wq_nutrient-criteria.htm

water quality criteria for pH and other pollutants, and changes to sections describing mixing zones, general criteria, and existing definitions.

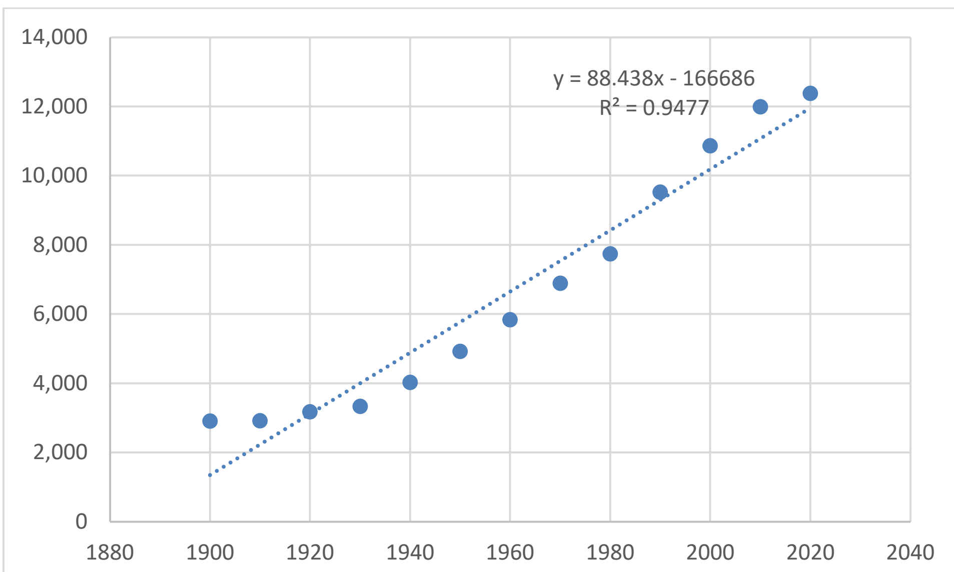
3.4 Treatment Plant Improvements Design Basis

Future projections of influent flow and loading were developed based on population growth models. This section describes the methodology used for these future projections.

3.4.1 Population Projections

Population projections were developed for the West Plains WWTP service area to assist in the development of future design flows and pollutant loadings. The 2020 population is projected at 12,382 people, and West Plains has been growing at a rate of approximately 1.5% since 1980. Historic population growth is shown in Figure 3-5. For the purpose of this evaluation, an annual growth rate of 1.5% was used for population projections.

Figure 3-5: West Plains Historic Population Growth



3.4.2 Forecasted Influent Flow

Flow rates entering the West Plains WWTP were forecasted through 2075 (Table 3-5) using the current per capital flow rate. The WWTP is projected to reach its design capacity of 3.0 MGD in 2075. Thus, the projected 2075 flow rates, with the exception of peak flow, will be used as the basis of design for improvements discussed herein. Hydraulic modeling was performed to determine the maximum flow that could be conveyed to the WWTP based on conveyance capacity of the collection system; refer to the Flow Analysis & Inflow and Infiltration Evaluation. Based on modeling results, a peak flow of 14 MGD will be used as the basis for the improvements discussed herein.

Table 3-5: Population and Flow Projections

	2020	2035	2040	2050	2075
Population	12,382	15,480	16,677	19,354	28,082
Flow (MGD)					
Average Day	1.34	1.68	1.80	2.09	3.00
Maximum Month	4.98	6.22	6.70	7.78	11.3
Maximum Day	6.07	7.58	8.17	9.48	13.8
Peak	7.00	8.75	9.43	10.9	15.9[1]

[1]Based on hydraulic modeling of the collection system, 14 MGD will be used as the peak flow design basis.

3.4.3 Forecasted Influent Loadings

The operating data for TSS and BOD had high maximum month and maximum day loadings because of the presence of outliers. The outliers could have resulted from a number of sources, including atypical slugs of solids/organic material, wet weather events, and errors in data collection or transcription. A frequency distribution plot was generated, and influent loading data was trimmed by removing any daily loading data points that exceeded the average of all loadings plus three standard deviations. Data trimming resulted in the omission of 4.7 percent of BOD and TSS loading data points.

The revised data sets were used to develop average day, maximum month, and maximum day loadings for TSS and BOD, using the methodology in Section 3.4.2. The peaking factors associated with maximum month and maximum day loadings are higher than typical domestic wastewater treatment facilities, as reported in WEF Manual of Practice No. 8, Vol. 1. This is generally attributed to solids deposition and accumulation in the collection system and the “first flush” effect that occurs during peak wet weather events. The updated TSS and BOD loadings are shown in Table 3-6.

Table 3-6: Current Influent Loading and Peaking Factors After Data Trimming

Parameter	Loading (lb/day)	Peaking Factor	Typical Peaking Factor
TSS			
Average Day	2,000	-	-
Maximum Month	4,700	2.35	1.36
Maximum Day	5,400	2.70	1.77
BOD			
Average Day	1,800	-	-
Maximum Month	2,800	1.56	1.35
Maximum Day	4,000	2.22	1.59

Limited total phosphorus data is available for the West Plains WWTP (ten data points from September 2020). The average concentration of the existing ammonia, TKN and TP data is approximately 13 mg/L, 46 mg/L, and 5.8 mg/L, respectively. Metcalf & Eddy (2014) lists ranges for typical per capita loading for ammonia, TKN, and TP. Per capita loadings were used for average day conditions that align with two-week sampling data provided by the City. A summary of this analysis is provided in Table 3-7. Table 3-8 provides projected influent loadings that were scaled up to 3.0 MGD; these loadings will serve as the basis of design for the purpose of this evaluation. Forecasted maximum month loadings used a 1.58 peaking factor, and maximum day loadings used a 2.28 peaking factor to align with the influent BOD peaking factors.

Table 3-7: Projected Ammonia, TKN, and TP Average Day Loadings

Parameter	Typical Contribution, lbs/capita-d	Sampling Data, lbs/capita-d	Proposed Value, lbs/capita-d	Projected Average Future Loading
Ammonia	0.011-0.26	0.012	0.02	560
TKN	0.02-0.048	0.043	0.035	980
TP	0.006-0.010	0.0044	0.005	140

Table 3-8: Influent Loading Design Basis

Parameter	Average Day (lb/day)	Maximum Month (lb/day)	Maximum Day (lb/day)
BOD	4,000	6,300	9,100
TSS	4,800	10,600	12,300
Ammonia	560	880	1,270
TKN	980	1,540	2,230
TP	140	220	320

4.0 BIOLOGICAL MODELING

BioWin is a wastewater treatment process simulator that models biological, chemical, and physical wastewater unit processes. It can be used to model WWTP upgrades and process optimization. The model is based on chemical oxygen demand (COD), nitrogen, and phosphorus mass balances. Models were created for the West Plains WWTP based on existing conditions and anticipated loadings at 3.0 MGD. The following sections describe model calibration and results at the projected design flow.

4.1 Model Calibration

A model was created of the existing secondary treatment process using historical data and targeted sampling data provided by the City. Average influent mass loadings were used as the model inputs, and the model was calibrated to average effluent concentrations for data from the two-week sampling period in September 2020. Calibration was achieved by adjusting kinetic and stoichiometric parameters, which affect growth and decay rates of microorganisms. The calibration model is shown in Figure 4-1, and results are included in Table 4-1.

Table 4-1: Calibration Results of the Biological Model

Parameter	September 2020 Special Sampling Average	Model Results
Temperature	No Data	20 degrees C
Flow	1.3 MGD	1.3 MGD
Influent BOD	130 mg/L	130 mg/L
Influent TSS	220 mg/L	220 mg/L
Influent Ammonia	14 mg/L	14 mg/L
Influent TN	46 mg/L	46 mg/L
Influent TP	5.0 mg/L	5.0 mg/L
Bioreactor DO Concentration	No Data	2.0 mg/L
Aerated SRT	No Data	17 days
Effluent BOD	2.0 mg/L	1.1 mg/L
Effluent TSS	2.1 mg/L	2.4 mg/L
Effluent Ammonia	0.11 mg/L	0.23 mg/L
Effluent TN	20 mg/L	19 mg/L
Effluent TP	1.4 mg/L	1.8 mg/L

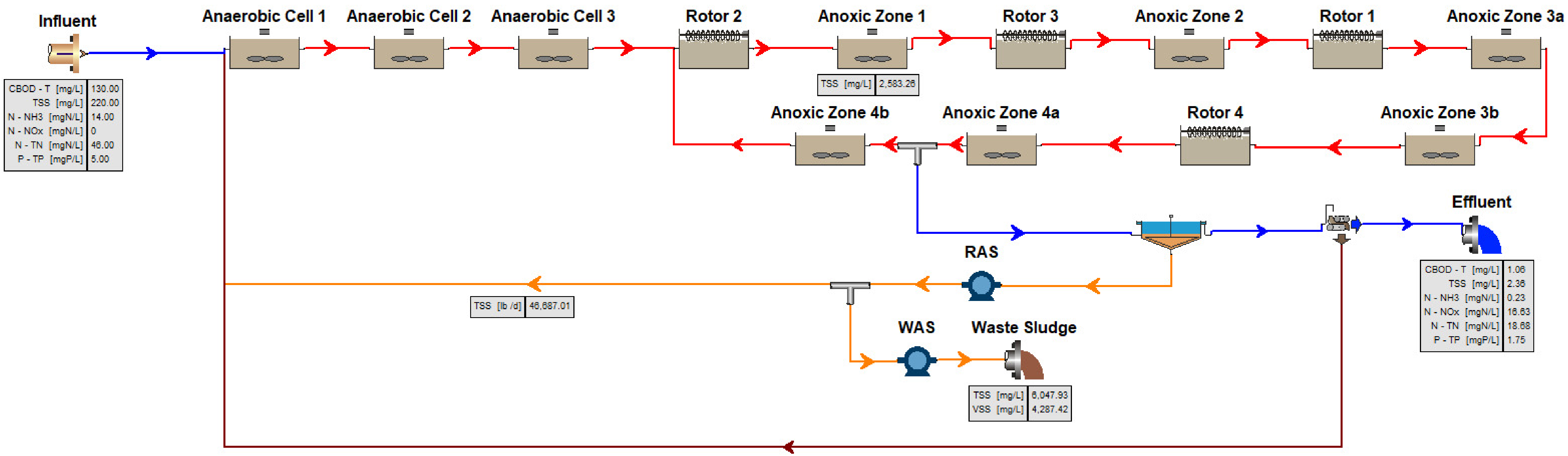


FIGURE 4-1

BIOWIN MODEL OF EXISTING OPERATIONS

4.2 Biological Modeling of 3.0 MGD Plant Capacity

Biological modeling was conducted at 3.0 MGD using design maximum month conditions to determine process volumes required to meet future discharge permit limits. Stoichiometric and kinetic parameters established during model calibration were used for modeling of design conditions.

The existing anaerobic capacity is sufficient to accomplish enhanced biological phosphorus removal (EBPR). A hydraulic retention time (HRT) of one to two hours is recommended for EBPR; the existing anaerobic volumes provides an HRT of 1.5 hours at an influent flow of 3.0 MGD. However, due to low-strength nature of the influent flow (low concentrations of influent BOD), a supplemental carbon feed system may be required to facilitate EBPR, as well as denitrification in downstream reactors. With adequate supplemental carbon, the existing anaerobic basins are anticipated to be adequate for achieving an effluent total phosphorus concentration of1 mg/L after effluent filtration.

The existing oxidation ditch volume is insufficient for nitrification at the projected influent total nitrogen loading at 3.0 MGD, and assuming a future effluent TN limit of 10 mg/L. An additional 1.4 MG of treatment volume would be required to achieve effluent ammonia requirements. A total of 3.7 MG of treatment volume would be sufficient to accommodate a flow of 3.0 MGD at the maximum month loadings (as specified in Section 3.4.3), when operated at an SRT of 14 days, an MLSS concentration of 3,600 mg/L, and a DO concentration of 1 mg/L in the aerated zones. The three 60-ft clarifiers are sufficient based on a peak surface overflow rate of 1,000 gpd/ft^2 and a peak solids loading rate of 35 lb/day-ft^2. Modeling results for are included in Table 4-2.

Table 4-2: Biological Modeling Results at a Design Flow of 3.0 MGD

Parameter	Maximum Month Concentration	Effluent Permit Limitation	Effluent Concentration
Temperature	12 degrees C	N/A	12 degrees C
Flow	3.0 MGD	N/A	3.0 MGD
BOD	240 mg/L	10 mg/L	2 mg/L
TSS	400 mg/L	15 mg/L	4 mg/L
Ammonia	35 mg/L	2.1 mg/L[a]	1.3 mg/L
TN	62 mg/L	N/A	10 mg/L
TP	9 mg/L	N/A	1 mg/L

(a) From Oct 1 through March 31st when influent temperature could be as low as 12 degrees C.

5.0 CAPITAL IMPROVEMENTS

The drivers for the improvements identified for the WWTP are based on regulations (both existing and anticipated) and performance-related issues, including plant hydraulics. The WWTP has historically been in compliance with effluent ammonia limits, other than during wet weather events. However, improvements to restore hydraulic capacity, provide sufficient aeration, and reduce sludge age are necessary to ensure future compliance. Further, regulations for biological nutrient removal are being planned for future implementation state-wide by MDNR; thus, provisions for future total nitrogen and total phosphorus removal should be considered.

The headworks, tertiary filtration, disinfection and solids handling systems also require improvements to better support the liquid stream process and operations. Further, the hydraulic bottleneck created by undersized piping needs to be corrected to allow the facility to operate effectively up to the design peak flow capacity. The following sections describe the recommended capital improvements at the WWTP.

5.1 Headworks

The existing WWTP has experienced hydraulic issues due to inadequate channel depth. Further, the significant portion of the influent is unscreened through the bypass pipe, which causes clogging, wearing of pumps, and accumulation of debris within treatment basins. The condition of the influent pump station is poor and requires rehabilitation to maintain adequate influent pumping capacity. Installation of a new screening facility and influent pump station is recommended; the following sections describe the recommended improvements.

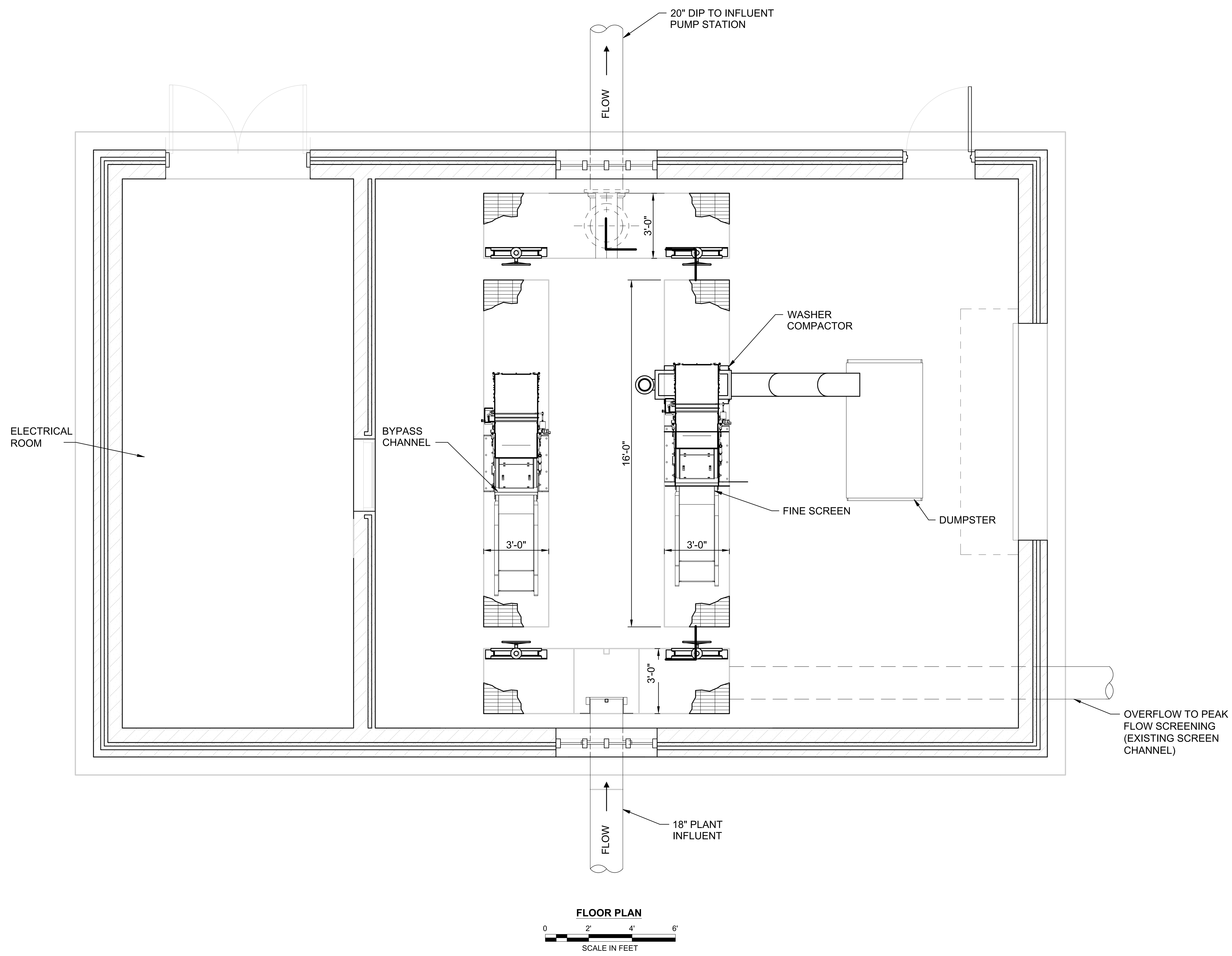
5.1.1 Screening

A commonly used screening technology for municipal wastewater plants is the multi-rake screen; the existing influent screen is a multi-rake style screen. Multi-rake screens consist of a bar rack and a series of rakes on a chain that collect and convey screenings from the channel to the operating floor (refer to Figure 5-1). Some multi-rake screens may require in-channel maintenance of the lower sprocket or guide that maintains alignment of the chain and rake mechanism. However, the existing Duperon screen does not have a lower sprocket, so all maintenance can be performed from the operating level.

Figure 5-1: Typical Multi-Rake Screen (Courtesy of Vulcan)

The existing headworks is a hydraulic bottleneck during peak events, and the bypass constructed to relieve hydraulic backups is unscreened. A new influent screening building would be constructed with deeper screen channels, set at the appropriate depth to prevent bottlenecking. The existing screen channel would be used as a bypass channel to screen influent routed to the peak flow pump station.

The new influent screening facility would comprise two channels, each designed for a peak hydraulic capacity of 7 MGD. One channel would be equipped with a multi-rake screen, and the second channel would have a manual bar rack. Quarter-inch spacing is recommended for the influent screens to protect downstream processes and facilitate land application by removing large debris and plastics. A washer/compactor would be installed to facilitate screenings dewatering and removal of organics prior to disposal. A building to house the new influent screening equipment is recommended to prevent freezing and protect the equipment from the elements. Screened influent would then flow by gravity to a new influent pump station, as described in Section 5.1.2. Refer to Figure 5-2 for a conceptual layout of the new screenings facility. Table 5-1 summarizes the design criteria for screen replacement.



BURNS McDONNELL

FIGURE 5-2

INFLUENT SCREENING
CONCEPTUAL LAYOUT

The existing screening channel would maintain its current hydraulic capacity and be repurposed as a peak flow screening channel. An overflow pipe would be constructed to connect the influent well of the proposed screening facility to the influent well of the existing screening channel. Flow in excess of 7 MGD would be diverted to this wet weather screening channel. The existing 24-inch main from the existing screening channel to the screw pump station would be plugged; the 24-inch main to the peak flow pump station would be maintained.

Table 5-1: Influent Screening Design Criteria

Number of Screens	2 (1 mechanical, 1 manual)
Number of Washer/Compactors	1
Design Peak Flow Capacity	7 MGD
Proposed Channel Depth	12.5 feet
Proposed Screen Depth	2.5 feet
Proposed Channel Width	3 feet
Screen Angle of Inclination	Approximately 80 degrees
Proposed Screen Bar Size	¼-inch

5.1.2 Influent Pumping

The existing influent pump station requires rehabilitation of both the equipment and structure to maintain operability. Screw pumps are capital-intensive and costly to repair relative to other pumping technologies. Further, the pump station structure requires significant concrete work to restore functionality to both pumps. Construction of a new submersible pump station is recommended in lieu of rehabilitating the existing pump station.

The pump station would be located downstream of the new headworks facility (southwest of the anaerobic selector) and would consist of a wet well containing three submersible pumps, and an adjacent valve vault. Each pump would be rated for 3.5 MGD, providing a firm pump station capacity of 7 MGD. A new influent force main would be routed to Cell 1 of the anaerobic selector. The screw pump station would be demolished. Refer to Figure 5-3 for the proposed layout of the influent pump station. Table 5-2 summarizes the design criteria for the proposed influent pump station.

Table 5-2: Design Criteria for Proposed Influent Pump Station

Number of Pumps	3 (2 duty / 1 standby)
Capacity Per Pump	3.5 MGD
Firm Capacity	7 MGD
Pump Type	Submersible

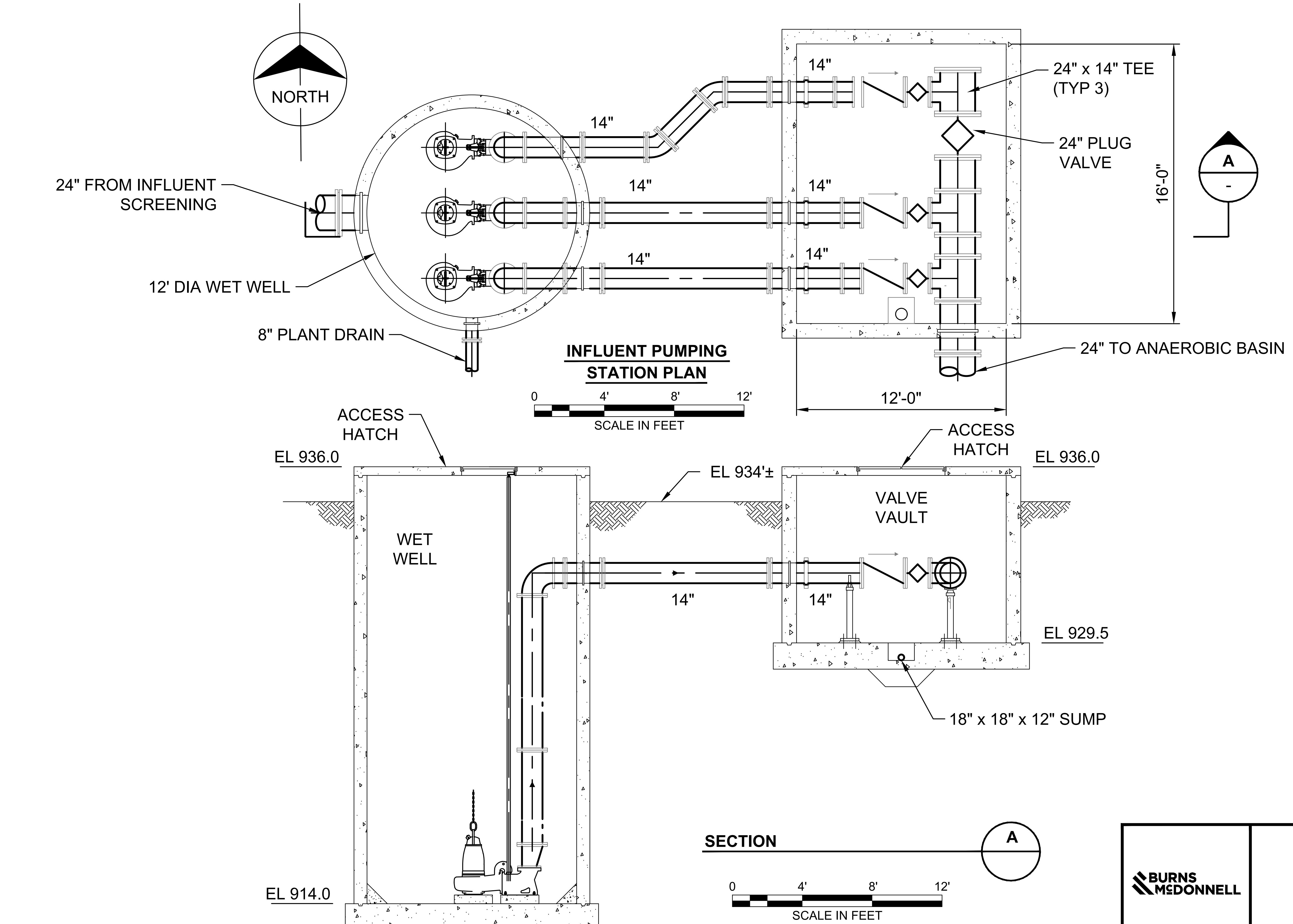


FIGURE 5-3

INFLUENT
PUMP STATION

BURNS McDONNELL

5.1.3 Grit Removal

The existing grit removal system consists of a plug flow channel that relies on settling to remove grit from the influent. In order to remove the grit that accumulates in the channels, the channel is drained, and a manual gate is raised in the side of the channel, and grit is manually loaded into a truck for hauling. The existing grit system is operator intensive and is not effective; thus, demolition is recommended. Constructing a new grit removal system is not recommended for this phase of improvements to allow the City to allocate capital to projects directly impacting compliance with effluent permit limits. However, provisions should be made for future installation.

5.2 Secondary Treatment

The secondary treatment process is anticipated to have insufficient volume and air supply for the design flow of 3.0 MGD. Additional process volume is required, as well as physical improvements to prevent future overtopping of the oxidation ditch. The following sections describe the recommended improvements to the secondary treatment process.

5.2.1 Anaerobic Basins

The selector basins maintain an anaerobic condition to condition for luxury uptake of phosphorus in the aerated process. The existing anaerobic basins consists of a single structure divided into three cells. Anaerobic selectors are typically designed to provide an SRT of one day and an HRT of approximately one to two hours. The current arrangement includes RAS feed to Cell 1, and influent wastewater feed to Cell 2, which allows only Cells 2 and 3 to contribute to the design SRT and HRT. In order to utilize the entire anaerobic basin volume, the new influent force main would be installed to discharge directly to Cell 1. Additionally, the side walls of the anaerobic basin must be raised to mitigate overtopping issues.

5.2.2 Oxidation Ditch

Oxidation ditch improvements would consist of rehabilitation of aeration equipment and basin bridges providing additional aeration capacity. The aeration basin is hydraulically overloaded due to downstream hydraulic bottlenecks and the basin’s effluent weir elevation. Further, the brush rotors are nearing the end of their useful life (typically 15 to 20 years), have limited oxygen transfer efficiency compared to other aeration technologies, and create a hazard for operations staff with the amount of splashing and airborne wastewater. Thus, retrofitting the oxidation ditch with a diffused aeration grid is recommend in lieu of surface aeration. Blowers would be required for air supply to the diffusers. Five blowers would be installed, each rated for 2,700 scfm, which would allow for implementation of cyclic aeration, as discussed below.

The WWTP currently has a single oxidation ditch, which complicates constructability of aeration improvements. Operation of the aeration process must be maintained through the duration of construction. In order to facilitate implementation of the aeration improvements, construction of an additional oxidation ditch is recommended. The new basin would add an additional 1.4 MG of aeration capacity. Once the new basin is constructed and commissioned, the existing aeration basin can be taken offline to install the diffused aeration system. It would also allow for cleaning of material that has deposited within the basin since it's commissioning in 1979. Operations staff speculate there is a significant amount of grit that has accumulated in the basin, which reduces available treatment volume.

Construction of additional treatment volume would also position the City for future nutrient removal requirements. The new basin would allow for implementation of a cyclic aeration (turning air on and off) treatment process to target total nitrogen removal. This process results in alternating aerobic/anoxic conditions and eliminates the need for separate treatment basins or dedicated internal recycle pumping, which increase cost and operational complexity. During the aerated cycle, ammonia is reduced to nitrite/nitrate. During the anoxic cycle, the nitrate is converted to nitrogen gas. Denitrification allows for lower oxygen demand in the process as well as the return of alkalinity to the wastewater. Cyclic aeration typically achieves BOD, ammonia, and TN removal. Extension of the off cycle will promote anaerobic conditions, which may provide a reduction of total phosphorus as well.

5.2.3 Final Clarification

The 10 States Standards and MDNR recommend final clarifiers be designed for a surface overflow rate of 1,000 gpd/ft^2 and a solids loading rate of 35 lb/day-ft^2 for an activated sludge process with nitrification at peak capacity. The existing clarifiers have sufficient capacity for the design loadings associated with peak flow. To promote the continued performance of the clarifiers, recoating of the existing mechanisms is recommended, as the mechanisms were installed in 2005.

5.2.4 Chemical Feed

The WWTP has relatively low influent BOD loading, likely attributable to inflow and infiltration (I/I) in the collection system. Both denitrification (conversion of nitrate to nitrogen gas) and phosphorus uptake require readily degradable BOD to be effective. Approximately six mols of carbon are required per mol of nitrogen, and approximately twenty mols of carbon per mol of phosphorus. Without sufficient influent BOD (carbon), the reactions for total nitrogen and total phosphorus removal may be hindered.

Biological phosphorus removal relies on biological reactions that occur in separate anaerobic and aerobic zones and requires fine-tuned operations to maintain the correct biology in the treatment system. During peak events, or if the plant receives a slug load, the system may experience a biological upset that hinders treatment performance. For this reason, a backup chemical feed system is recommended for chemical phosphorus removal. A chemical feed system will also position the City for compliance with future, more stringent total phosphorus effluent limits. Metal salts, such as ferric chloride or aluminum sulfate, are typically used for chemical phosphorus removal. Aluminum sulfate will be used as the basis for the recommendations described herein, as ferric chloride could adversely affect treatment performance of the UV disinfection system.

Supplemental carbon could be added in the form of methanol or MicroC™; addition of MicroC™ is recommended, as there are a number of safety hazards associated with storage and use of methanol. Both the carbon and alum feed systems would consist of a chemical storage tank, containment, metering pumps, piping, and accessories.

Typically, chemical feed systems are sized to store at least thirty days of chemicals on-site at the design loading. Chemical metering pumps are sized for the peak loading to the facility. Two peristaltic pumps would be provided for an n +1 configuration for redundancy for each feed system. The chemical feed skid would be housed in a new chemical feed building adjacent to the outdoor chemical storage tank containment area. The alum would be injected into the clarifier influent splitter to provide adequate contact time and mixing to promote coagulation prior to filtration. An injection quill would also be added to the discharge header of the intermediate pump station to provide a secondary feed point. Table 5-3 lists a summary of the design criteria for the chemical feed systems. This analysis was developed using alum as the coagulant. The final design should evaluate alternative coagulants from a treatment capability and cost assessment standpoint.

Table 5-3: Chemical Feed System Summary

Feed Rate – Average Day Loading	35 gpd (alum) 25 gpd (MicroC)
Feed Rate – Peak Day Loading	100 gpd (alum) 40 gpd (MicroC)
Days of Storage	30
Storage Volume Required (gallons)	3,000 gallons (each, alum & carbon)
Number of Tanks	2 (1 for each chemical)
Number of Pumps	1 duty/1 redundant for each chemical

5.2.5 Summary of Secondary Treatment Improvements

Table 5-4 summarizes the design criteria for Secondary Treatment improvements.

Table 5-4: Proposed Secondary Treatment Improvements

Selector Basin	
Recommended Improvement	Yard piping modifications; raise basin wall elevation
Aeration	
Flow	3.0 MGD
Additional Volume Needed	1.37 MG
Maximum Month MLSS	3,600 mg/L
Standard Oxygen Requirement	33,000 lb/day
Aeration Type	Diffused
Number of Blowers	4 (3 duty, 1 standby)
Blower Capacity, Each	2,400 scfm
Mixers Required	Existing Basin: 4 New Basin: 2
Final Clarification	
Recommended Improvement	Recoat mechanisms
Chemical Feed - Carbon	
Dosing Range	20-100 gpd
Days of Storage	30 days
Chemical Storage	3,000 gallons
Chemical Feed - Alum	
Dosing Range	35-100 gpd (alum) 25-40 gpd (carbon)
Days of Storage	30 days
Chemical Storage	3,000 gallons (alum) 3,000 gallons (carbon)

5.3 Intermediate Pump Station

Interceptor capacity to convey flow to the WWTP is approximately 14 MGD. The WWTP, with the improvements described herein, will have capacity to convey and treat a peak flow of 7 MGD from influent screening through secondary treatment. Flow in excess of this would be conveyed to the peak weather treatment system and would then combine with full treated wastewater upstream of filtration.

Peak flow, in a blending scenario, typically goes through primary treatment prior to being blended with fully treated effluent. In fact, the West Plains WWTP has piping in place to accommodate blending. However, the pipe from the peak flow clarifier to the filter splitter structure (the pipe that would facilitate blending) is undersized and does not allow gravity flow between the two structures.

With the proximity and elevation of the 100-year flood relative to UV disinfection and tertiary filtration, the WWTP effluent is unable to flow by gravity during periods when Howell Creek is high. The filtration and UV equipment needs to be set above the 100-year flood elevation to allow for gravity flow out of the plant. To facilitate this, an intermediate pump station would be required to pump flow from the final clarifiers to the filters. The effluent from the peak flow clarifier would also be routed to the pump station; the wet well would be designed to allow gravity flow from the peak flow clarifier. The elevation of the filters and UV equipment would be set above the flood elevation, which would allow for gravity discharge.

The pump station would be designed with a firm capacity of 14 MGD (9,722 gpm). It would be equipped with 5 submersible pumps, each rated for 3.5 MGD (2,430 gpm). Two pumps would meet the peak demand of the flow receiving full treatment, and all four pumps would be required during peak events. Refer to Figure 5-3 for the proposed layout of the influent pump station. Table 5-5 summarizes the design criteria for the proposed influent pump station.

Table 5-5: Design Criteria for Proposed Intermediate Pump Station

Number of Pumps	5 (4 duty / 1 standby)
Capacity Per Pump	3.5 MGD (2,430 gpm)
Firm Capacity	14 MGD (9,722 gpm)
Pump Type	Submersible

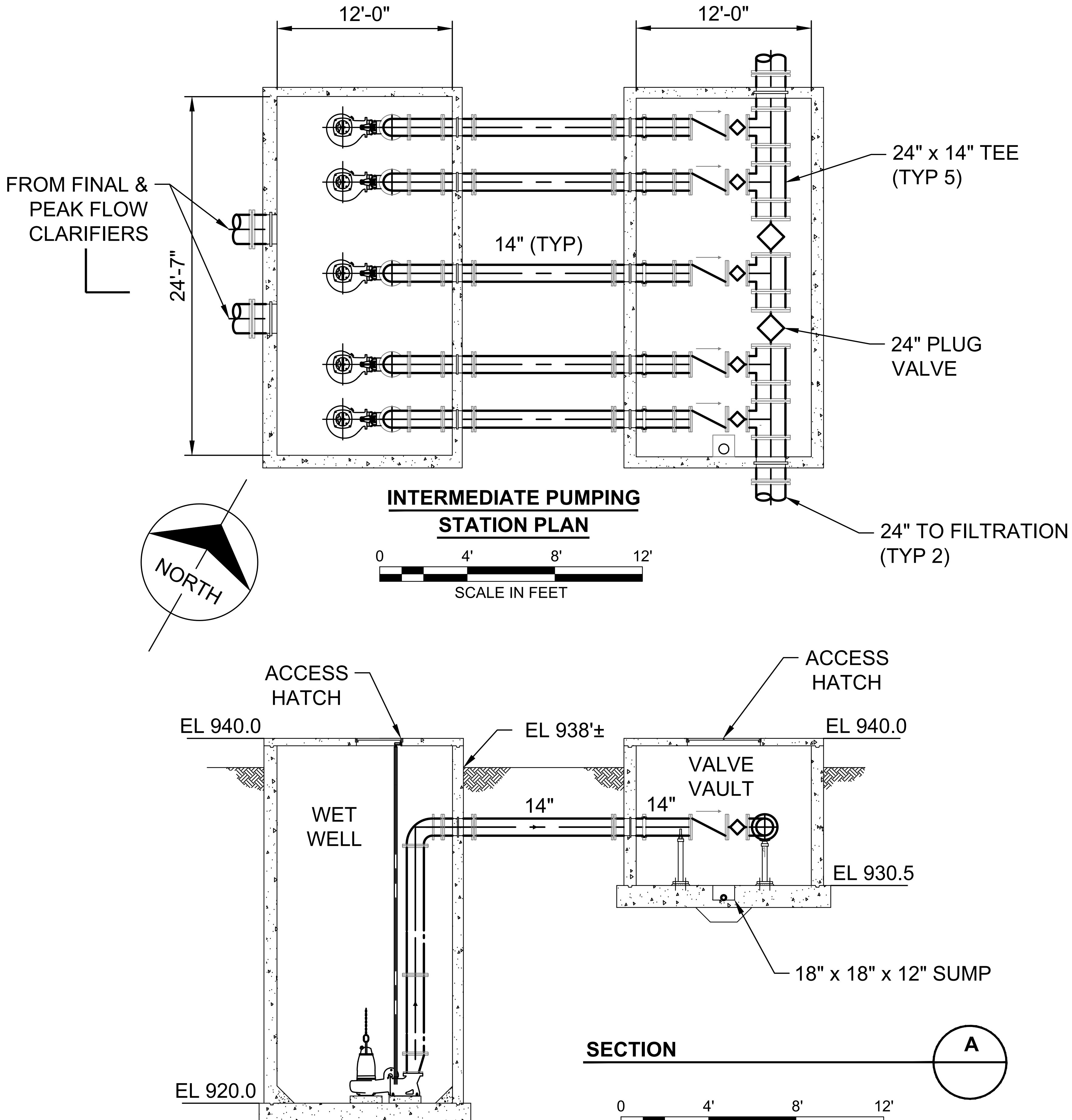


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FIGURE 5-4

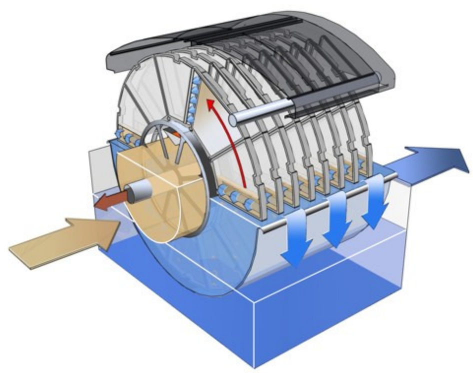
INTERMEDIATE PUMP STATION CONCEPTUAL LAYOUT

5.4 Tertiary Filtration

The existing traveling bridge sand filters are inefficient and lack the performance of cloth media filtration. Replacement of the traveling bridge sand filters with disc filters is recommended to maintain TSS and total phosphorus removal.

Within a disc filter, each disc is typically comprised of multiple filter modules mounted around a rotating drum. The water flow pattern of the filter is an "inside-out" design, that is, water to be cleaned enters the open end of the hollow rotating drum and exits out through the submerged portion of the filter disc. Solids in the water are collected on the inside surface of each filter panel and are washed off by counter-current spray water as the filter panels pass over a collection trough mounted inside the rotating drum. The filter media is a cloth woven from non-metallic polymer fibers such that 10 to 15-micron openings are created. The filter media is often pleated in the plastic frame, which allows more surface area for filtration. Refer to Figure 5-5 for a pictorial representation of a typical disc filter.

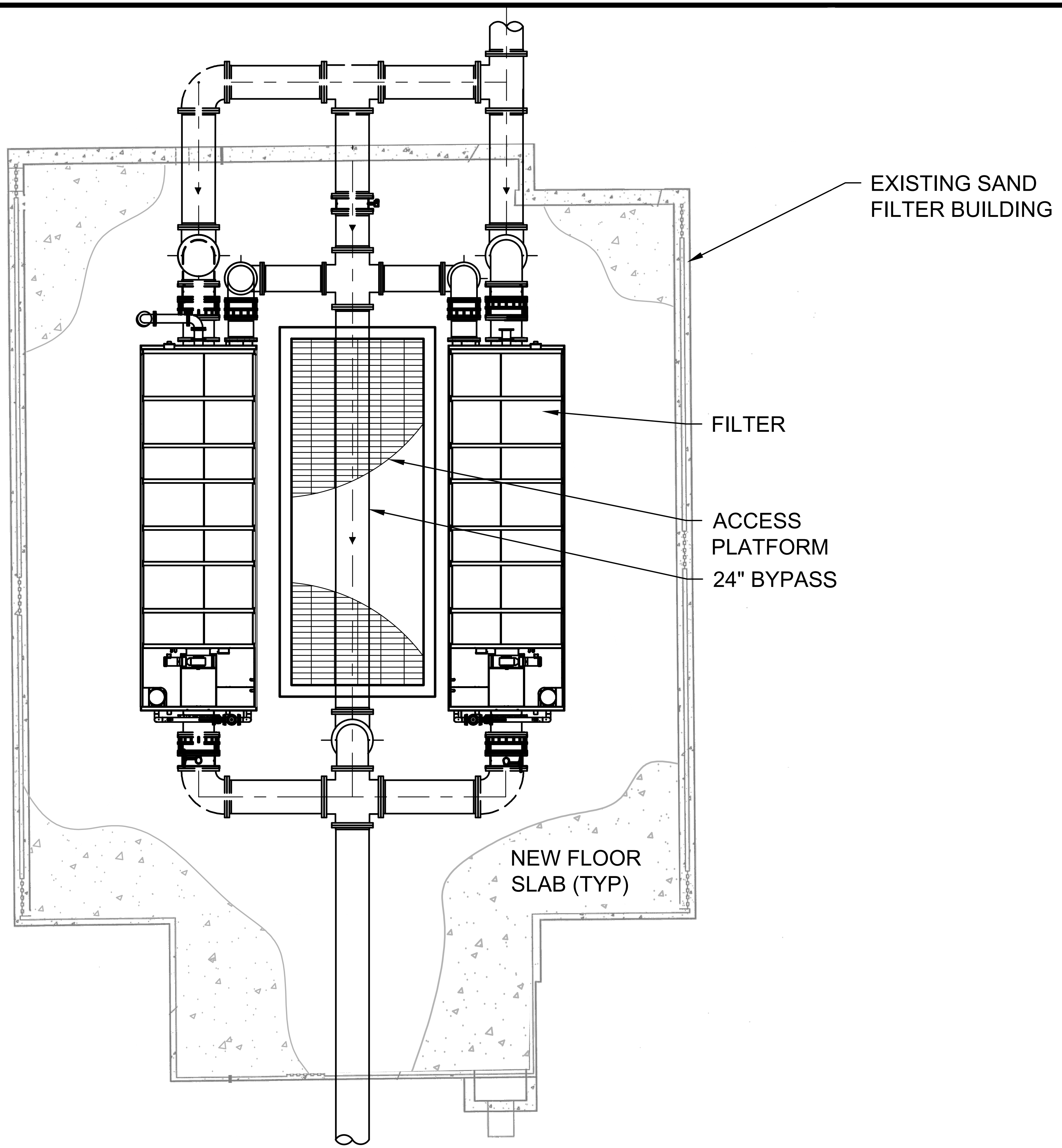
Figure 5-5: Disc Filter Schematic (Courtesy of Nordic)



With the construction of an intermediate pump station, the disc filters could be installed within the existing filter channels with limited structural modifications required. Two disc-filters, each rated for 7 MGD, would be installed in filter channels 1 and 2 and would receive clarifier effluent and peak flow from the peak flow clarifier (refer to Figure 5-6). One disc-filter would be used during normal operation, and both would be operated during peak conditions. Table 5-6 summarizes the design criteria for filter replacement.

Table 5-6: Proposed Tertiary Filtration Design Criteria

Number of Filters	2
Capacity Per Filter	7 MGD
Total Capacity	14 MGD
Peak Hydraulic Loading Rate	5 gpm/ft^2
Average Effluent TSS	10 mg/L



PLAN
NOT TO SCALE

FIGURE - 5-6

PROPOSED FILTER
IMPROVEMENTS

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5.5 Disinfection

The existing UV disinfection system was installed in 2005 and is nearing the end of its useful design life. Further, the process creates a hydraulic bottleneck during wet weather events, as it was designed for a peak flow of 3.5 MGD. Operations staff indicated that the UV structure has flooded during large peak events, as it is located within the 100-year floodplain. The UV system should be sized to accommodate the peak influent flow of 14 MGD. The UV system would disinfect the 7 MGD treated by the plant and the 7 MGD peak wet weather flow conveyed from the peak flow clarifier. In order to reduce capital cost and make use of existing infrastructure, new, closed-vessel equipment could be installed in filter channel 3.

Closed vessel UV systems (Figure 5-7) were created to reduce the potential for short-circuiting and improve treatment efficacy. This configuration eliminates dead space and provides constant contact between the wastewater and UV light, which reduces the overall number of components (lamps, ballasts, cabinets, etc.). Further, the closed vessels are equipped with medium-pressure bulbs to reduce the number of lamps and ballasts required. The enclosed configuration improves on the potency of UV lamps without sacrificing efficiency or lamp size.

Figure 5-7: Closed Vessel UV Disinfection System

The UV system will deliver a minimum UV dosage of 30 mJ/cm^2 at peak flow. In order to disinfect 14 MGD and achieve 126 CFU/100 mL, approximately two vessels would be required (one duty, one standby); refer to Figure 5-8. Isolation of each UV module would be accomplished with butterfly valves located at the influent and effluent line of each module. The influent valves would be controlled manually, and the effluent valves would be automated to bring units in and out of service based on plant

Figure 5-8: UV Disinfection Conceptual Layout

flow. Refer to Table 5-7 for design criteria of the UV improvements.

Table 5-7: UV Disinfection Design Criteria for Proposed Improvements

Number of Vessels	2
Peak Capacity	14 MGD
Anticipated UV Transmittance	65%
System Type	Closed Vessel
Bulb Type	Medium Pressure
Design Removal	126 colonies/100 mL

5.6 Solids Handling

The solids handling process has insufficient capacity to meet Class B biosolids requirements for the WWTP's rated capacity of 3.0 MGD, and the plant lacks the ability to dewater solids prior to hauling. Thus, construction of additional digestion capacity and a dewatering process are recommended, as described herein. The sludge quantities developed for this evaluation were based on influent BOD, TSS and temperature data provided by operations staff. The design wasting rate to maintain a 10-to-15-day SRT in the secondary treatment process is 116,000 gpd at 0.8% solids, which equates to approximately 8,000 lb/day. This solids loading will be used as the design basis for the solids processing system.

5.6.1 Aerobic Digestion

WAS and scum from the final clarifiers is currently pumped directly to the aerobic digestion system, which consists of six aerobic digesters operated in parallel. The process has historically produced biosolids that meet the requirements for Class B biosolids. The six existing digesters do not provide sufficient capacity for pathogen destruction at the design flow (3.0 MGD) and loadings, discussed in Section 3.4. The SRT required by the EPA for Class B biosolids requirements is 60 days at a minimum temperature of 15 degrees Celsius. A 60-day SRT requires approximately 2.0 million gallons (MG) of digester capacity, assuming a 2% solids concentration is maintained across the digesters. The six existing digesters provide approximately 550,000 gallons of digestion volume; thus, an additional 1.45 MG of digestion volume is required to meet future design solids loadings. Table 5-8 summarizes the design criteria for aerobic digestion improvements.

Rather than maintaining the existing digester volume and constructing an additional 1.45 MG of digester capacity, construction of 2.0 MG of digester capacity is recommended to consolidate equipment and maintenance activities. The existing digester tankage would be repurposed as sludge holding and supernatant storage. WAS would continue to be pumped to the existing tankage, where it would be stored prior to digestion. WAS would then flow by gravity from sludge holding to the newly constructed digesters, where a solids concentration between 2.0% and 2.5% would be maintained.

Two aerobic digesters would be constructed, 100-ft in diameter and a side water depth of 18 feet. The digesters would be equipped with telescoping valves to decant and thicken the digester contents, and the supernatant would be routed to two of the sludge tanks for storage. The supernatant storage would allow operations staff to gradually dose the supernatant back to the head of the plant, which would prevent slug loads of nitrogen and phosphorus from the solids process.

The digesters would be equipped with medium or fine bubble diffusers to facilitate volatile solids destruction and mixing within the tanks. Blowers would be required for air supply to the diffusers. Five blowers would be installed, each rated for 2,000 scfm. Refer to Table 5-8 for a summary of improvements recommended for the digestion process.

Table 5-8: Design Criteria for Aerobic Digestion Improvements

Digesters	
Additional Digesters	2
Diameter	100 ft
Side Water Depth	18 ft
SRT	60 days
Minimum Temperature	15 degrees C
Solids Concentration	2.0%
Aeration Equipment	
Aeration Type	Diffused Aeration (Fine or Medium Bubble)
Number of Blowers	5 (4 duty / 1 standby)
Blower Type	Positive Displacement
Air Supply, each	2,000 scfm
Anticipated Motor Size	125 hp

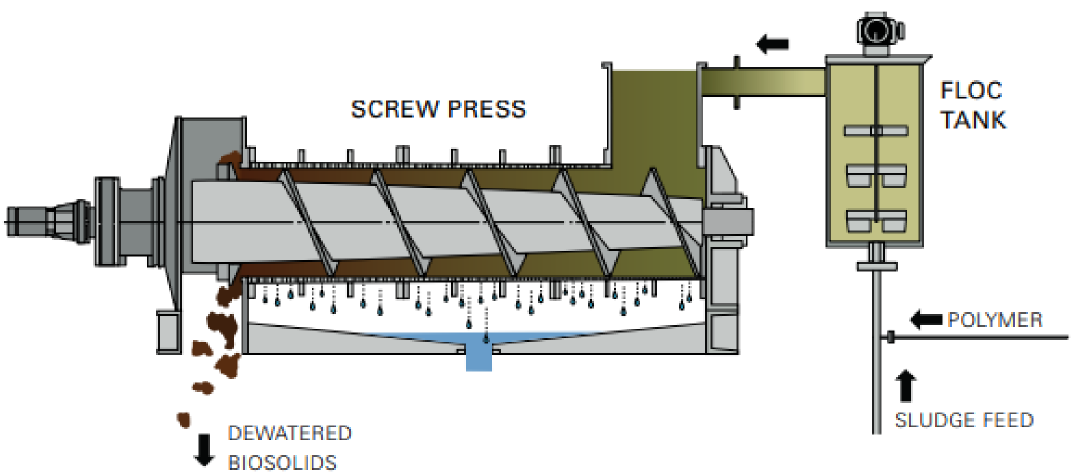
5.6.2 Solids Dewatering

Solids dewatering reduces the water content in sludge and is typically used to reduce the volume of solids required to be hauled offsite for disposal or beneficial reuse. A typical dewatering process receives a thin sludge (0.5-3% typically for municipal applications) and produces a dewatered cake and a supernatant. For activated sludge only applications, dewatered cake is typically produced with 15-25% total solids content, depending on the technology used. The cake is hauled off site for disposal or land application, and the supernatant is typically returned to the head of the WWTP.

Dewatering technologies that are commonly used in the municipal wastewater industry include belt filter presses, centrifuges, and screw presses. A belt filter press relies on gravity drainage and compression for solids separation. The biosolids are sandwiched between two porous belts that pass over and under rollers of differing diameters. The belt filter press is the most established dewatering technology, but the technology is somewhat dated and has largely been replaced by the centrifuge and screw press, in recent years.

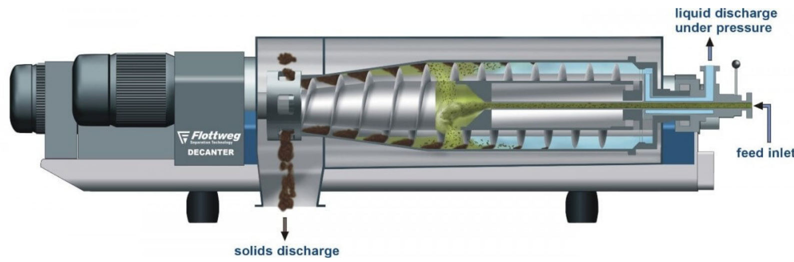
A screw press (Figure 5-9) is similar to a belt filter press in that it relies on gravity drainage and compression for solids separation. Solids are conveyed through the basket screen along an auger whose shaft diameter progressively increases. The volume available for solids decreases as it travels up the auger, which increases the applied pressure and friction. Screw presses are less prominent in the wastewater industry but are gaining popularity due to their ease of operation and low power consumption. Screw presses have a low rotational speed (less than one rpm), which allows for a relatively small motor (approximately 5 hp).

Figure 5-9: Screw Press Dewatering Schematic (Courtesy of FKC)



Centrifuge dewatering (Figure 5-10) relies on high-speed rotation to separate water and solids. Centrifuges contain a conical-shaped bowl that rotates at a pre-set speed. The feed rotates with the bowl, and the solids within the feed are forced against the bowl wall by the centrifugal force generated by the unit's rotation. An internal scroll within the bowl pushes the separated solids toward the smaller end of the bowl for disposal. Centrifuges have several advantages over other dewatering technologies. They are the smallest technology in terms of footprint but still produce a drier cake. When operations are stable, the centrifuge requires minimal operational oversight. However, the mechanics of a centrifuge are complex and require repairs by the manufacturers should an issue arise. Centrifuge dewatering requires higher power-consumption than belt filter presses and screw presses.

Figure 5-10: Centrifuge Dewatering Schematic (Courtesy of Flottweg)



During design, further evaluation of the appropriate dewatering technology is warranted. For the purpose of this memorandum, dewatering improvements consider use of a screw press. Two 570-gpm progressive cavity or rotary lobe pumps would send stabilized solids from the aerobic digesters to the dewatering process. Two screw presses would be required, each capable of processing 700 lb/hr and would dewater digested sludge from approximately 2% solids to approximately 16% solids. The screw press would be sized to handle 0.5% solids as well in order to accommodate a process upset. Filtrate from the dewatering process would be conveyed to the plant headworks. Dewatered cake would be conveyed via a shaftless screw conveyor to a sludge storage pad. The storage pad would be covered and sized for three months of storage. Table 5-9 summarizes the design criteria for sludge dewatering.

Table 5-9: Design Criteria for Sludge Dewatering

Feed Rate at 0.5% Solids	570 gpm
Feed Rate at 2% Solids	150 gpm
Loading	1,400 lb/hr
Influent Solids Concentration	0.5%-2.0%
Discharge Solids Concentration	16%-18%
Days Storage	90 days

The screw press, rotary lobe pumps, and polymer feed system (for sludge conditioning upstream of the screw press) would be located in a new building that will provide sufficient space to house a screw press or centrifuge and associated chemical feed equipment. The building would include an adjacent exterior area with overhead cover for cake storage and loadout. The proposed location for the new dewatering building is north of the anaerobic selectors; this would provide easy truck access for sludge hauling. Refer to Figure 5-14 for a site plan showing the proposed location of solids dewatering.

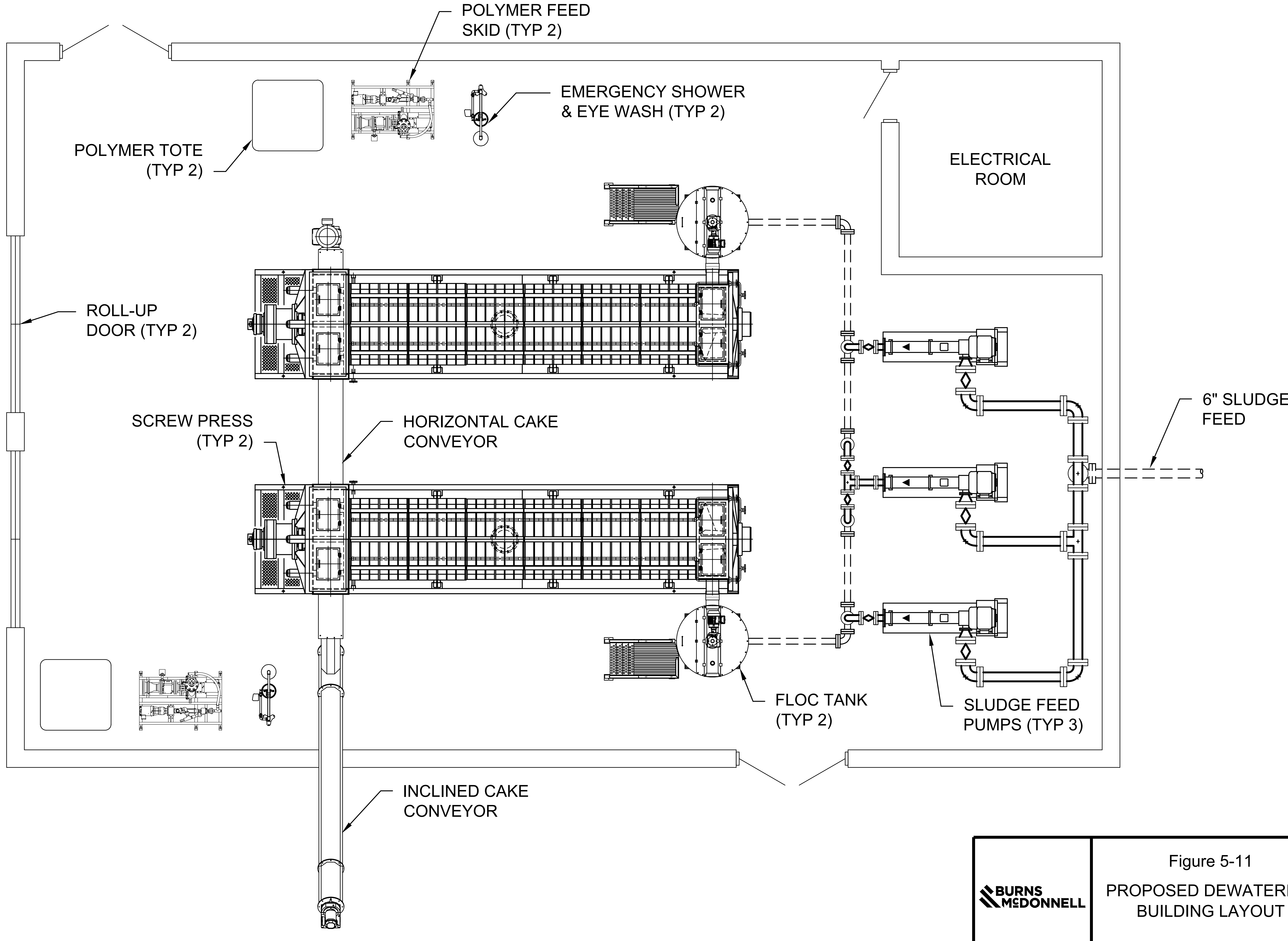


Figure 5-11

PROPOSED DEWATERING BUILDING LAYOUT

5.7 Project Phasing

Several immediate improvements are required to address hydraulic bottlenecks and to accommodate a peak flow of 14 MGD. However, there is an opportunity to phase improvements as capital becomes available. Blending and secondary treatment improvements need to be constructed in the near term, as they directly impact permit compliance. Thus, influent screening, influent pumping, secondary treatment, tertiary filtration, and UV disinfection improvements should be included in the first phase. The solids handling system has sufficient capacity for current flow and loading and is not an immediate need. Similarly, grit removal is not essential to meeting effluent permit requirements. Therefore, aerobic digestion, solids dewatering, and grit removal improvements could be constructed in a future Phase 2.

5.8 Opinion of Probable Cost

A summary of recommended improvements and process capacities are provided in Figure 5-12 and Figure 5-13 for the liquid and solids streams, respectively. Figure 5-14 provides a proposed site plan for Phase I and Phase II improvements. Based on these improvements, Burns & McDonnell developed opinions of probable construction costs. The cost opinions show the capital required for each project and the opportunity for phasing.

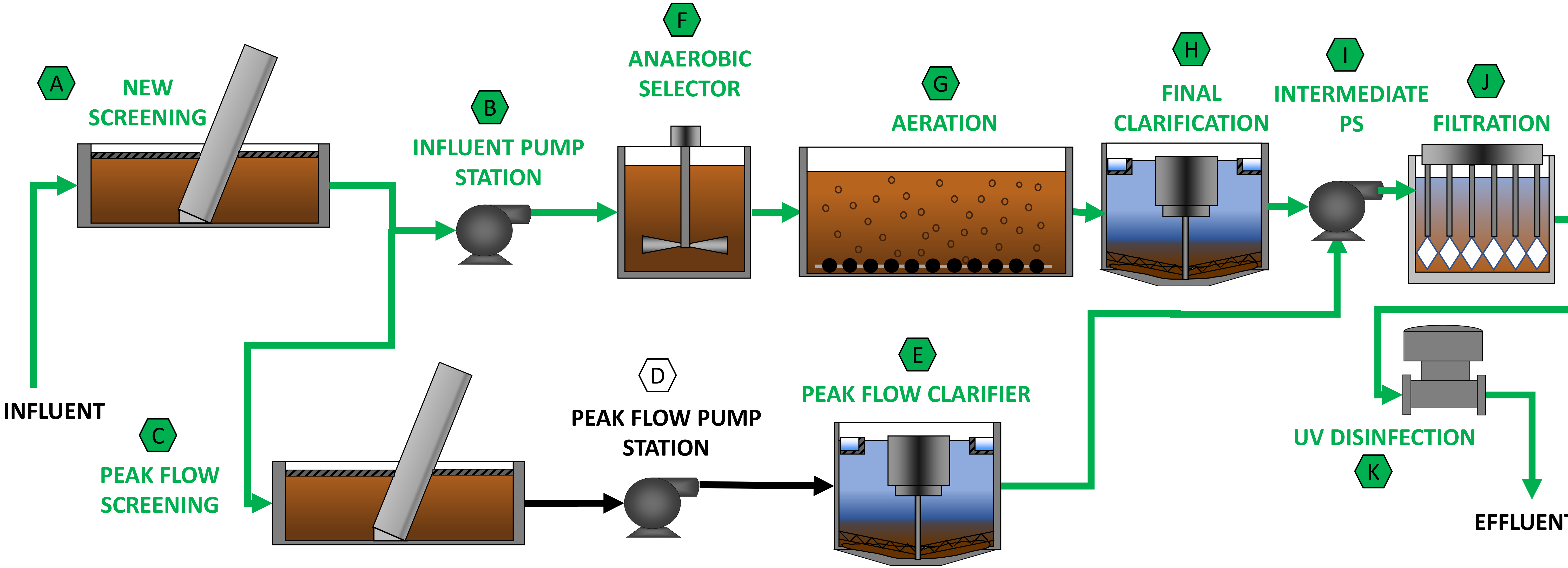
These order-of-magnitude cost opinions are based primarily on our experience and judgment as a professional consultant combined with information from past experience, vendors, and published sources. Since Burns & McDonnell has no control over weather, cost, availability of labor, availability of material and equipment, labor productivity, construction contractor's procedures and methods, unavoidable delays, construction contractor's methods of determining prices, economic conditions, government regulations and laws (including the interpretation thereof), competitive bidding or market conditions, and other factors affecting such opinions or projections, Burns & McDonnell does not guarantee the actual rates, costs, etc. will not vary from the opinions and projections developed herein.

A 30-percent contingency is included to cover all types of unaccounted-for project costs resulting from conditions, details, or components which are not normally known or determined until final detailed design. Costs specifically do not include geotechnical evaluations, deep foundations, surveys, permitting preparation and fees, utility services to site, and taxes.

Table 5-10: WWTP Opinions of Probable Cost

Project[1]	Capital Cost
Phase I	
Influent Screening	$2,800,000
Influent Pumping	$2,000,000
Secondary Treatment	$10,800,000
Intermediate Pump Station	$2,800,000
Tertiary Filtration	$5,400,000
Disinfection	$3,800,000
Phase I Total	**$27,600,000**
Phase II	
Grit Removal	$2,500,000
Aerobic Digestion	$8,400,000
Sludge Dewatering	$4,200,000
Phase II Total	**$15,100,000**

[1]Piping changes described in Section 2.7 are included within each project.
[2]Refer to Appendix B for preliminary estimates of operations and maintenance costs.



	A	B	C	D	E	F	G	H	I*	J*	K*
Recommended Improvement	Construct new headworks building to screen influent flow	Construct new submersible pump station	Use existing screen channel for peak flow screening	N/A	Coat mechanism	Adjust weir heights	Construct additional tankage; replace surface aerators with diffused aeration	Coat mechanisms	Construct intermediate pump station to help plant hydraulics	Retrofit filters 1 & 2 with cloth media filters	Install closed vessel UV system in filter 3 building
Future Capacity	7.0 MGD mechanical screen; 7.0 MGD manual screen	7.0 MGD (firm) 10.5 MGD (total)	10 MGD	7.0 MGD (firm) 10.5 MGD (total)	7.0 MGD	3.0 MGD (ADF) 7.0 MGD (Peak)	Train 1: 2.5 MGD Train 2: 1.0 MGD Total: 3.5 MGD	2.3 MGD (per Clarifier) 7.0 MGD (total)	14 MGD (firm) 17.5 MGD (total)	7 MGD (per channel) 14 MGD (total)	14 MGD
	• Total influent flow to plant assumed to be 14 MGD until modeling results are available										

*GREEN PIPING AND EQUIPMENT LABELING INDICATES NEW

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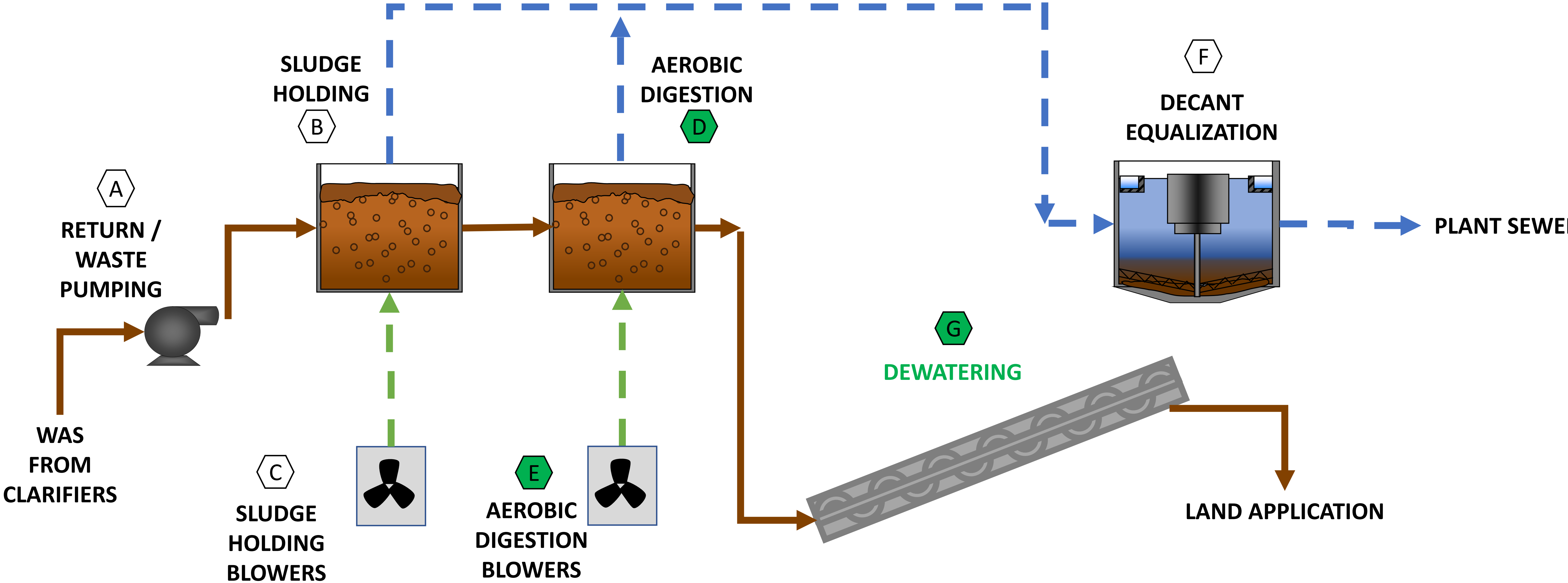
FIGURE 5-12
LIQUID STREAM IMPROVEMENTS DIAGRAM



	A	B	C	D	E	F	G
Existing Capacity	4.5 MGD 2.25 MGD per pump	550,000 gallons (used as aerobic digesters)	850 scfm total 425 scfm per blower	N/A	N/A	N/A	N/A
Recommended Improvement	None	Convert four of six existing digesters to sludge holding	None	Construct two new aerobic digesters that provide sufficient capacity to achieve Class B biosolids at 3.0 MGD	Provide sufficient aeration capacity to provide sufficient mixing to two new digesters	Convert two of six existing digesters to supernatant storage	Construct dewatering facility to reduce water content of sludge prior to hauling
Future Capacity	4.5 MGD 2.25 MGD per pump	367,000 gallons	850 scfm total 425 scfm per blower	100-ft diameter; 18 ft depth 2 MG total	8,000 scfm total 2,000 scfm per blower	183,000 gallons	1,400 lb/hr 140 gpm @ 2.0%

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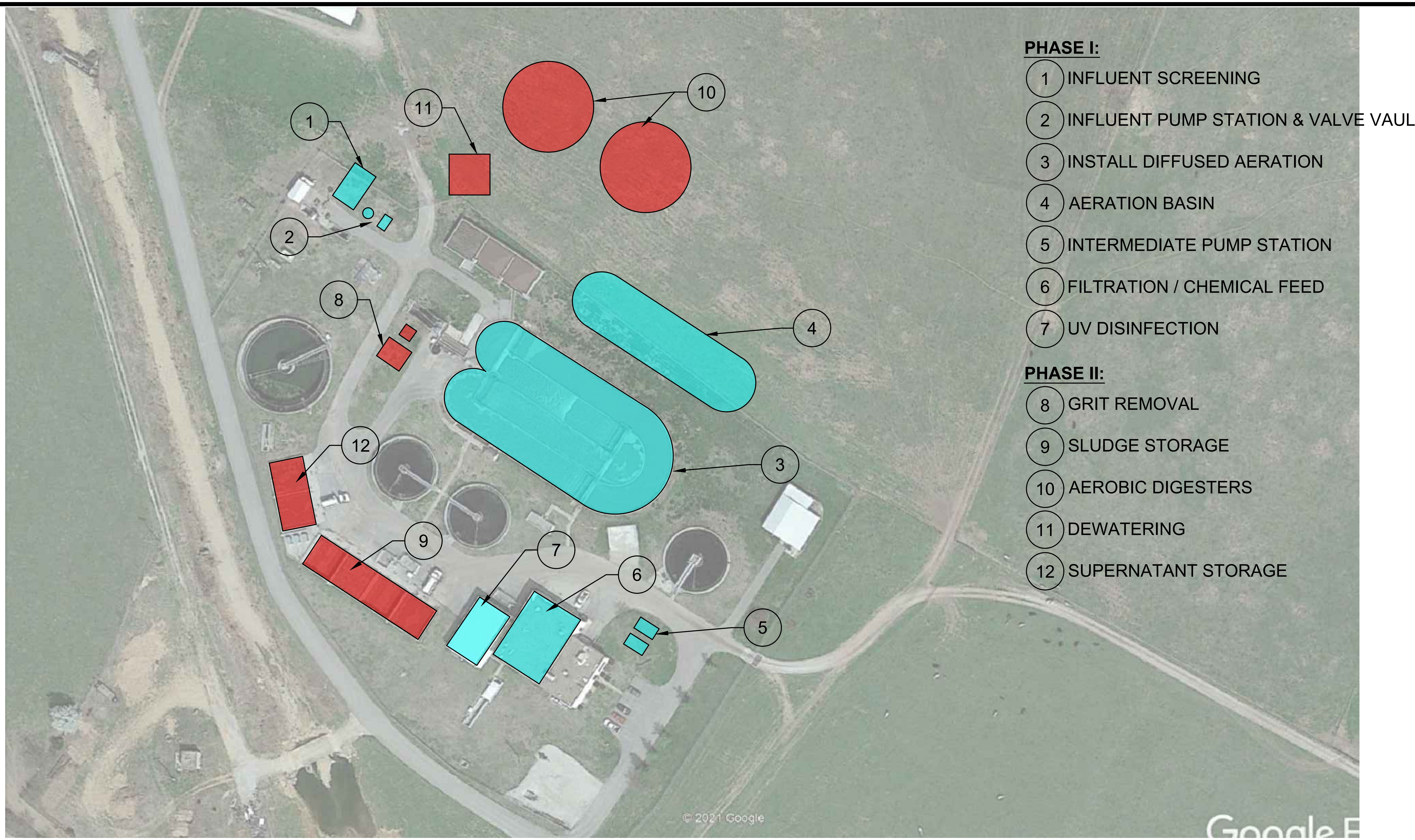
FIGURE 5-13
SOLID STREAM IMPROVEMENTS SUMMARY



SITE PLAN
NOT TO SCALE

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BURNS McDONNELL	FIGURE 5-14 PROPOSED SITE PLAN FOR PHASE I AND PHASE II IMPROVEMENTS

APPENDIX A - WEST PLAINS WWTP NPDES PERMIT

STATE OF MISSOURI

DEPARTMENT OF NATURAL RESOURCES

MISSOURI CLEAN WATER COMMISSION

MISSOURI STATE OPERATING PERMIT

In compliance with the Missouri Clean Water Law, (Chapter 644 R.S. Mo. as amended, hereinafter, the Law), and the Federal Water Pollution Control Act (Public Law 92-500, 92nd Congress) as amended,

Permit No.	MO-0096610
Owner:	City of West Plains
Address:	P.O. Box 710, West Plains, MO 65775
Continuing Authority:	Same as above
Address:	Same as above
Facility Name:	West Plains Wastewater Treatment Facility
Facility Address:	0.34 mi S of 1508 County Road 8240, West Plains, MO 65775
Legal Description:	Sec. 27, T24N, R8W, Howell County
UTM Coordinates:	X = 604780, Y = 4064666
Receiving Stream:	Howell Creek (C) (losing)
First Classified Stream and ID:	Howell Creek (C) (2582) (losing)
USGS Basin & Sub-watershed No.:	(11010010-0201)

is authorized to discharge from the facility described herein, in accordance with the effluent limitations and monitoring requirements as set forth herein:

FACILITY DESCRIPTION

Outfall #001 – POTW – SIC #4952
The use or operation of this facility shall be by or under the supervision of a Certified **A** Operator.
Bar screen / grit removal / influent screw pump / flow measurement / nutrient removal / oxidation ditch / clarifiers (4) / sand filter / UV disinfection / aerated sludge holding tanks (6) / sludge is land applied / facility does not have materials stored or conduct operations in a manner that would cause the discharge of pollutants via stormwater
Design population equivalent is 15,600.
Design flow is 3.0 MGD.
Actual flow is 1.8 MGD.
Design sludge production is 327.6 dry tons/year.

This permit authorizes only wastewater under the Missouri Clean Water Law and the National Pollutant Discharge Elimination System; it does not apply to other regulated areas. This permit may be appealed in accordance with Section 621.250 RSMo, Section 640.013 RSMo and Section 644.051.6 of the Law.

October 1, 2018
Effective Date

Edward B. Galbraith, Director, Division of Environmental Quality

March 31, 2023
Expiration Date

Chris Wieberg, Director, Water Protection Program

OUTFALL #001	TABLE A. FINAL EFFLUENT LIMITATIONS AND MONITORING REQUIREMENTS

The permittee is authorized to discharge from outfall(s) with serial number(s) as specified in the application for this permit. The final effluent limitations shall become effective on **October 1, 2018** and remain in effect until expiration of the permit. Such discharges shall be controlled, limited and monitored by the permittee as specified below:

EFFLUENT PARAMETER(S)	UNITS	FINAL EFFLUENT LIMITATIONS			MONITORING REQUIREMENTS	
		DAILY MAXIMUM	WEEKLY AVERAGE	MONTHLY AVERAGE	MEASUREMENT FREQUENCY	SAMPLE TYPE
Flow	MGD	*		*	once/weekday***	24 hr. total
Biochemical Oxygen Demand$_5$	mg/L		15	10	once/week	composite**
Total Suspended Solids	mg/L		20	15	once/week	composite**
E. coli (Note 1, Page 3)	#/100mL	126		*	once/week	grab
Ammonia as N (Apr 1 – Sep 30) (Oct 1 – Mar 31)	mg/L	 5.9 10.9		 1.2 2.1	once/week	grab
Oil & Grease	mg/L	15		10	once/month	grab
MONITORING REPORTS SHALL BE SUBMITTED MONTHLY; THE FIRST REPORT IS DUE NOVEMBER 28, 2018. THERE SHALL BE NO DISCHARGE OF FLOATING SOLIDS OR VISIBLE FOAM IN OTHER THAN TRACE AMOUNTS.						
Bis (2-ethylhexyl) phthalate	µg/L	*		*	once/quarter *****	grab
Cyanide, Amenable to Chlorination	µg/L	*		*	once/quarter *****	grab
Total Phosphorus	mg/L	*		*	once/quarter *****	grab
Total Nitrogen	mg/L	*		*	once/quarter *****	grab
MONITORING REPORTS SHALL BE SUBMITTED QUARTERLY; THE FIRST REPORT IS DUE JANUARY 28, 2019.						
EFFLUENT PARAMETER(S)	UNITS	MINIMUM		MAXIMUM	MEASUREMENT FREQUENCY	SAMPLE TYPE
pH – Units****	SU	6.5		9.0	once/week	grab
MONITORING REPORTS SHALL BE SUBMITTED MONTHLY; THE FIRST REPORT IS DUE NOVEMBER 28, 2018.						

EFFLUENT PARAMETER(S)	UNITS	MONTHLY AVERAGE MINIMUM	MEASUREMENT FREQUENCY	SAMPLE TYPE
Biochemical Oxygen Demand$_5$ – Percent Removal (Note 2, Page 3)	%	85	once/week	calculated
Total Suspended Solids – Percent Removal (Note 2, Page 3)	%	85	once/week	calculated
MONITORING REPORTS SHALL BE SUBMITTED MONTHLY; THE FIRST REPORT IS DUE NOVEMBER 28, 2018.				

* Monitoring requirement only.

** A 24-hour composite sample is composed of 48 aliquots (subsamples) collected at 30 minute intervals by an automatic sampling device.

*** Once each weekday means: Monday, Tuesday, Wednesday, Thursday, and Friday.

**** pH is measured in pH units. pH is to be reported as a single instantaneous value or a consecutive 4-day average. At no time may an instantaneous pH value deviate from the technology based effluent limit range of 6.0-9.0 SU per 10 CSR 20-7.015.

***** See table on Page 3 for quarterly sampling requirements.

Quarterly Minimum Sampling Requirements			
Quarter	**Months**	**Total Phosphorous and Total Nitrogen**	**Report is Due**
First	January, February, March	Sample at least once during any month of the quarter	April 28th
Second	April, May, June	Sample at least once during any month of the quarter	July 28th
Third	July, August, September	Sample at least once during any month of the quarter	October 28th
Fourth	October, November, December	Sample at least once during any month of the quarter	January 28th

Note 1 – Effluent limits of 126 #/100 mL daily maximum and monitoring only for monthly average for *E. coli* are applicable year round due to losing stream designation. No more than 10% of samples over the course of a calendar year shall exceed the 126 #/100 mL daily maximum.

Note 2 – Influent sampling is not required when the facility does not discharge effluent during the reporting period. Samples are to be collected prior to any treatment process. Percent Removal is calculated by the following formula: [(Average Influent –Average Effluent) / Average Influent] x 100% = Percent Removal. Influent and effluent samples are to be taken during the same month. The Average Influent and Average Effluent values are to be calculated by adding the respective values together and dividing by the number of samples taken during the month. Influent samples are to be collected as a 24-hour composite sample, composed of 48 aliquots (subsamples) collected at 30 minute intervals by an automatic sampling device.

OUTFALL #00X	**TABLE A-2. WHOLE EFFLUENT TOXICITY FINAL EFFLUENT LIMITATIONS AND MONITORING REQUIREMENTS**					
The permittee is authorized to discharge from outfall(s) with serial number(s) as specified in the application for this permit. The final effluent limitations shall become effective on **October 1, 2018** and remain in effect until expiration of the permit. Such discharges shall be controlled, limited and monitored by the permittee as specified below:						
EFFLUENT PARAMETER(S)	UNITS	**FINAL EFFLUENT LIMITATIONS**			MONITORING REQUIREMENTS	
		DAILY MAXIMUM	WEEKLY AVERAGE	MONTHLY AVERAGE	MEASUREMENT FREQUENCY	SAMPLE TYPE
Acute Whole Effluent Toxicity (Note 3)	TU_a	*			once/year	composite**
MONITORING REPORTS SHALL BE SUBMITTED ANNUALLY; THE FIRST REPORT IS DUE JANUARY 28, 2019.						
Chronic Whole Effluent Toxicity (Note 4)	TU_c	*			once/permit cycle	composite**
WET TEST REPORTS SHALL BE SUBMITTED ONCE PER PERMIT CYCLE; THE FIRST REPORT IS DUE JANUARY 28, 2023.						

* Monitoring requirement only.
** A 24-hour composite sample is composed of 48 aliquots (subsamples) collected at 30 minute intervals by an automatic sampling device.

Note 3 – The Acute WET test shall be conducted once per year during the 1st, 2nd, 3rd, and 5th year of the permit cycle. See Special Condition #19 for additional requirements.

Note 4 –The Chronic WET test shall be conducted during the 4th year of the permit cycle. See Special Condition #20 for additional requirements.

B. STANDARD CONDITIONS

In addition to specified conditions stated herein, this permit is subject to the attached Parts I, II, & III standard conditions dated August 1, 2014, May 1, 2013, and March 1, 2015, and hereby incorporated as though fully set forth herein.

C. SPECIAL CONDITIONS

1. Electronic Discharge Monitoring Report (eDMR) Submission System.
   (a) Discharge Monitoring Reporting Requirements. The permittee must electronically submit compliance monitoring data via the eDMR system. In regards to Standard Conditions Part I, Section B, #7, the eDMR system is currently the only Department approved reporting method for this permit.
   (b) Programmatic Reporting Requirements. The following reports (if required by this permit) must be electronically submitted as an attachment to the eDMR system until such a time when the current or a new system is available to allow direct input of the data:
      (1) Collection System Maintenance Annual Reports;
      (2) Sludge/Biosolids Annual Reports;
         i. In addition to the annual Sludge/Biosolids report submitted to the Department, the permittee must submit Sludge/Biosolids Annual Reports electronically using EPA's NPDES Electronic Reporting Tool ("NeT") (https://cdx.epa.gov/).
      (3) Significant Industrial Users Compliance Reports (in municipalities without approved pretreatment programs); and
      (4) Any additional report required by the permit excluding bypass reporting.

      After such a system has been made available by the Department, required data shall be directly input into the system by the next report due date.
   (c) Other actions. The following shall be submitted electronically after such a system has been made available by the Department:
      (1) Notices of Termination (NOTs);
      (2) No Exposure Certifications (NOEs); and
      (3) Bypass reporting, See Special Condition #10 for 24-hr. bypass reporting requirements.
   (d) Electronic Submissions. To access the eDMR system, use the following link in your web browser: https://edmr.dnr.mo.gov/edmr/E2/Shared/Pages/Main/Login.aspx.
   (e) Waivers from Electronic Reporting. The permittee must electronically submit compliance monitoring data and reports unless a waiver is granted by the Department in compliance with 40 CFR Part 127. The permittee may obtain an electronic reporting waiver by first submitting an eDMR Waiver Request Form: http://dnr.mo.gov/forms/780-2692-f.pdf. The Department will either approve or deny this electronic reporting waiver request within 120 calendar days. Only permittees with an approved waiver request may submit monitoring data and reports on paper to the Department for the period that the approved electronic reporting waiver is effective.

2. The full implementation of this operating permit, which includes implementation of any applicable schedules of compliance, shall constitute compliance with all applicable federal and state statutes and regulations in accordance with §644.051.16, RSMo, and the Clean Water Act (CWA) section 402(k); however, this permit may be reopened and modified, or alternatively revoked and reissued:
   (a) To comply with any applicable effluent standard or limitation issued or approved under Sections 301(b)(2)(C) and (D), 304(b)(2), and 307(a)(2) of the CWA, if the effluent standard or limitation so issued or approved:
      (1) contains different conditions or is otherwise more stringent than any effluent limitation in the permit; or
      (2) controls any pollutant not limited in the permit.
   (b) To incorporate an approved pretreatment program pursuant to 40 CFR 403.8(a).

3. All outfalls must be clearly marked in the field.

4. Report as no-discharge when a discharge does not occur during the report period.

5. Changes in existing pollutants or the addition of new pollutants to the treatment facility

   The permittee must provide adequate notice to the Director of the following:
   (a) Any new introduction of pollutants into the POTW from an indirect discharger which would be subject to section 301 or 306 of CWA if it were directly discharging those pollutants; and
   (b) Any substantial change in the volume or character of pollutants being introduced into that POTW by a source introducing pollutants into the POTW at the time of issuance of the permit.
   (c) For purposes of this paragraph, adequate notice shall include information on;
      (1) the quality and quantity of effluent introduced into the POTW, and
      (2) any anticipated impact of the change on the quantity or quality of effluent to be discharged from the POTW.

C. SPECIAL CONDITIONS (continued)

6. Reporting of Non-Detects:
   (a) An analysis conducted by the permittee or their contracted laboratory shall be conducted in such a way that the precision and accuracy of the analyzed result can be enumerated.
   (b) The permittee shall not report a sample result as "Non-Detect" without also reporting the detection limit of the test. Reporting as "Non Detect" without also including the detection limit will be considered failure to report, which is a violation of this permit.
   (c) The permittee shall provide the "Non-Detect" sample result using the less than sign and the minimum detection limit (e.g. <10).
   (d) Where the permit contains a Minimum Level (ML) and the permittee is granted authority in the permit to report zero in lieu of the < ML for a specified parameter (conventional, priority pollutants, metals, etc.), then zero (0) is to be reported for that parameter.
   (e) See Standard Conditions Part I, Section A, #4 regarding proper detection limits used for sample analysis.
   (f) When calculating monthly averages, one-half of the method detection limit (MDL) should be used instead of a zero. Where all data are below the MDL, the "<MDL" shall be reported as indicated in item (c).

7. It is a violation of the Missouri Clean Water Law to fail to pay fees associated with this permit (644.055 RSMo).

8. The permittee shall comply with any applicable requirements listed in 10 CSR 20-9, unless the facility has received written notification that the Department has approved a modification to the requirements. The monitoring frequencies contained in this permit shall not be construed by the permittee as a modification of the monitoring frequencies listed in 10 CSR 20-9. To request a modification of the operational control testing requirements listed in 10 CSR 20-9, the permittee shall submit a permit modification application and fee to the Department requesting a deviation from the operational control monitoring requirements. If the request is approved, the Department will modify the permit.

9. The permittee shall develop and implement a program for maintenance and repair of the collection system. The recommended guidance is the US EPA's Guide for Evaluating Capacity, Management, Operation, And Maintenance (CMOM) Programs at Sanitary Sewer Collection Systems (Document number EPA 305-B-05-002) or the Departments' CMOM Model located at http://dnr.mo.gov/env/wpp/permits/docs/cmom-template.doc. For additional information regarding the Departments' CMOM Model, see the CMOM Plan Model Guidance document at http://dnr.mo.gov/pubs/pub2574.htm.

   The permittee shall also submit a report via the Electronic Discharge Monitoring Report (eDMR) Submission System annually, by January 28th, for the previous calendar year. The report shall contain the following information:
   (a) A summary of the efforts to locate and eliminate sources of excessive infiltration and inflow into the collection system serving the facility for the previous year.
   (b) A summary of the general maintenance and repairs to the collection system serving the facility for the previous year.
   (c) A summary of any planned maintenance and repairs to the collection system serving the facility for the upcoming calendar year. This list shall include locations (GPS, 911 address, manhole number, etc.) and actions to be taken.

10. Bypasses are not authorized at this facility unless they meet the criteria in 40 CFR 122.41(m). If a bypass occurs, the permittee shall report in accordance to 40 CFR 122.41(m)(3), and with Standard Condition Part I, Section B, subsection 2. Bypasses are to be reported to the Southeast Regional Office during normal business hours or by using the online Sanitary Sewer Overflow/Facility Bypass Application located at: http://dnr.mo.gov/modnrcag/ or the Environmental Emergency Response spill-line at 573-634-2436 outside of normal business hours. Once an electronic reporting system compliant with 40 CFR Part 127, the National Pollutant Discharge Elimination System (NPDES) Electronic Reporting Rule, is available all bypasses must be reported electronically via the new system. Blending, which is the practice of combining a partially-treated wastewater process stream with a fully-treated wastewater process stream prior to discharge, is not considered a form of bypass. If the permittee wishes to utilize blending, the permittee shall file an application to modify this permit to facilitate the inclusion of appropriate monitoring conditions.

11. The facility must be sufficiently secured to restrict entry by children, livestock and unauthorized persons as well as to protect the facility from vandalism.

12. At least one gate must be provided to access the wastewater treatment facility and provide for maintenance and mowing. The gate shall remain closed except when temporarily opened by the permittee to access the facility to perform operational monitoring, sampling, maintenance, or mowing. The gates shall also be temporarily opened for inspections by the Department. The gate shall be closed and locked when the facility is not staffed.

13. An all-weather access road shall be provided to the treatment facility.

C. SPECIAL CONDITIONS (continued)

14. At least one (1) warning sign shall be placed on each side of the facility enclosure in such positions as to be clearly visible from all directions of approach. There shall also be one (1) sign placed for every five hundred feet (500') (150 m) of the perimeter fence. A sign shall also be placed on each gate. Minimum wording shall be SEWAGE TREATMENT FACILITY—KEEP OUT. Signs shall be made of durable materials with characters at least two inches (2") high and shall be securely fastened to the fence, equipment or other suitable locations.

15. An Operation and Maintenance (O & M) manual shall be maintained by the permittee and made available to the operator. The O & M manual shall include key operating procedures and a brief summary of the operation of the facility.

16. The discharge from the wastewater treatment facility shall be conveyed to the receiving stream via a closed pipe or a paved or rip-rapped open channel. Sheet or meandering drainage is not acceptable. The outfall sewer shall be protected against the effects of floodwater, ice or other hazards as to reasonably insure its structural stability and freedom from stoppage. The outfall shall be maintained so that a sample of the effluent can be obtained at a point after the final treatment process and before the discharge mixes with the receiving waters.

17. The media in the filter beds shall be properly maintained to prevent surface pooling, vegetative growth, and accumulation of leaf litter.

18. Expanded Effluent Testing: Permittee must sample and analyze for the pollutants listed in 40 CFR 122.21 Appendix J, Table 2 along with Aluminum and Iron. Pursuant to 40 CFR 122.21(j)(4) the permittee shall provide this data with the permit renewal application from a minimum of three samples taken within four and one-half years prior to the date of the permit application. Samples must be representative of the seasonal variation in the discharge from each outfall. Approved and sufficiently sensitive testing methods listed in 40 CFR 136.3 must be utilized to detect pollutant concentrations below the Water Quality Criteria established in 10 CSR 20-7.031.

19. Acute Whole Effluent Toxicity (WET) tests shall be conducted as follows:
    (a) Freshwater Species and Test Methods: Species and short-term test methods for estimating the acute toxicity of NPDES effluents are found in the most recent edition of *Methods for Measuring the Acute Toxicity of Effluents and Receiving Waters to Freshwater and Marine Organisms* (EPA/821/R-02/012; Table IA, 40 CFR Part 136). The permittee shall concurrently conduct 48-hour, static, non-renewal toxicity tests with the following species:
    - The fathead minnow, *Pimephales promelas* (Acute Toxicity EPA Test Method 2000.0).
    - The daphnid, *Ceriodaphnia dubia* (Acute Toxicity EPA Test Method 2002.0).

    (b) Chemical and physical analysis of the upstream control sample and effluent sample shall occur immediately upon being received by the laboratory, prior to any manipulation of the effluent sample beyond preservation methods consistent with federal guidelines for WET testing that are required to stabilize the sample during shipping. Where upstream receiving water is not available or known to be toxic, other approved control water may be used.
    (c) Test conditions must meet all test acceptability criteria required by the EPA Method used in the analysis.
    (d) The Allowable Effluent Concentration (AEC) for this facility is 100% with the dilution series being: 100%, 50%, 25%, 12.5%, and 6.25%.
    (e) All chemical and physical analysis of the effluent sample performed in conjunction with the WET test shall be performed at the 100% effluent concentration.
    (f) The facility must submit a full laboratory report for all toxicity testing. The report must include a quantification of acute toxic units ($TU_a = 100/LC_{50}$) reported according to the test methods manual chapter on report preparation and test review. The Lethal Concentration 50 Percent (LC_{50}) is the effluent concentration that would cause death in 50 percent of the test organisms at a specific time.

C. SPECIAL CONDITIONS (continued)

20. Chronic Whole Effluent Toxicity (WET) tests shall be conducted as follows:
    (g) Freshwater Species and Test Methods: Species and short-term test methods for estimating the chronic toxicity of NPDES effluents are found in the most recent edition of *Short-term Methods for Estimating the Chronic Toxicity of Effluents and Receiving Waters to Freshwater Organisms (EPA/821/R-02/013; Table IA, 40 CFR Part 136).* The permittee shall concurrently conduct 7-day, static, renewal toxicity tests with the following species:
    - The fathead minnow, *Pimephales promelas* (Survival and Growth Test Method 1000.0).
    - The daphnid, *Ceriodaphnia dubia* (Survival and Reproduction Test Method 1002.0).

    (h) Chemical and physical analysis of the upstream control sample and effluent sample shall occur immediately upon being received by the laboratory, prior to any manipulation of the effluent sample beyond preservation methods consistent with federal guidelines for WET testing that are required to stabilize the sample during shipping. Where upstream receiving water is not available or known to be toxic, other approved control water may be used.
    (i) Test conditions must meet all test acceptability criteria required by the EPA Method used in the analysis.
    (j) The Allowable Effluent Concentration (AEC) is 100%, the dilution series is: 100%, 50%, 25%, 12.5%, and 6.25%.
    (k) All chemical and physical analysis of the effluent sample performed in conjunction with the WET test shall be performed at the 100% effluent concentration.
    (l) The facility must submit a full laboratory report for all toxicity testing. The report must include a quantification of chronic toxic units ($TU_c = 100/IC_{25}$) reported according to the *Methods for Measuring the Chronic Toxicity of Effluents and Receiving Waters to Freshwater and Marine Organisms* chapter on report preparation and test review. The 25 percent Inhibition Effect Concentration (IC_{25}) is the toxic or effluent concentration that would cause 25 percent reduction in mean young per female or in growth for the test populations.

APPENDIX B – PRELIMINARY O&M COSTS

Client	West Plains, MO
Project Number	126220
Description	O&M Cost - Plant Improvements

O&M Analysis Basis	3.5	MGD
Planning Period Start	2022	
Power Cost	0.07	$/kW-h

Equipment Replacement	Total # of Units	Capacity, each	Replacement Frequency (Years)	Replacement Unit Cost	Total Replacement Cost	Annual Replacement Cost
Influent Screen (with W/C)	1	7 MGD	20	$235,000	$235,000	$11,750
Peak Flow Screen (with W/C)	1	10 MGD	20	$250,000	$250,000	$12,500
Influent Pumps	3	3.5 MGD	20	$42,000	$126,000	$6,300
Peak Flow Pumps	3	3.5 MGD	20	$42,000	$126,000	$6,300
Peak Flow Clarifier Mechanism	1	96 ft	20	$385,000	$385,000	$19,250
Anaerobic Mixers	3	2.7 hp	15	$15,000	$45,000	$3,000
Aeration Blowers	5	150 hp	20	$90,000	$450,000	$22,500
Final Clarifier Mechanisms	3	60 ft	20	$210,000	$630,000	$31,500
Intermediate PS Pumps	5	3.5 MGD	20	$42,000	$210,000	$10,500
Disc Filters	2	7 MGD	20	$490,000	$980,000	$49,000
Chemical Tanks	2	3,000 gal	15	$15,000	$30,000	$2,000
Chemical Metering Skids	2	N/A	15	$20,000	$40,000	$2,667
UV Disinfection	2	7 MGD	20	$365,000	$730,000	$36,500
RAS Pumping	3	2.25 MGD	20	$37,500	$112,500	$5,625
Digester Blowers	3	50 hp	20	$40,000	$120,000	$6,000
Total Annual Equipment Replacement Cost						$ 225,392

Operating Power	Quantity	Unit hp	Operating Units at Basis of Analysis	Annual Operating Hours	Total kW-hr	Annual Operating Cost
Influent Screen	1	0.5	1	8760	3,266	$ 229
Washer/Compactor (Influent)	1	5	1	8760	32,662	$ 2,286
Peak Flow Screen	1	0.5	1	0	0	$ -
Washer/Compactor (PF)	1	5	1	0	0	$ -
Influent Pumps	3	40	1	8760	261,293	$ 18,291
Peak Flow Pumps	3	40	2	0	0	$ -
Peak Flow Clarifier Mechanism	1	0.75	1	0	0	$ -
Anaerobic Mixers	3	2.7	3	8760	52,912	$ 3,704
Aeration Blowers	5	150	4	8760	3,919,399	$ 274,358
Final Clarifier Mechanisms	3	0.5	3	8760	9,798	$ 686
Intermediate PS Pumps	5	40	1	8760	261,293	$ 18,291
Filter Backwash Pumps	2	25	1	5840	108,872	$ 7,621
Disc Filters	2	2	1	8760	13,065	$ 915
UV Disinfection (in kW)	2	32	1	8760	280,320	$ 19,622
RAS Pumping	3	14.8	2	8760	193,357	$ 13,535
Digester Blowers	3	50	2	8760	653,233	$ 45,726
Total Power Consumption Cost						$ 405,263

Chemical Consumption	Chemical Requirement at Basis of Analysis (gpd)	Operating Days/year	Chemical Usage (gal/year)	Chemical Unit Cost ($/Gal)	Extended Cost
Alum	35	365	12,775	$ 3.30	$ 42,158
Carbon	25	365	9,125	$ 3.00	$ 27,375
Total Chemical Cost					$ 69,533

Total	**$ 700,187**

Client	West Plains, MO
Project Number	126220
Description	O&M Cost - Plant Improvements

O&M Analysis Basis	3.5	MGD
Planning Period Start	2022	
Power Cost	0.07	$/kW-h

Equipment Replacement	Total # of Units	Capacity, each	Replacement Frequency (Years)	Replacement Unit Cost	Total Replacement Cost	Annual Replacement Cost
Influent Screen (with W/C)	1	7 MGD	20	$235,000	$235,000	$11,750
Peak Flow Screen (with W/C)	1	10 MGD	20	$250,000	$250,000	$12,500
Influent Pumps	3	3.5 MGD	20	$42,000	$126,000	$6,300
Peak Flow Pumps	3	3.5 MGD	20	$42,000	$126,000	$6,300
Peak Flow Clarifier Mechanism	1	96 ft	20	$385,000	$385,000	$19,250
Stacked Tray Grit	1	7 MGD	20	$175,000	$175,000	$8,750
Grit Pump	1	5 hp	15	$35,000	$35,000	$2,333
Grit Classifier	1	150 gpm	20	$165,000	$165,000	$8,250
Anaerobic Mixers	3	2.7 hp	15	$15,000	$45,000	$3,000
Aeration Blowers	5	150 hp	20	$90,000	$450,000	$22,500
Final Clarifier Mechanisms	3	60 ft	20	$210,000	$630,000	$31,500
Intermediate PS Pumps	5	3.5 MGD	20	$42,000	$210,000	$10,500
Disc Filters	2	7 MGD	20	$490,000	$980,000	$49,000
Chemical Tanks	2	3,000 gal	15	$15,000	$30,000	$2,000
Chemical Metering Skids	2	N/A	15	$20,000	$40,000	$2,667
UV Disinfection	2	7 MGD	20	$365,000	$730,000	$36,500
RAS Pumping	3	2.25 MGD	20	$37,500	$112,500	$5,625
Digester Blowers	5	125 hp	20	$40,000	$200,000	$10,000
Dewatering Pumps	2	570 gpm	20	$15,000	$30,000	$1,500
Polymer Feed	2	N/A	20	$30,000	$60,000	$3,000
Screw Press	2	700 lb/hr	20	$350,000	$700,000	$35,000
Total Annual Equipment Replacement Cost						$ 288,225

Operating Power	Quantity	Unit hp	Operating Units at Basis of Analysis	Annual Operating Hours	Total kW-hr	Annual Operating Cost
Influent Screen	1	0.5	1	8760	3,266	$ 229
Washer/Compactor (Influent)	1	5	1	8760	32,662	$ 2,286
Peak Flow Screen	1	0.5	1	0	0	$ -
Washer/Compactor (PF)	1	5	1	0	0	$ -
Influent Pumps	3	40	1	8760	261,293	$ 18,291
Peak Flow Pumps	3	40	2	0	0	$ -
Peak Flow Clarifier Mechanism	1	0.75	1	0	0	$ -
Grit Pump	1	5	1	8760	32,662	$ 2,286
Grit Classifier	1	5	1	8760	32,662	$ 2,286
Anaerobic Mixers	3	2.7	3	8760	52,912	$ 3,704
Aeration Blowers	5	150	4	8760	3,919,399	$ 274,358
Final Clarifier Mechanisms	3	0.5	3	8760	9,798	$ 686
Intermediate PS Pumps	5	40	1	8760	261,293	$ 18,291
Filter Backwash Pumps	2	25	1	5840	108,872	$ 7,621
Disc Filters	2	2	1	8760	13,065	$ 915
UV Disinfection (in kW)	2	32	1	8760	280,320	$ 19,622
RAS Pumping	3	14.8	2	8760	193,357	$ 13,535
Digester Blowers	5	125	4	8760	3,266,166	$ 228,632
Dewatering Pumps	3	10	2	2086	31,111	$ 15,555
Polymer Feed	2	5	2	2086	15,555	$ 7,778
Screw Press	1	5	2	2086	15,553	$ 7,777
Total Power Consumption Cost						$ 623,850

Chemical Consumption	Chemical Requirement at Basis of Analysis (gpd)	Operating Days/year	Chemical Usage (gal/year)	Chemical Unit Cost ($/Gal)	Extended Cost
Alum	35	365	12,775	$ 3.30	$ 42,158
Carbon	25	365	9,125	$ 3.00	$ 27,375
Polymer	16	261	4,171	$ 14.50	$ 60,486
Total Chemical Cost					$ 130,018

Total	**$ 1,042,094**

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