WASTEWATER TREATMENT FACILITY

TREATMENT AND HYDRAULIC EXPANSION STUDY

FOR

WEST MEMPHIS UTILITY COMMISSION

CITY OF WEST MEMPHIS, ARKANSAS

STATE OF ARKANSAS
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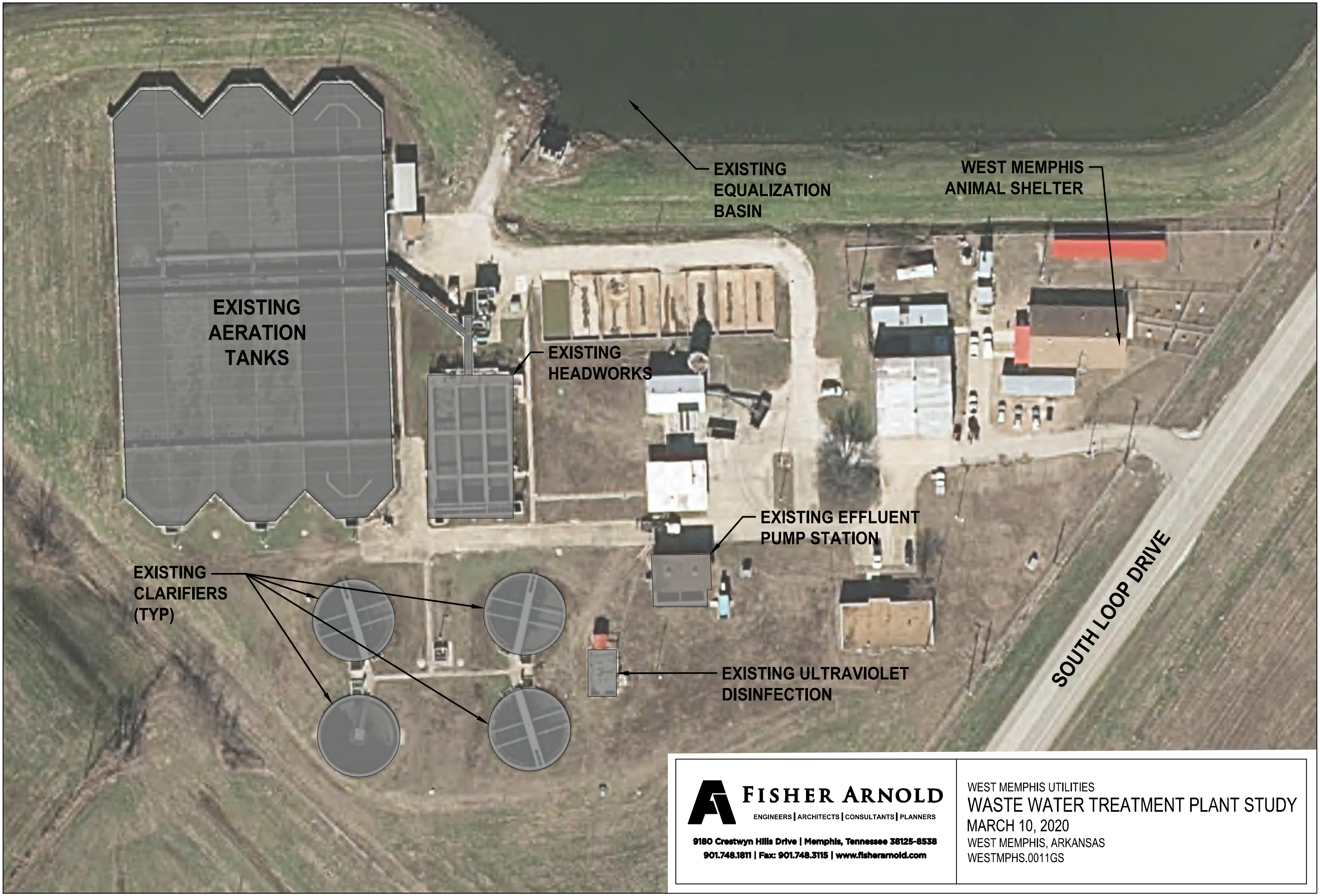
I. INTRODUCTION

The City of West Memphis operates one wastewater treatment facility (WWTP) that receives residential, commercial and industrial wastewater generated within the city limits and surrounding area. Treated final effluent is discharged into the Mississippi River. This treatment facility, and its ability to hydraulically convey and biologically treat all flows directed to it from the city's collection system, is the focus of this report. The existing treatment system consists of equalization (EQ), preliminary treatment, oxidation ditch activated sludge, and ultraviolet (UV) disinfection. The WWTP is located on the city's south side. The Arkansas Department of Environmental Quality (ADEQ) permits the discharge to the Mississippi River under NPDES Permit number AR0022039. The NPDES Permit specifies the effluent limits and monitoring requirements for the treatment facility.

The oxidation ditch facility currently must meet secondary treatment discharge standards. These NPDES effluent limits are noted below:

Table 1. Current NPDES Effluent Limits

BOD_5	30 mg/L (monthly average)
TSS	30 mg/L (monthly average)
Ammonia-N	No limit
D.O.	2.0 Monthly Min.
Total P	No limit
Total N	No limit
Fecal Coliform	200/100 mL (summer)
Fecal Coliform	1,000/100 mL (winter)



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At the present time, nutrient limits have not been proposed by the ADEQ. However, there is no guarantee that nutrient limits will not be added to the NPDES permit when it is reissued again. The design alternatives considered in this report have been developed in light of the possibility that nutrient limits could indeed become effective in the future. In the meantime, samples for effluent Total Nitrogen and Total Phosphorus should be taken by the WWTP staff at least on a monthly basis.

The WWTP is capable of meeting current discharge standards on a consistent basis during dry-weather periods. However, during some wet-weather periods (with significant infiltration/inflow coming to the plant), untreated wastewater may be bypassed around the liquid treatment process and combined with the liquid treatment process effluent for final discharge to the Mississippi River. During these bypass events, the combined effluent (untreated plus treated wastewater) may or may not satisfy the specific effluent limits noted above. Moreover, the wastewater collection system has several overflows each year that send untreated wastewater to local surface waters. These overflows and bypass episodes are unacceptable under current NPDES regulations and therefore must be eliminated. As identified in flow monitoring and modeling efforts by the RJN Group during 2018 and 2019 (see *Wastewater Collection System Evaluation-Flow Monitoring Technical Memorandum, November 2018, and Wastewater Capacity Analysis and Preliminary Augmentation Options Final Report, December 2019*), it is expected that during a 2-year, 24-hour storm event, up to 20 MGD of collected wastewater is directed to the WWTP. During these times of high flow, those flows exceeding approximately 7 MGD are diverted to the WWTP's EQ Basin. This EQ Basin has a working volume of approximately 95 million gallons. However, once this EQ Basin is full, any flows in excess of 7 MGD must be bypassed by plant personnel to prevent the EQ Basin from overtopping. This is obviously a violation of the facility's NPDES permit.

Because of collection system problems and WWTP bypass problems noted above, the ADEQ has placed West Memphis under a Consent Order to take appropriate remedial measures in the near future. Consequently, the collection system needs to be rehabilitated and the WWTP needs to have a more reliable and robust treatment process that can handle extreme high-flow events. Clearly, the WWTP must be renovated to provide all incoming wastewater with excellent BOD removal, TSS removal, and disinfection. *The purpose of this facilities plan is to propose WWTP modifications to solve existing problems and provide the City of West Memphis reliable treatment capacity for the next 20 years.* This will allow the city to comply with the Consent Order while at the same time allowing for future commercial and industrial growth. With the American economy in excellent shape presently, industrial growth in the West Memphis area should be anticipated. New industries will provide jobs and increase the tax base while at the same time potentially discharging significant hydraulic and organic loadings to the city's sanitary sewer system. West Memphis needs to be in position to accommodate these prospective increased industrial wastewater loadings.

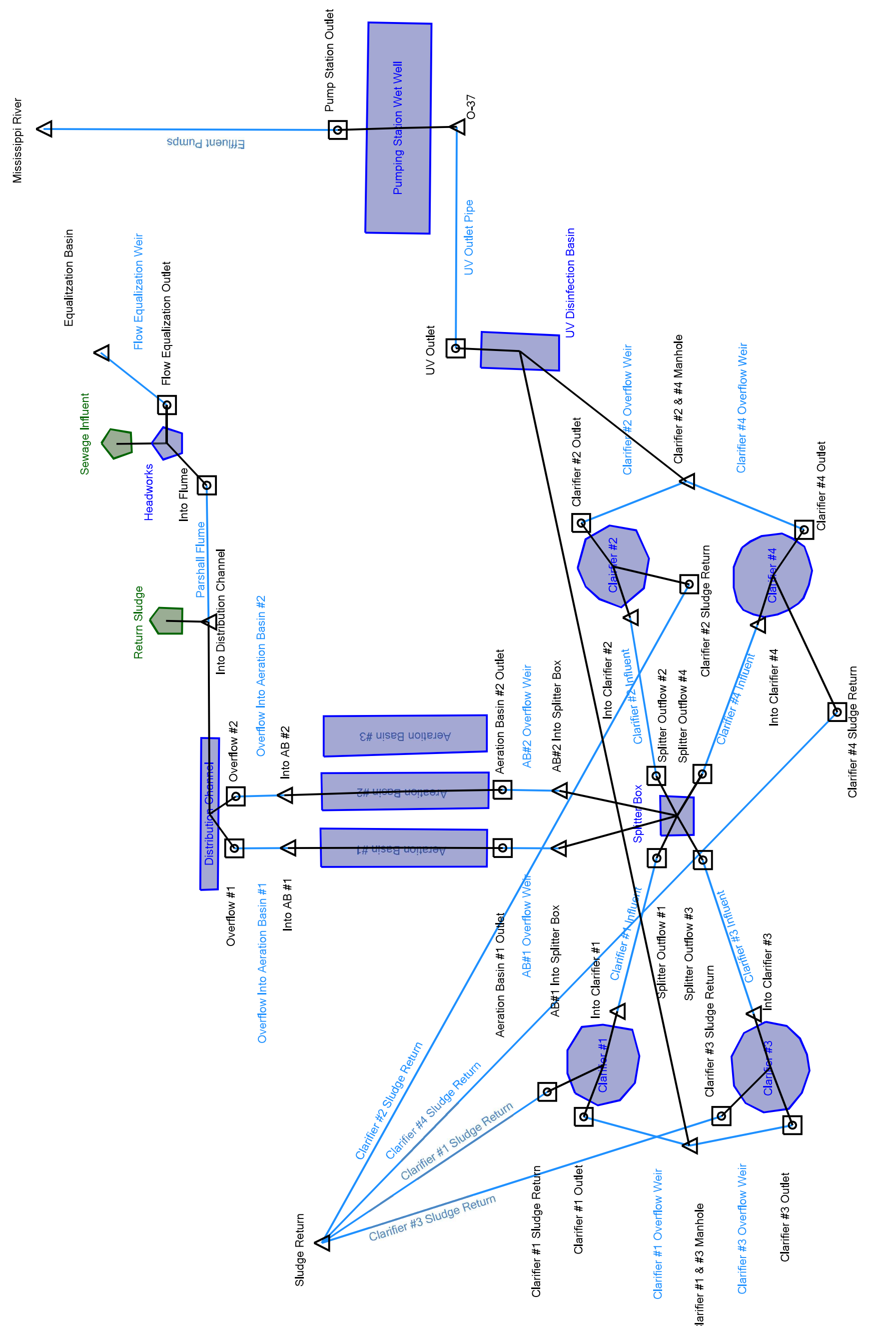
The West Memphis WWTP is an oxidation ditch process that was designed to achieve BOD removal, TSS removal, nitrification, and disinfection. This oxidation ditch process has been operating for about 35 years. This WWTP is designed to handle 6.3 million gallons per day (mgd) (average daily basis) of municipal wastewater. If the equalization (EQ) basin is not full, a maximum daily flow rate of about 7 mgd of raw wastewater can be directed through the liquid treatment process without the need to divert influent to the EQ basin. If the EQ basin is not full, the liquid treatment process can handle about 7 mgd of raw wastewater; thus, influent flow rates above 7 mgd will be diverted to the EQ basin for storage until the EQ basin becomes full. If the EQ basin is mostly full, the liquid treatment process can handle about 7 mgd of raw wastewater; thus, influent flow rates above 7 mgd (with the EQ basin full) will be diverted around the liquid treatment process. This bypass scenario needs to be eliminated once the WWTP is upgraded in the near future.

Due to the current hydraulic overloading of this WWTP during certain wet weather events, limited ability to accommodate future growth, and certain unit process and hydraulic limitations within this facility, it is recommended that the City explore feasible options to alleviate these concerns. This Report is intended to identify and highlight those hydraulic and unit process issues within this facility that limits its ability to meet the requirements of its NPDES Permit. This Report will further evaluate the City's options and to propose a wastewater treatment approach that will meet the NPDES permit requirements consistently and in the most cost-effective manner while at the same time eliminating bypasses of raw wastewater to the receiving stream during the wet weather design storm events.

II. HYDRAULIC ASSESSMENT OF EXISTING WASTEWATER TREATMENT FACILITY UNIT PROCESSES

As is typical for many extended aeration facilities in this region, the flow path of incoming raw sewage, once delivered to the head of the plant from the collection system, is by gravity flow through a series of open channels, treatment basins, interconnecting piping, and finally through an effluent pumping station where the treated wastewater is delivered to the receiving stream. Typically, these channels, basins and interconnecting piping are sized not only for the average daily flow, but also for expected peak flows as well as added return activated sludge between the clarifiers and aeration basins. However, the West Memphis WWTP does experience flow restrictions that limit its ability to handle flows that exceed its average daily capacity. These restrictions directly result in bypass events reported by the City to ADEQ. As required by ADEQ's Consent Administrative Order (CAO), West Memphis is to determine the hydraulic capacity of this WWTP by analyzing each unit process and interconnecting channels and piping to identify those limiting structures that result in these reported bypasses.

Utilizing available construction plans supplemented by on site observations and field measurements, Fisher Arnold has modeled the WWTP. Field measurements were obtained in order to confirm and/or establish the measurements of the headworks, Parshall flume, influent channels, aeration basins and weirs, clarifiers, effluent piping from clarifiers to UV disinfection, and effluent pumping station. Additionally, field measurements were provided Fisher Arnold of the equalization basin in order to establish the storage capacity of the basin.



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WMU WWTP - Hydraulic Model Layout

All flows generated within the WMU collection system are transferred to the WWTP via two sewage lift stations. These flows, delivered via 30-inch and 20-inch diameter force mains, discharge at the headworks structure at the WWTP. The headwork structure at this WWTP consists of an elevated 13’-6” by 7’-6” by 6’ deep basin where a 20-inch and 30-inch force main from the collection system empty. It is also at this location that a 36-inch line connects this headwork basin to the facility’s equalization basis. This line is separated from the incoming force mains by an adjustable slide gate that is manually operated during time of high incoming flow so that excess flows can be diverted to the equalization basin. The combined flows from these two force mains are then directed to parallel open concrete channels, one 3’-6” wide and the other 3’-0” wide. Installed within these channels are two mechanical bar screens, each rated for a maximum flow of 10 MGD. These units are new, both installed within the last 12 months. Immediately downstream of these two units, flow is combined into a common channel that at one time flowed to a grit removal system. During a past plant modification, this combined flow was diverted around the grit removal system and redirected to a new open channel that directs flow to the aeration basins. This channel, measuring approximately 4’-0” in width with an approximate depth of 2’-6”.

Existing Headwork Structure Showing 36" to EQ Basin, 30" and 20" Influent Force Main

Existing Influent Flow Distribution Channel atop Aeration Basins

The combined influent and return sludge flow is then distributed into three parallel operated aeration basins, via a long open channel that spans atop the three aeration basin with three wall openings that theoretically allows flow to be equally split into each of these basins. In reality, the operator must manually adjust slide gates to try and regulate flow into each basin with no way to physically measure the actual flow entering each basin. Additionally, it is plainly evident that the channel is of insufficient cross-sectional area to convey the influent flow when combined with the return sludge. At the location where the sludge is returned to this influent channel, the combined volume causes the channel to overtop when total flow (influent plus return) exceeds 10 MGD.

Overflowing condition of Flow Distribution Channel Immediately Downstream of RAS Connection

Each aeration basin has a volume of approximately 2.1 million gallons. Effluent flow from each aeration basin is controlled by a rectangular weir that has an 8'-0" long crest and 2'-0" depth. The crest of each effluent weir is 2'-0" below the top of the aeration basin walls. Assuming the plant is receiving an average daily flow of 6.3 MGD and the operator is returning at a rate of 125%, the total flow exiting each basin is 4.73 MGD. This results in a total flow depth over each weir of 0.43 feet. At this flow, there remains over 18 inches of freeboard. Once flow exists each aeration basin, it is collected in a 24-inch diameter buried pipe that directs this combined flow into a splitter box where flow is then distributed to four 62-foot diameter clarifiers.

Once flow is collected into this splitter box, it is directed to four clarifiers; one center fed, and the other three utilizing a peripheral feed. Flow enters each clarifier via a 16-inch diameter pipe. Based upon the diameter of each clarifier, they can theoretically handle 9.6 MGD at a surface overflow rate of 800 gpd/SF. Effluent flow from each clarifier is through an underground pipe network that increases in size from 16-inches from each unit, increasing to 24-inches and ultimately 36-inches in diameter once all four clarifiers are connected.

View of Effluent Weir of Original Peripheral Fed Clarifier

View of Center Fed Clarifier

From the clarifiers, all flow is then delivered to a UV disinfection system. This UV system is situated within one open channel structure and is rated for a maximum flow of 10 MGD. This UV system replaced the original chlorine contact system in 2005. From the UV channel flow is then directed to the WWTP's effluent pump station through a 36-inch gravity line. This effluent pump station is equipped with four dry-pit submersible pumps; three 20-inch units, and one 14-inch unit. Combined, these pumps can flow over 23 MGD. Effluent from this pump station is delivered to the Mississippi River via a 30-inch force main.

Existing UV Channel and Equipment

Effluent PS Wetwell

With this information, Fisher Arnold was able to prepare hydraulic models of the overall system and specific portions of the system. The two primary hydraulic modeling software programs utilized to analyze the treatment flow were the Bentley programs PondPack v.8i and CivilStorm v.8i. These programs provided a means of analyzing various aspects of the treatment train to identify limitations as related to maximum possible treatment flow before problems occurred that would result in a breakdown of the treatment train at some point or an overflow. PondPack was used primarily to determine where volume capacity issues occurred, while CivilStorm allowed for a more detailed analysis of the hydraulics of the conveyance conduits within the facility, such as the flume channels and the piping. These models were tested with various influent flow rates, including the permitted flow rate of 6.3 MGD and the storm rate established by the *West Memphis Wastewater Capacity Analysis and Preliminary Augmentation Options* report provided by RJN Group.

A limitation of both models is the inability to model the opening of the equalization basin gate in the headworks, which results in the diversion of that portion of the influent that is greater than the downstream treatment capacity of the facility. When the gate is opened, the excessive portion of the influent is diverted to the equalization basin. Due to this limitation, Fisher Arnold performed graphical calculations to determine the volume of diverted flow that would be directed to the equalization basin during the 72-hour routing of the design storm, based on a 24-hr, 2-year return rain event. Based on field data provided to Fisher Arnold, it was calculated that the storage capacity of the EQ basin is 95.3 million gallons. According to the design flow hydrograph provided by RJN Group, if the influent entering the treatment facility is limited to a peak flow of 5 MGD, that over a 72-hour period during a 24-hr, 2-yr storm event, the amount of flow that would need to be diverted to the EQ basin is 13.4 million gallons. If the amount being treated by the facility is the permitted flow of 6.3 MGD, the volume diverted to the EQ basin would be reduced to 6.3 million gallons. If the EQ basin is managed such that it can be pumped through the treatment system during low flow periods to recapture storage area, our calculations show that the excessive influent from several days of 24-hr, 2-yr storm events should be able to be stored in the EQ basin without overtopping. It must be noted that if several rain events occur in succession, the limited time for pumping stored influent from the EQ basin and back through the treatment facility could cause storage issues. The 5 MGD limitation is due to the maximum combined flow of 10 MGD that can be conveyed within the conveyance channel to the aeration basin without overtopping the channel walls.

Overtopping of Influent Channel at Head of Aeration Basins

Utilizing CivilStorm, various effluent flow rates were analyzed for the clarifiers routing to the UV basin, in order to establish the maximum effluent that could be conveyed to the UV basin before hydraulically the effluent back up into the clarifier. The models for the clarifiers were based on the weir/rim edge of the clarifier being at an elevation of 209.67. The model confirms that the piping that conveys the effluent from the clarifiers to the UV basin is capable of conveyance of the total effluent without backup for treatment flows up to and including the permitted flow rate of 6.3 MGD. However, at this rate, several of the pipes within the pipe network between the clarifiers and the UV basin would function under pressure. 6.3 MGD is very close to the absolute maximum that can be conveyed before effluent backs up and cannot be released from the clarifiers. As noted in the RJN Group report, during a 24-hr, 2-yr storm event the total flow being run through the facility exceeds the permitted flow rate of 6.3 MGD.

Based on the results of further modeling it was determined that between 6.5 MGD and 6.8 MGD of total influent flow, effluent backs up into some of the clarifiers.

III. ASSESSMENT OF EXISTING WASTEWATER TREATMENT FACILITIES

A. Preliminary Wastewater Treatment

The WWTP has an equalization (EQ) basin, which has a maximum volume of 95 million gallons (mil gal). During high-flow periods, some of the incoming raw wastewater is diverted to the EQ basin for storage prior to ultimately being sent into or around the plant. During high-flow periods, the influent flow rate can reach as high as 25 mgd. Since only about 7 mgd can be sent through the oxidation ditch process without hydraulically overloading the final clarifiers, any flow rates above 7 mgd are either diverted to the EQ basin or are diverted around the liquid treatment process depending on how much wastewater is in the EQ basin. All final effluent is pumped to the river by the effluent pump station, which can pump up to 23 mgd of effluent to the Mississippi River.

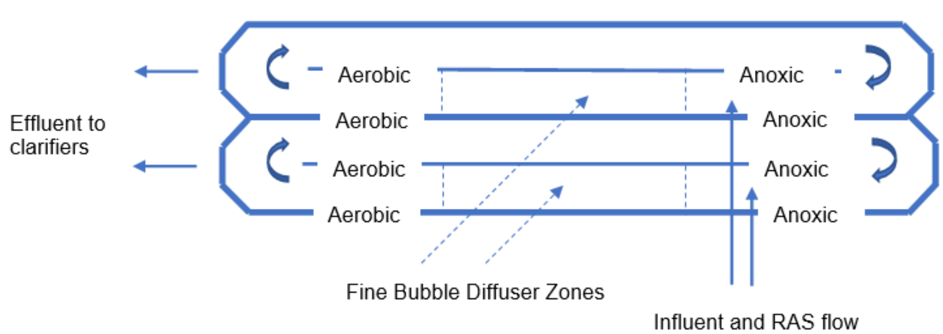
Upon entering the liquid treatment process, raw and (as necessary) equalized wastewater flows into two mechanically cleaned bar screens. Each bar screen can handle up to 10 mgd of wastewater. Thus, 20 mgd can be adequately handled by the mechanical bar screens. However, as discussed previously, only 7 mgd can be sent into the liquid treatment process due to final clarifier hydraulic limitations. The mechanical bar screens are relatively new and are working very well. After exiting the bar screens, the wastewater flows directly to the oxidation ditch process. There is an old grit removal basin (Pista Grit) that is no longer in service. Consequently, grit removal is not provided. It is believed that most of the grit entering the oxidation ditch process is removed in the oxidation ditches. Thus, it is expected that a significant accumulation of grit exists at the bottom of the three oxidation ditches. *Any accumulated grit needs to be removed from the three ditches in the near future to restore full treatment capacity* *to each oxidation ditch*. Although mixers and fine-bubble diffused aeration provide moderate turbulence in the oxidation ditches, the turbulence is not enough to keep the grit from settling in the ditches.

B. Oxidation Ditch Wastewater Treatment Process

The oxidation ditch process is a variation of the extended aeration process. Extended aeration activated sludge is an excellent wastewater treatment system for medium and small-scale treatment facilities. A typical oxidation ditch is a single or closed loop channel, typically 6 to 12 feet deep with 45-degree sloping sidewalls or straight sidewalls, that operates in the extended aeration mode. Each of three oxidation ditches is approximately 180 feet in length (overall) and 12 feet water depth, providing a volume of about 2.1 million gallons (mil gal). Thus, the total aeration volume is 6.3 mil gal. However, oxidation ditch #1 is currently out of service and has been out of service for about 3 years because the diffused aeration pipes at the bottom are in unusable condition. Oxidation ditches #2 and #3 are in fairly good condition and are currently in service. The aeration system consists of two 100-HP Lamson centrifugal blowers, one 50-HP Lamson centrifugal blower, and two 150-HP Lamson centrifugal blowers. Although these blowers are about 20 years old, they have been very reliable and they perform extremely well. The diffusers near the bottom of the ditches are ceramic disk diffusers, which provide fine bubbles to enhance oxygen transfer capability. Typically, one 100-HP blower, one 50-HP blower, and one 150-HP blower are operating and supply adequate oxygen to the two active oxidation ditches at current loadings.

Each oxidation ditch is also equipped with two submerged 7.5-HP banana-blade mixers that enhance mixing but do not provide any oxygen transfer. The primary purpose of the mixers is to impart a gentle horizontal velocity to the mixed liquor in the ditches. In most oxidation ditches, the mechanical aerators typically used will provide an average horizontal velocity of about 1 foot per second to maintain biomass in suspension and provide a total liquid circulation time in the ditches of 5 to 15 minutes. In most conventional oxidation ditches, the flow rate of the mixed liquor around the ditch channel (due to mechanical aeration) usually is twenty to forty times the influent flow rate. This, in effect, makes oxidation ditches completely mixed activated sludge reactors. However, in this oxidation ditch process, the circulation time is believed to be considerably less because of moderate horizontal liquid movement around the ditches. Moreover, the ceramic disk diffusers are located only on one side channel of each oxidation ditch. This creates an aerobic zone (DO = 1 to 2.5 mg/L as measured by plant staff recently) primarily on the south portion of each ditch and an anoxic zone (DO = 0.0 to 0.3 mg/L as measured by plant staff recently) primarily on the north portion of each ditch. The anoxic zones facilitate denitrification and allow the process to achieve relatively good nitrogen removal (as shown by effluent nitrite/nitrate-N data during the last two years and recent composite sampling five days per week for two weeks). See the following drawing (Figure 1) of the two operating ditches.

Figure 1. Schematic Drawing of the Two Operating Ditches



The diffused (fine bubble) aeration system supplies oxygen to the activated sludge biomass and provides some mixing of biomass in the basin. A high degree of nitrification (conversion of ammonia to nitrate) is often achieved because of the long solids retention times (SRTs) utilized. An oxidation ditch process has considerable amounts of cell fragments due to the extended aeration process and long solids retention time (SRT); note that SRT = sludge age. SRT is the approximate time in days that biomass reside in the activated sludge system before the biomass exits the system as waste activated sludge (WAS) or as suspended solids (TSS) in the effluent.

Separation of the treated effluent from the biomass is provided in four final clarifiers that are 62 feet in diameter. Three of these clarifiers were constructed with the original plant and are peripheral-feed, center draw-off clarifiers that are 10 feet deep. The final clarifier was constructed at a later date and is a center-feed, peripheral draw-off unit that is 12 feet deep. **The final clarifiers are a major problem for the West Memphis WWTP.** Based on a maximum surface overflow rate of 800 gpd/ft^2, they can hypothetically handle up to 9.6 mgd of flow rate and continue to perform well. In actual practice, if wastewater flow rate (not including the return activated sludge (RAS) flow rate) exceeds 6.8 mgd, clarifier water depth increases and submerges the effluent V-notch weirs due to excessive head loss in the combined clarifier effluent pipe system leading to the UV basin. This is an unacceptable condition that leads to poor settling and the possibility of excess TSS escaping into the final effluent. *Clearly, the final clarifiers are the hydraulic "bottleneck" of the oxidation ditch process that contributes to raw wastewater bypassing during high-flow periods during and following significant rainfall events*.

The clarifier effluent flows to an ultraviolet light (UV) disinfection system containing several banks of lamps that are designed to destroy pathogenic microorganisms prior to discharge into the receiving stream. The UV system has worked extremely well in destroying pathogens and ensuring compliance with effluent limits on fecal coliform. However, the UV system is susceptible to flooding under certain conditions, which creates problems in terms of system reliability and performance. Therefore, a new UV system is needed that will not be susceptible to flooding.

C. Summary Assessment & Bio-Kinetic Evaluation of the Oxidation Ditches

An extended aeration process is relatively easy to maintain and manage, and it is stable against load fluctuation. The process cost of treatment is generally comparable to other biological processes in the range of wastewater flows between 0.1 mgd and 10 mgd. In this oxidation ditch process using only two ditches currently, the solids retention time usually is 18 to 30 days, aeration basin detention time is 10 to 25 hours, and the actual BOD_5 loading is 10 to 15 lbs $BOD_5/1000\ ft^3$ of aeration volume per day. Overall, the oxidation ditch process produces a high-quality effluent (about 95% BOD_5 and TSS removal along with excellent nitrification).

If the oxidation ditch process is expected to provide efficient wastewater treatment for the City of West Memphis for the next 20 years, the treatment system will need to be modified to address existing deficiencies. These deficiencies are primarily related to the facts that this WWTP has severe hydraulic limitations related to less than optimal use of the EQ basin, inadequate flow distribution to the aeration basins, inefficient design/operation of the final clarifiers, lack of grit removal, inadequate horizontal velocity and relatively poor mixing in the oxidation ditches, and flooding concerns in the UV disinfection system. Modifications to the treatment process will be discussed in detail in a subsequent section of this report.

The existing West Memphis oxidation ditch activated sludge process was evaluated using Dr. Larry Moore’s Bio-Tiger Model. This model was developed in 2017 for the U.S. Department of Energy (USDOE) via a funded research project conducted by the University of

Memphis Civil Engineering Department. Prior to 2017, Dr. Moore used a spreadsheet model that he developed in 2010 to model the activated sludge process. Between 2010 and 2017, he used the initial model to evaluate over 100 activated sludge processes throughout the U.S. In 2013, EPA Region 4 became aware of Dr. Moore's spreadsheet model because he conducted about 30 energy assessment projects in which EPA participated with him. Dr. Moore trained EPA engineering staff how to effectively use the model, especially for energy assessment projects. EPA subsequently made the USDOE aware of the utility and simplicity of Dr. Moore's model. Thus, USDOE funded the research project noted above so that Dr. Moore could make the model user friendly and could write a user manual for the Bio-Tiger Model. Consequently, USDOE, EPA Region 4, and the Tennessee Department of Environment & Conservation (TDEC) currently use the Bio-Tiger model for simulating activated sludge. The Bio-Tiger Model and user manual are available on the USDOE (Industrial Assistance Centers) and TDEC websites.

Using Dr. Moore's Bio-Tiger Model for the West Memphis WWTP, detailed operating data at *design loadings* for the existing oxidation ditch process (three ditches in service) were developed and are provided below in Table 2. Detailed operating data at *current summer loadings* for the existing oxidation ditch process (two ditches in service) are provided in Table 3. Detailed operating data at *current winter loadings* for the existing oxidation ditch process (two ditches in service) are provided in Table 4. West Memphis Utilities' Director of Wastewater, Paul Holloway, has confirmed the Bio-Tiger Model simulations of the West Memphis oxidation ditch process are reasonably accurate when compared to actual plant operating data for the last two years.

Table 2. Operating Conditions (Existing Design)	
Total average daily flow rate (mgd)	**6.30**
Aeration volume in service (mil gal)	**6.30**
Influent BOD5 concentration (mg/L)	**150**
Influent BOD5 mass loading (lb/day)	**7,881**
Sec ww Oxid N load (lb/day)	**1,419**
Sec ww TSS load (lb/day)	**10,508**
F/M ratio	**0.082**
Solids Retention Time (day)	**28.0**
MLSS (mg/L)	**2,521**
MLVSS (mg/L)	**1,835**
TSS Sludge Production (lb/day)	**4,101**
TSS in activated sludge effluent (lb/day)	**630.5**
Total Oxygen Requirements (lb/day)	**15,384**
Total Oxygen Req'd W/Denit. (lb/day)	**13,215**
Total oxygen supplied (lb/day)	**15,198**
Mixing intensity in the reactor (hp/mil gal)	**79**
RAS flow rate (mgd)	**2.45**
RAS recycle percentage (%)	**38.9**
WAS flow rate (mgd)	**0.055**
RAS TSS concentration (mg/L)	**9,000**
Total sludge production (lb/day)	**4,731**
Reactor Detention Time (hr)	**24.0**
VOLR (lb BOD/(thou cu ft-day))	**9.36**
Effluent CBOD5 (mg/L)	**4.7**
Effluent TSS (mg/L)	**12.0**
Effluent Ammonia-N (mg/L)	**0.15**
Effluent NO3-N with denitrification (mg/L)	**6.2**

Table 3. Current Operating Conditions (Summer)	
Total average daily flow rate (mgd)	**4.00**
Aeration volume in service (mil gal)	**4.20**
Influent BOD5 concentration (mg/L)	**170**
Influent BOD5 mass loading (lb/day)	**5,671**
Sec ww Oxid N load (lb/day)	**967**
Sec ww TSS load (lb/day)	**6,672**
F/M ratio	**0.093**
Solids Retention Time (day)	**26.0**
MLSS (mg/L)	**2,363**
MLVSS (mg/L)	**1,736**
TSS Sludge Production (lb/day)	**2,816**
TSS in activated sludge effluent (lb/day)	**367.0**
Total Oxygen Requirements (lb/day)	**10,765**
Total Oxygen Req'd W/Denit. (lb/day)	**9,315**
Total oxygen supplied (lb/day)	**10,836**
Mixing intensity in the reactor (hp/mil gal)	**71**
RAS flow rate (mgd)	**3.58**
RAS recycle percentage (%)	**89.6**
WAS flow rate (mgd)	**0.068**
RAS TSS concentration (mg/L)	**5,000**
Total sludge production (lb/day)	**3,183**
Reactor Detention Time (hr)	**25.2**
VOLR (lb BOD/(thou cu ft-day))	**10.10**
Effluent CBOD5 (mg/L)	**4.5**
Effluent TSS (mg/L)	**11.0**
Effluent Ammonia-N (mg/L)	**0.18**
Effluent NO3-N with denitrification (mg/L)	**6.5**

Table 4. Current Operating Conditions (Winter)	
Total average daily flow rate (mgd)	**6.30**
Aeration volume in service (mil gal)	**4.20**
Influent BOD5 concentration (mg/L)	**140**
Influent BOD5 mass loading (lb/day)	**7,356**
Sec ww Oxid N load (lb/day)	**1,261**
Sec ww TSS load (lb/day)	**7,881**
F/M ratio	**0.113**
Solids Retention Time (day)	**18.0**
MLSS (mg/L)	**2,449**
MLVSS (mg/L)	**1,859**
TSS Sludge Production (lb/day)	**4,189**
TSS in activated sludge effluent (lb/day)	**578.0**
Total Oxygen Requirements (lb/day)	**12,312**
Total Oxygen Req'd W/Denit. (lb/day)	**10,594**
Total oxygen supplied (lb/day)	**12,363**
Mixing intensity in the reactor (hp/mil gal)	**71**
RAS flow rate (mgd)	**4.47**
RAS recycle percentage (%)	**71.0**
WAS flow rate (mgd)	**0.085**
RAS TSS concentration (mg/L)	**5,900**
Total sludge production (lb/day)	**4,767**
Reactor Detention Time (hr)	**16.0**
VOLR (lb BOD/(thou cu ft-day))	**13.10**
Effluent CBOD5 (mg/L)	**6.4**
Effluent TSS (mg/L)	**11.0**
Effluent Ammonia-N (mg/L)	**0.72**
Effluent NO3-N with denitrification (mg/L)	**4.9**

Based on the last two years of plant operating records and the recent sampling data, the following effluent quality was achieved at average conditions (Table 5).

Table 5. Current Effluent Quality for the West Memphis WWTP

BOD_5	8 mg/L
TSS	12 mg/L
Ammonia-N	0.5 mg/L
D.O.	≥ 2 mg/L
Total P	3.6 mg/L
TKN	2.8 mg/L
Nitrite/Nitrate-N	3.7 mg/L
Total N	6.5 mg/L

D. Sludge Treatment and Disposal

Waste activated sludge (WAS) is sent to the thickening/decanting tank (volume = 0.135 mil gal) prior to aerobic digestion. Thickened WAS is pumped into the aerobic digester (volume = 0.135 mil gal), which is used to destroy organic suspended solids via aerobic digestion. It is estimated that currently about 0.025 mgd of thickened sludge (1.5% TSS) is pumped into the aerobic digester on an average day. The sludge is aerobically digested for an average time of about 5 days. Microbial activity beyond cell synthesis is stimulated by aeration, oxidizing degradable organic matter and cellular materials into carbon dioxide, water, and nitrate-N. Oxidation of cellular material is known as endogenous respiration, which is the predominant reaction during aerobic digestion of waste activated sludge. Because oxidation ditches typically operate at SRTs of 20 to 40 days, the biological solids are relatively well digested prior to aerobic digestion. Thus, further oxidation in the aerobic digester may be limited. Benefits of aerobic digestion include less odor potential, destruction of volatile solids, and better sludge dewaterability. Oxygen for aerobic digestion at this plant is provided by two centrifugal blowers, each 30 HP. Each blower can deliver 750 cfm of air to the digester, and each blower is only two years old. Typically, one blower is in service at most times. With one blower running, the mixing intensity in the digester is 222 HP/mil gal (more than adequate). Approximate oxygen requirements for the digester at existing loading are shown below:

Average sludge production (TSS) ≈ 3500 lb/day

Average sludge production (VSS) ≈ 2400 lb/day

At SRT of 22 days (average) in the oxidation ditches, about 25% VSS destruction is achieved. Assume an additional 10% VSS destruction in the aerobic digester.

VSS destruction in aerobic digester ≈ 0.1 x 2400 lb/day = 240 lb/day

Oxygen required in digester = 2.3 x 240 lb/day = 550 lb/day

Assume field OTR of the digester blower = 0.9 lb/(HP-hour)

Blower run-time ≈ 550 lb/day ÷ (0.9 x 24 lb/HP-day x 30 HP) = 0.8 day

Thus, the 30-HP blower should be able to deliver the necessary oxygen by running about 20 hours per day. DO in the digester should be 1 to 2 mg/L.

A third tank (volume = 0.135 mil gal) is used to store supernatant that is decanted from the thickening/decanting tank. Supernatant is returned to the headworks. Digested sludge is sent from the aerobic digester to the belt filter press two to three days per week for dewatering.

The thickened and digested waste activated sludge (WAS) is then sent to the belt filter press for dewatering. According to plant staff, the belt filter press is operated about 9 hours/day for 2.5 days per week to dewater the sludge. The 1-meter belt filter press can process about 1,000 lb/hr of dry solids (TSS). Approximately 3,200 lb/day of digested dry solids (TSS) are produced by the digester, which is about 22,000 lb per week. The Andritz belt filter press is very reliable and performs extremely well. At design conditions (2040 loadings), the belt filter press will need to operate 9 hours/day for 5 to 6 days per week to dewater the sludge. Therefore, upgrading of the sludge dewatering process is not necessary at the present time.

IV. ASSESSMENT OF FUTURE CONDITIONS

A. Population Forecast

The population estimate for the City of West Memphis at present is 24,636 persons. Based on input from city planners, the following table was developed which indicates the expected growth in West Memphis over the next 20 years:

Table 6: West Memphis, Arkansas Population Estimates

Date	West Memphis Population
2020	24,636
2023	24,710
2027	24,810
2030	24,880
2034	24,980
2037	25,060
2040	25,130

Evaluating the City's growth rate from 2020 to 2040, the population is expected to increase by 0.1% on an annual average basis. Using an annual growth rate of 0.1%, the projected population in 2040 would be 25,130 people.

The projected population for 2040 is determined with the following analysis:

$$F = P(1+i)^N$$

Where:

F	=	Projected future population
P	=	Population from 2020
i	=	Growth rate
N	=	Number of years

B. Design Conditions for the Year 2040

In the last 24 months, approximately 365 gal/cap-day of wastewater was produced during the month with the highest average daily flow rate. The average daily flow rate in April 2019 was 9.0 mgd (based on plant operating data). This includes domestic, commercial, industrial, and public wastewater as well as significant infiltration and inflow (I/I). Evaluating the wastewater flow in 2040, designing for the month with the highest average daily flow rate, and allowing for significant industrial growth, the average daily flow rate to the treatment facility will be 12 MGD in the design year 2040. Assuming low to medium strength municipal wastewater, the existing facilities would need to be designed to treat 16,000 lb/day (160 mg/L) BOD_5, and 20,000 lb/day (200 mg/L) TSS on an average day. For an aeration volume of 6.3 mil gal and an average daily flow rate of 12 mgd, the detention time in the oxidation ditch process at design conditions will be 12.6 hours, which is adequate. For design loadings, the volumetric organic loading rate will be 19 lb BOD_5/1000 ft^3 of aeration volume per day, which is moderately high. However, because the WWTP has secondary treatment standards in its NPDES permit, the oxidation ditches should perform well and easily satisfy the existing effluent requirements. *When the renovated plant incurs average daily BOD_5 loadings exceeding 13,000 lb/day, the city may need to consider constructing a fourth oxidation ditch*. This loading condition is anticipated to occur about year 2035. For *average daily design loadings*, the operating conditions (summer) for this oxidation ditch process are shown in Table 7 (based on Bio-Tiger Model simulation). For *peak daily design loadings*, the operating conditions (summer) for this oxidation ditch process are shown in Table 8 (based on Bio-Tiger Model simulation).

Table 7. Operating Conditions (Design Average Loadings)	
Total average daily flow rate (mgd)	**12.00**
Aeration volume in service (mil gal)	**6.30**
Influent BOD5 concentration (mg/L)	**160**
Influent BOD5 mass loading (lb/day)	**16,013**
Sec ww Oxid N load (lb/day)	**3,002**
Sec ww TSS load (lb/day)	**20,016**
F/M ratio	**0.110**
Solids Retention Time (day)	**20.0**
MLSS (mg/L)	**3,739**
MLVSS (mg/L)	**2,759**
TSS Sludge Production (lb/day)	**9,023**
TSS in activated sludge effluent (lb/day)	**800.6**
Total Oxygen Requirements (lb/day)	**30,666**
Total Oxygen Req'd W/Denit. (lb/day)	**26,246**
Total oxygen supplied (lb/day)	**26,396**
Mixing intensity in the reactor (hp/mil gal)	**131**
RAS flow rate (mgd)	**7.17**
RAS recycle percentage (%)	**59.7**
WAS flow rate (mgd)	**0.108**
RAS TSS concentration (mg/L)	**10,000**
Total sludge production (lb/day)	**9,824**
Reactor Detention Time (hr)	**12.6**
VOLR (lb BOD/(thou cu ft-day))	**19.01**
Effluent CBOD5 (mg/L)	**4.0**
Effluent TSS (mg/L)	**8.0**
Effluent Ammonia-N (mg/L)	**0.27**
Effluent NO3-N with denitrification (mg/L)	**6.6**

Table 8. Operating Conditions (Design Peak Day Loadings)	
Total average daily flow rate (mgd)	**18.00**
Aeration volume in service (mil gal)	**6.30**
Influent BOD5 concentration (mg/L)	**110**
Influent BOD5 mass loading (lb/day)	**16,513**
Sec ww Oxid N load (lb/day)	**3,002**
Sec ww TSS load (lb/day)	**30,024**
F/M ratio	**0.118**
Solids Retention Time (day)	**14.0**
MLSS (mg/L)	**3,651**
MLVSS (mg/L)	**2,664**
TSS Sludge Production (lb/day)	**12,502**
TSS in activated sludge effluent (lb/day)	**1201.0**
Total Oxygen Requirements (lb/day)	**29,739**
Total Oxygen Req'd W/Denit. (lb/day)	**25,521**
Total oxygen supplied (lb/day)	**25,552**
Mixing intensity in the reactor (hp/mil gal)	**131**
RAS flow rate (mgd)	**10.35**
RAS recycle percentage (%)	**57.5**
WAS flow rate (mgd)	**0.150**
RAS TSS concentration (mg/L)	**10,000**
Total sludge production (lb/day)	**13,703**
Reactor Detention Time (hr)	**8.4**
VOLR (lb BOD/(thou cu ft-day))	**19.61**
Effluent CBOD5 (mg/L)	**4.6**
Effluent TSS (mg/L)	**8.0**
Effluent Ammonia-N (mg/L)	**0.25**
Effluent NO3-N with denitrification (mg/L)	**4.2**

V. LONG - RANGE TREATMENT FACILITY ALTERNATIVES

A. Upgrade the Existing Oxidation Ditch Process (Parallel Operation)

One approach to solve the city's wastewater problem is to continue to use the existing three oxidation ditches in parallel, with the flow rate split relatively uniformly to each basin. The aeration system will be upgraded by installing new Orbal aerators, which will supply oxygen to the mixed liquor as well as provide excellent mixing in each basin. Each oxidation ditch will be provided with four 75-HP Orbal aerators. Installation of these new aerators will require the retrofit of new structural supports for each unit in all three ditches.

With four 75-HP Orbal aerators running in each ditch, the mixing intensity will be 143 HP/mil gal. With three Orbal aerators running in each ditch, the mixing intensity will be 107 HP/mil gal, which is adequate. At average daily design conditions, the oxygen demand will be about 26,200 lb/day. At an average operating DO concentration of 1 mg/L in the three ditches, eleven 75-HP Orbal rotors will be needed and will supply 26,400 lb/day of oxygen to the mixed liquor. One or more of the aerators will be turned off for a few hours each day to effect anoxic conditions in parts or all of each oxidation ditch to accomplish denitrification. In addition, variable frequency drives (VFDs) will be installed on each aerator to provide another means of achieving denitrification.

The standard oxygen transfer rate (SOTR) for the Orbal aerators is estimated to be 2.2 lb/(HP-hr). At an operating DO concentration of 1 mg/L, the approximately field oxygen transfer rate (field OTR) will be approximately 1.52 lb/(HP-hr). At average daily *design* conditions, eleven 75-HP Orbal rotors will be operating 21 hours per day to satisfy the oxygen demand of the incoming wastewater. Biological denitrification will occur during the 3-hour period the aerators are inactive.

At peak day loadings, the oxygen demand of the wastewater will not change much because at an equalized peak day flow rate of 18 mgd through the biological treatment process, the influent BOD_5 concentration and influent TKN concentration will be about 110 mg/L and 20 mg/L, respectively. Because of additional solids loading during wet-weather conditions, the SRT of the oxidation ditch process will be about 14 days at peak loading. Hydraulic detention time in the oxidation ditches will be 8.4 hours, which will be adequate to achieve excellent effluent quality because of reduced influent BOD_5 and TKN concentrations. As stated previously, the EQ basin pumps and controls will be re-designed to ensure that peak day flow rate into the oxidation ditch process does not exceed 18 mgd.

Four new 100-foot diameter clarifiers that have an average depth of 12 feet will be constructed adjacent to the existing final clarifiers. Important design calculations for the new final clarifiers are provided below:

Surface area of each new clarifier = π x (100 ft)^2 ÷ 4 = 7,854 ft^2

Surface overflow rate (SOR) at average daily flow rate = 3,000,000 gpd ÷ 7,854 ft^2

= 382 gpd/ft^2 (good)

Surface overflow rate (SOR) at peak daily flow rate = 4,500,000 gpd ÷ 7,854 ft^2

= 573 gpd/ft^2 (excellent)

Under worst-case conditions, each clarifier hypothetically could process 6.28 mgd (total flow rate = 25 mgd) of wastewater at a peak surface overflow rate of 800 gpd/ft^2. Under these hypothetical peak flow conditions, the final clarifiers and oxidation ditch process should continue to perform very well.

The hypothetical maximum return activated sludge (RAS) rate for each clarifier is approximately 3.75 mgd, which is about 125% of the average daily design raw wastewater flow rate. The new return sludge pumps (common wet well serving all four final clarifiers) should be capable of a maximum RAS rate of 15 mgd (125% return rate). Two RAS pumps will have a maximum flow rate of 2.5 mgd each, and two RAS pumps will have a maximum flow rate of 5 mgd each. All the RAS pumps will be provided with VFDs. At initial conditions after the plant upgrade, the average RAS pumping rate will be about 2 mgd (total) in dry weather and about 4.5 mgd (total) in wet weather. At the average daily design flow rate of 12 mgd, the average RAS pumping rate will be about 7.2 mgd (total). At the peak daily design flow rate of 18 mgd, the average RAS pumping rate may be as high as 10.4 mgd (total).

The waste activated sludge (WAS) flow rate will normally be between 0.05 mgd and 0.25 mgd; the WAS rate should be 0.11 mgd (total) at average daily design conditions. The WAS flow rate will be achieved by diverting a portion of the RAS flow rate to the sludge treatment system, or separate WAS pumps will be provided if necessary.

Another of the existing plant deficiencies is the lack of a supervisory control and data acquisition (SCADA) system that is used to control, monitor, and operate the wastewater treatment plant (WWTP). Interface to the plant control system is a sophisticated digital data link with a special protocol involving special software written for the oxidation ditch process. The software will be compatible with plant operations and will facilitate ease of operation. At least one ORP probe needs to be placed at a designated location in each aeration basin, and ORP needs to be incorporated into the software for plant operations. A DO/ORP control system may be installed in each ditch to activate and deactivate aerators and to change aerator speed as needed to promote effective control of the aerators, to promote aerator energy savings, and to ensure efficient aerobic/anoxic operation in each ditch.

Effluent BOD_5, TSS, and ammonia-N concentrations will be less than 10 mg/L, 15 mg/L, and 1 mg/L, respectively. The anticipated Total P and Total N concentrations in the effluent are 3 mg/L and 9 mg/L, respectively.

B. Upgrade the Existing Oxidation Ditch Process (Series Operation)

Another approach to solve the city's wastewater problem is to use the existing three oxidation ditches in series, with all the wastewater flowing through each ditch sequentially. This approach will require extensive modification of the hydraulic structures conveying wastewater/RAS into and out of each ditch and of the hydraulic structures conveying wastewater/RAS to the new final clarifiers. Each aeration basin will be upgraded by using new Orbal aerators, which will supply oxygen to the mixed liquor as well as provide excellent mixing in each basin. The first oxidation ditch will be provided with five 75-HP Orbal aerators; the second ditch will be provided with four 75-HP Orbal aerators; the third ditch will be provided with three 75-HP Orbal aerators. Physical modification of the original aerator support structures will be provided, or new structures will be constructed to ensure that the new surface aerators are stable and are located appropriately.

The *first oxidation ditch* will be operated at a dissolved oxygen (DO) concentration in the range of 0.0 to 0.3 mg/L and at an oxidation reduction potential (ORP) in the range of -150 to zero millivolts by controlling the number of rotors in service. The SCADA system will monitor DO and ORP in the first ditch and turn aerators on and off as necessary to keep DO and ORP in the desired range. Under these conditions, the first ditch will be designed to provide simultaneous nitrification-denitrification. Total oxygen requirements at design conditions were given by the Bio-Tiger Model as 26,200 lb/day. Aerator requirements in the first ditch are shown below:

Oxygen required = 50% of the total = 0.5 x 26,200 lb/day = 13,100 lb/day

Desired DO concentration in first ditch = 0.1 mg/L for simultaneous nitrification-denitrification

Field transfer rate of Orbal rotors = 1.72 lb/HP-hr at DO = 0.1 mg/L

* Reduction in aerator efficiency is due to field conditions adjustments for temperature, pressure, DO level, and characteristics of wastewater.

Design temperature	=	28°C
Elevation	=	210 feet
α	=	0.84
β	=	0.92
DO concentration	=	0.1 mg/L (nitrif/denitrif. zone)

O_2 requirement in first ditch from above calculations = 13,100 lb/day

Horsepower (HP) required for oxygen transfer (design needs)

= 13,100 lb O_2 /day ÷ (1.72 lb/HP-hr x 24 hr/day)
= 317 HP

To meet design needs, use five 75-HP Orbal rotors in the first oxidation ditch.

The *second ditch* will operate at a DO concentration in the range of 1.0 to 2.0 mg/L. Aerator requirements in the second oxidation ditch can be determined according to the following calculations:

Oxygen required = 33% of the total = 0.33 x 26,200 lb/day = 8,650 lb/day

Desired DO concentration in second ditch = 1.5 mg/L for excellent CBOD removal and nitrification

Field transfer rate of Orbal rotors = 1.38 lb/HP-hr at DO = 1.5 mg/L

* Reduction in aerator efficiency is due to field conditions adjustments for temperature, pressure, DO level, and characteristics of wastewater.

Design temperature	=	28°C
Elevation	=	210 feet
α	=	0.84
β	=	0.92
DO concentration	=	1.5 mg/L (nitrification zone)

O_2 requirement in second ditch from above calculations = 8,650 lb/day

Horsepower (HP) required for oxygen transfer (design needs)

= 8,650 lb O_2 /day ÷ (1.38 lb/HP-hr x 24 hr/day)
= 261 HP

To meet design needs, use four 75-HP Orbal rotors in the second oxidation ditch.

The *third ditch* will operate at a DO concentration in the range of 2.0 to 3.0 mg/L. Aerator requirements in the third oxidation ditch can be determined according to the following calculations:

Oxygen required = 17% of the total = 0.17 x 26,200 lb/day = 4,450 lb/day

Desired DO concentration in third ditch = 2.5 mg/L for excellent CBOD removal and nitrification

Field transfer rate of Orbal rotors = 1.13 lb/HP-hr at DO = 2.5 mg/L

* Reduction in aerator efficiency is due to field conditions adjustments for temperature, pressure, DO level, and characteristics of wastewater.

Design temperature	=	28°C
Elevation	=	210 feet
α	=	0.84
β	=	0.92
DO concentration	=	2.5 mg/L (nitrification zone)

O_2 requirement in third ditch from above calculations = 4,450 lb/day

Horsepower (HP) required for oxygen transfer (design needs)

$$= 4{,}450 \text{ lb } O_2\text{/day} \div (1.13 \text{ lb/HP-hr} \times 24 \text{ hr/day})$$
$$= 164 \text{ HP}$$

To meet design needs, use three 75-HP Orbal rotors in the third oxidation ditch.

Check mixing requirements in the third ditch. With three aerators operating in the third ditch, the mixing intensity will be 107 HP per mil gal (adequate to maintain biomass in suspension).

C. Upgrade the Oxidation Ditch to SBR Process

A third option for upgrading the oxidation ditch process is to convert the ditches to three SBR reactors. The primary advantage of this approach is that final clarifiers and return sludge pumping will not be needed. The three ditches will be substantially modified by installing a new diffused aeration system (flexible membrane fine-bubble diffusers) and adding an automatic decanter to withdraw treated wastewater from each SBR basin after the settling period. The existing five blowers will be used because they are very reliable, and two new blowers will be added (one 100-HP blower and one 150-HP blower). Thus, the total blower capacity for the SBR process will be 800 HP. At design conditions, it is anticipated that 600 HP of blowers will need to be operating at most times to satisfy the oxygen demand of the incoming wastewater. New waste sludge pumps and sludge collection system will be needed to waste sludge from the three SBRs. Construction of the diffusers and sludge collection system will be a challenge. The three SBRs will have four operating cycles each day, which will occur sequentially.

Each operating cycle consists of the following steps:

Mixed Fill	50 minutes (anoxic metabolism)
React Fill	70 minutes (aerobic metabolism)
React	96 minutes (aerobic metabolism)
Settle	60 minutes
Decant	84 minutes
Total Cycle Time	360 minutes

D. Miscellaneous Design Items (Part of Each Alternative Above)

1. Upgrade Equalization Facilities and Preliminary Treatment

The existing equalization (EQ) basin and pumps are inadequate to regulate pumping into the preliminary treatment process. Moreover, the EQ basin is believed to have a couple of feet of grit and other solids accumulated at the bottom. The basin will be pumped out and accumulated solids will be removed. New pumps and pump controls will be installed so that the basin can remove wastewater in a timely manner after major storm events have ended. New surface aerators would typically be installed in the EQ basin to keep the raw wastewater aerobic and to keep nearly all solids in suspension. However, plant operators have said the EQ basin has operated for several years without aeration and no significant odor problems have occurred. The renovated biological treatment process will be capable of handling influent flow rates up to 18 mgd. At flow rates exceeding 18 mgd, raw wastewater will be diverted to the EQ basin for temporary storage. Influent flow rates exceeding 18 mgd are expected to occur 10 to 20 days per year. It is estimated that peak day flow rates will average about 25 mgd. Thus, as much as 140 million gallons of influent per year will need to be diverted to the EQ basin. Clearly, in the renovated plant, influent diversion to the EQ basin will only occasionally be necessary. Thus, after plant renovation the ability to handle all peak day flow events without bypassing the liquid treatment process will be greatly enhanced.

Another modification that will be required for each alternative is the enhancement of how incoming raw sewage and RAS is introduced into the modified aeration basins. It is proposed that after raw sewage is screened through the existing bar screens, the flow will be redirected to a new channel that will allow for the installation of a grit removal system as well as a new influent flow meter.

One or two new Pista Grit basins will be installed in the area between the bar screens and the oxidation ditches. Grit removal is absolutely necessary to protect mechanical equipment (e.g., aeration equipment and RAS pumps) from clogging and abrasion caused by sand, pieces of glass, pebbles, etc. that are found in raw municipal wastewater. The grit basin(s) will be designed to handle a peak flow rate of 20 mgd. Preliminary plans are to install two Pista Grit basins, each capable of handling a peak flow rate of 10 mgd.

After flow is measured, a common influent header pipe sized to accommodate 18 MGD will be installed that connects this new grit removal and flow channel and a new distribution box located immediately north of the three aeration basins. From this distribution box, flow will be diverted to each of the three basins across rectangular weirs. RAS will be directed into each aeration basin through individual connections from the new RAS pumping station.

2. Ultraviolet Disinfection System

The ultraviolet disinfection system is in relatively good shape. However, because of the current flooding problems, the UV system must be replaced. Two UV channels will be provided, and each channel will be capable of treating a peak flow rate of 10 mgd. One of the primary advantages of using UV disinfection is that it leaves no residual in the treated wastewater to affect aquatic life in receiving waters. It is proposed that two to four vertically oriented banks of lamps be used in each UV channel. The UV system will be either a low-pressure high-output system or a low-pressure system because they use significantly less energy than a medium-pressure system. Estimated energy savings will be about $40,000 per annum if a low-pressure UV system is used.

3. Aerobic Digestion

At design loadings, it is estimated that 0.11 mgd of waste activated sludge (WAS) will be produced at average conditions; this amounts to about 9,000 lb/day of sludge solids (TSS). Thus, WAS production at design loadings will be about 150% more than the current WAS production. Therefore, another aerobic digester with a volume of 0.135 mil gal will be provided. One new 50-HP blower will be needed to supply oxygen to the aerobic digesters.

4. Lime Feed System

A lime feed system should not be needed based on the calculations provided below:

<u>Case #1: Assume no denitrification takes place</u>

Alkalinity of raw wastewater = 130 mg/L (based on recent sampling data)

Assume oxidizable nitrogen in the raw wastewater = 24 mg/L

7.1 lb of alkalinity are needed to oxidize 1 lb of ammonia-N to nitrate-N

Alkalinity required for nitrification = 7.1 x 24 mg/L = 170 mg/L

Alkalinity in final effluent = 130 – 170 = -40 mg/L (unacceptable)

Case #2: Assume denitrification takes place

Alkalinity of raw wastewater = 130 mg/L

Assume oxidizable nitrogen in the raw wastewater = 24 mg/L

Alkalinity required for nitrification = 7.1 x 24 mg/L = 170 mg/L

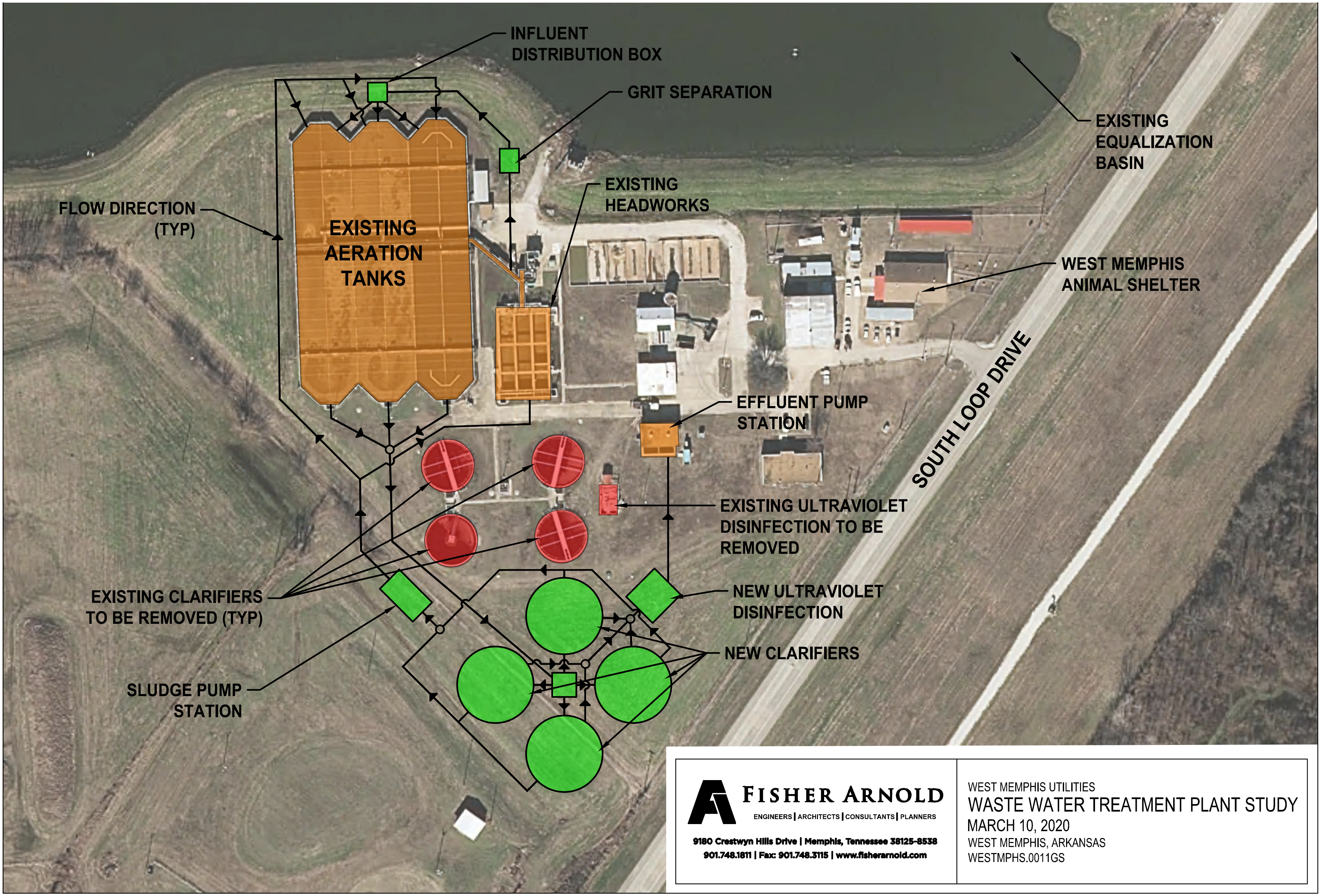
Alkalinity recovered by denitrification = 3.55 lb per lb NO_3-N reduced

Assume 83% conversion of nitrate-N to nitrogen gas = 0.83 x 24 mg/L = 20 mg/L

Alkalinity recovered by denitrification = 3.55 x 20 mg/L = 71 mg/L

Alkalinity in final effluent = 130 – 170 + 71 = 31 mg/L (barely acceptable)

If the raw wastewater has higher concentrations of oxidizable nitrogen in the future, it may be necessary to provide for alkalinity addition at some point. However, based on present conditions, lime addition will not be required as long as the activated sludge process is designed to achieve nitrification-denitrification.


INFLUENT DISTRIBUTION BOX
GRIT SEPARATION
EXISTING EQUALIZATION BASIN
FLOW DIRECTION (TYP)
EXISTING AERATION TANKS
EXISTING HEADWORKS
WEST MEMPHIS ANIMAL SHELTER
EFFLUENT PUMP STATION
SOUTH LOOP DRIVE
EXISTING ULTRAVIOLET DISINFECTION TO BE REMOVED
EXISTING CLARIFIERS TO BE REMOVED (TYP)
NEW ULTRAVIOLET DISINFECTION
NEW CLARIFIERS
SLUDGE PUMP STATION
FISHER ARNOLD
ENGINEERS | ARCHITECTS | CONSULTANTS | PLANNERS
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WASTE WATER TREATMENT PLANT STUDY
MARCH 10, 2020
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VI. EVALUATION OF ALTERNATIVES AND PLAN SELECTION

In evaluating the options presented in each of the alternatives, the fact that WMU does not have enough land available to build all new aeration basins while continuing to operate their existing aeration system is very important. Due to this lack of land, any option chosen must be effected while keeping the existing treatment facility is operation. A careful consideration of converting these aeration basins to either series operation or to an SBR facility will show that the conversions will not be easily performed. In fact, conversion to an SBR facility will almost certainly require extensive intermediate bypass pumping within functioning basins while one existing basin is taken out of service for conversion. As such, the most viable option seems to be keeping the aeration basins in parallel operation, as they are now, and taking one basin out of service at a time to 1) remove the existing air diffusers and to clean the basin of accumulated grit/sludge, at to retrofit the new Orbal-type rotors into each basin. It is felt that this sequencing affords the greatest degree of practicality in terms of constructability and continued operation.

VII. CONCLUSIONS AND RECOMMENDATIONS

Comparing the biological wastewater treatment alternatives described in detail above, the parallel flow oxidation ditch activated sludge system is the best solution for the City of West Memphis. By using the existing oxidation ditch process with the necessary plant modifications, the city will increase its average daily hydraulic capacity from 6.3 mgd to 12 mgd. The peak day hydraulic capacity will be increased from 7 mgd to 18 mgd, which should eliminate all bypasses of raw wastewater to the receiving stream. The proposed plant renovation will have a capital cost of about $19,350,000 and an annual O&M cost of about $1,350,000 at design loadings. The revised treatment process will be able to consistently satisfy the NPDES effluent requirements established by the Arkansas Department of Environmental Quality.

All cost estimates for plant modifications are very preliminary, but they should be reasonable. Upon final design and approval at all levels and after completion of the bidding process, these modifications should take about 20 months of construction time. The process should be operational by September 2023 and should be meeting the final NPDES effluent limits no later than October 31, 2023.

VIII. OPERATING CONDITIONS AT DESIGN FLOW RATE

With a design flow rate of 12 mgd, the renovated oxidation ditch process will be operating well within its proposed design conditions. Because of this, the activated sludge process will produce outstanding effluent quality. The only parameter that may be of concern is the effluent total phosphorus concentration. It is believed that on-off operation of the aeration system will promote some biological P removal. Preliminary operating criteria (based on design loadings for the year 2040) for the activated sludge system are as follows:

Operating Criteria

BOD_5 to activated sludge	=	160 mg/L
TSS to activated sludge	=	200 mg/L
Volumetric BOD_5 loading	=	19 lb BOD_5/1000 ft^3 / day
Basin MLSS	=	3,700 mg/L
Basin MLVSS	=	2,800 mg/L
Hydraulic residence time	=	12.6 hours
Solids retention time	=	20 days
F/M ratio	=	0.11 lb BOD_5/day/ lb MLVSS
Effluent soluble BOD_5	=	1.5 mg/L
Effluent total BOD_5	=	4 mg/L
Effluent Ammonia-N	=	0.5 mg/L
Effluent Total N	=	9 mg/L
Effluent Total P	=	3 mg/L

Aeration Tank Volume

Design Flow	=	12 mgd (average daily)
Aeration Tank Volume	=	842,000 ft^3 (total)

Activated Sludge Produced

Waste Sludge Produced	=	9,000 lb/day (dry solids)
Waste Sludge Flow Rate	=	0.108 mgd

Aeration Requirement

Oxygen capacity of Orbal aerators	=	28,700 lb/day (12 aerators operating)
Oxygen capacity of Orbal aerators	=	26,300 lb/day (11 aerators operating)
Oxygen capacity of Orbal aerators	=	23,900 lb/day (10 aerators operating)
Oxygen capacity of Orbal aerators	=	21,500 lb/day (9 aerators operating)
Design oxygen requirements (2040)	=	26,200 lb/day (design loadings)
Actual oxygen requirements (2030)	=	18,600 lb/day

Final Clarifier (each of four will be 100 feet diameter and 12 ft deep)

Surface Area (each)	=	7,854 ft^2
Volume (each)	=	0.705 mil gal
Det. time at average flow rate	=	5.6 hours
Det. time at peak flow rate	=	3.8 hours
Surface overflow rate at ave. flow	=	382 gpd/ft^2
Surface overflow rate at peak flow	=	573 gpd/ft^2
Weir loading rate at peak flow	=	15,400 gpd/ft

IX. ARRANGEMENTS FOR PROJECT IMPLEMENTATION

The total project cost estimate is:

Table 9. DETAILED COST ESTIMATE FOR OXIDATION DITCH PROCESS MODIFICATION

Planning Level Opinion of Probable Cost WMU 12 MGD WWTF Expansion West Memphis, Arkansas					
Item	Description	Quantity	Unit	Unit Price	Item Cost
1	HEADWORKS				
1a.	Structure / Construction	1	LS	$ 750,000.00	$ 750,000.00
1b.	Equipment	1	LS	$ 650,000.00	$ 650,000.00
2	EQ BASIN PUMP STATION				
2a.	Structure / Construction	1	LS	$ 500,000.00	$ 500,000.00
2b.	Equipment	1	LS	$ 250,000.00	$ 250,000.00
3	AERATION BASIN CONVERSION				
3a.	Influent Split	1	LS	$ 620,000.00	$ 620,000.00
3b.	Structural Mod. for Orbal Rotors	1	LS	$ 600,000.00	$ 600,000.00
3c.	Equipment	1	LS	$ 2,405,000.00	$ 2,405,000.00
4	CLARIFIERS				
4a.	Structure / Construction	1	LS	$ 4,100,000.00	$ 4,100,000.00
4b.	Equipment	1	LS	$ 1,500,000.00	$ 1,500,000.00
5	RAS/WAS PUMP STATION				
5a.	Structure / Construction	1	LS	$ 680,000.00	$ 680,000.00
5b.	Equipment	1	LS	$ 670,000.00	$ 670,000.00
6	WAS STORAGE TANK/BLOWER				
6a.	Structure / Construction	1	LS	$ 500,000.00	$ 500,000.00
6b.	Equipment	1	LS	$ 150,000.00	$ 150,000.00
7	SEPTIC HAULER WASTE STATION	1	LS	$150,000.00	$ 150,000.00
8	UV DISINFECTION				
8a.	Structure / Construction	1	LS	$ 500,000.00	$ 500,000.00
8b.	Equipment	1	LS	$ 500,000.00	$ 500,000.00
9	SITE IMPROVEMENTS	1	LS	$ 150,000.00	$ 150,000.00
10	MISC. METALS	1	LS	$ 250,000.00	$ 250,000.00
8	PIPING	1	LS	$ 600,000.00	$ 600,000.00
9	ELECTRICAL	1	LS	$ 1,000,000.00	$ 1,000,000.00
10	CONTROLS	1	LS	$ 300,000.00	$ 300,000.00
11	SUBTOTAL				$ 16,825,000.00
12	Engineering, Construction Administration, and Inspection	10%			$ 1,682,500.00
13	Contingency (Subtotal)	15%			$ 2,523,750.00
14	TOTAL				$ 21,031,250.00

The following activities are identified to implement design and construction of the modifications to the West Memphis Wastewater Treatment Plant:

Table 10. Overall Project Schedule

Component	Target Date
Financial analysis by West Memphis Planning Staff	April, 2020
Discussions with financial institution regarding issuing bonds	May, 2020
Finalize fee structure to fund the project	June, 2020
Approval of Board of Aldermen for project go-ahead	July, 2020
Engineer begins preparation of Plans & Specifications	August, 2020
Engineer submits Plans & Specs to ADEQ	March, 2021
ADEQ approval of Plans & Specs	July, 2021
Advertise for Bids on WWTP Construction	September, 2021
Receive Bids	November, 2021
Begin Construction	February, 2022
Complete construction	September, 2023