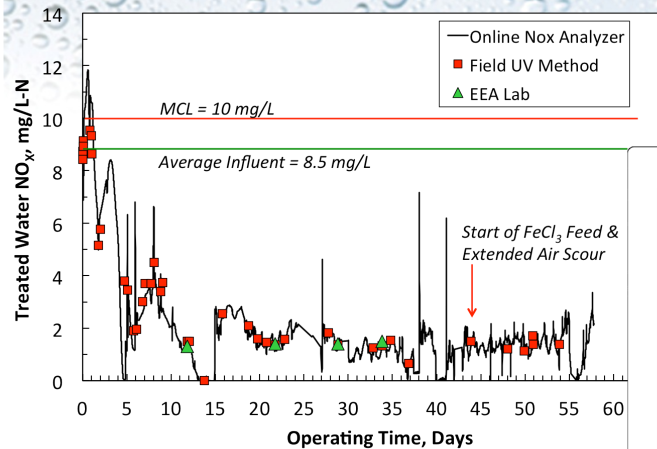

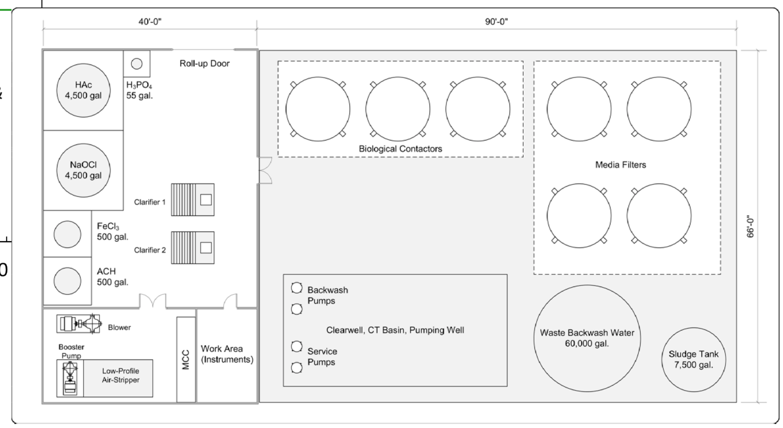

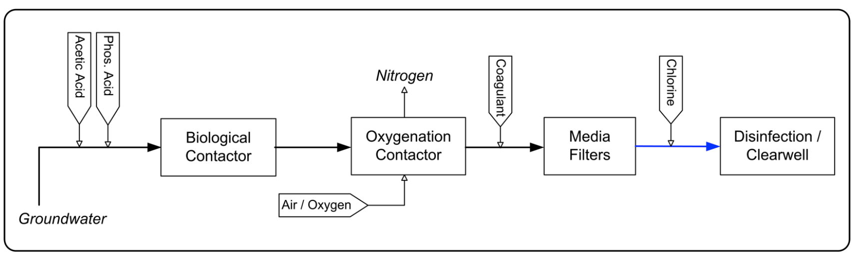

Application of Biological Denitrification for Nitrate Removal From District 37 Groundwater

Technical Report

Prepared for

Los Angeles County Department of Public Works
Water Works Division

WQTS
Water Quality & Treatment Solutions, Inc.
An environmental engineering and science consulting company

June 26, 2013

APPLICATION OF BIOLOGICAL DENITRIFICATION FOR NITRATE REMOVAL FROM DISTRICT 37 GROUNDWATER

TECHNICAL REPORT

PREPARED FOR:

LOS ANGELES COUNTY
DEPARTMENT OF PUBLIC WORKS
WATER WORKS DIVISION

PREPARED BY:

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EXECUTIVE SUMMARY

BACKGROUND

Elevated nitrate levels in groundwater supplies are a significant problem across California and the nation. In most situations, elevate nitrate levels are caused by excess fertilizer applications in agricultural areas. However, it could also be caused by leaching from underground septic systems. Due to its adverse health effects, nitrate is strictly regulated in drinking water, with a drinking water maximum contaminant limit (MCL) of 45 mg/L; also expressed as 10 mg/L as N. The adverse health effects of nitrate are acute. For this reason, exceedance of the nitrate MCL requires a water system to issue a public notification within 24-hrs of becoming aware of such exceedance.

District 37 of the Los Angeles County Department of Public Works (LACDPW) Biological Denitrification (BDN) owns and operates three wells in Acton, California, which contain elevated levels of nitrate ranging from 3.7 mg/L as N to 8.5 mg/L as N. The County currently relies on blending of the three wells as the sole strategy to maintain nitrate below its regulatory limit. The County is interested in evaluating options to removing nitrate from its groundwater supply to improve system independence and reliability.

Conventional nitrate removal technologies include ion-exchange (IX) and reverse osmosis (RO). Both technologies are proven drinking water treatment technologies. Unfortunately, they both generate a high-salinity waste stream that has very limited disposal options. For District 37, there is no viable local disposal option for such a waste stream, which renders both technologies inapplicable for nitrate removal from District 37 wells.

Biological Denitrification (BDN) is a mature technology in wastewater treatment commonly used for the removal of nitrate from wastewater. However, it is an emerging technology for drinking water treatment. The greatest advantage of BDN treatment for District 37 is that it only generates a relatively benign waste backwash water stream that can be discharged into any municipal sewer system. The technology relies on utilizing naturally occurring bacteria in the groundwater to consume nitrate and convert it to nitrogen gas, N_2. The challenge is to engineer the system to achieve nitrate removal as efficiently as possible (i.e., at the lowest cost) while producing water that meets all drinking water regulatory and aesthetic requirements. BDN treatment of drinking water has been used with success in Europe, but has been limited in the US to groundwater remediation projects. However, a 2,000 gpm BDN treatment system is currently been constructed by the East Valley Municipal Water District to remove perchlorate from contaminated groundwater. The system is the same as that used for nitrate removal since the bacteria capable of consuming nitrate are also capable of consuming perchlorate.

In 2012, the County partnered with the Water Research Foundation to co-fund a pilot-testing program aimed at evaluating nitrate removal from District 37 groundwater. The study is ongoing, but sufficient data and information have been gathered to provide the County with an understanding of the configuration and cost of a 1000-gpm BDN treatment system. This report includes a discussion of the pilot testing results to-date, a presentation of the configuration and layout of a 1,000-gpm BDN treatment system, as well as an estimate of its probable capital and annual operations and maintenance costs.

BIOLOGICAL DENITRIFICATION TREATMENT SYSTEM

Figure ES-1 shows a schematic representation of a BDN treatment process train. The treatment system includes four treatment processes: A biological contactor in which nitrate is removed, an aeration contactor in which oxygen is added to the water and excess nitrogen gas is removed from the water, a filtration process to remove particles and bacteria present in the effluent of the biological contactor, and a disinfection process aimed at meeting the minimum disinfection requirements set by the California Department of Public Health (CDPH) for groundwater BDN treatment systems. Acetic acid (vinegar) is added to the water as it enters the treatment plant. The acetic acid is the food source used by the natural bacteria to consume nitrate. A low dose of phosphorus is added as a nutrient to promote efficient bacterial growth. Other chemicals added at the plant include a coagulant to help filter performance, and chlorine to meet the disinfection requirements.

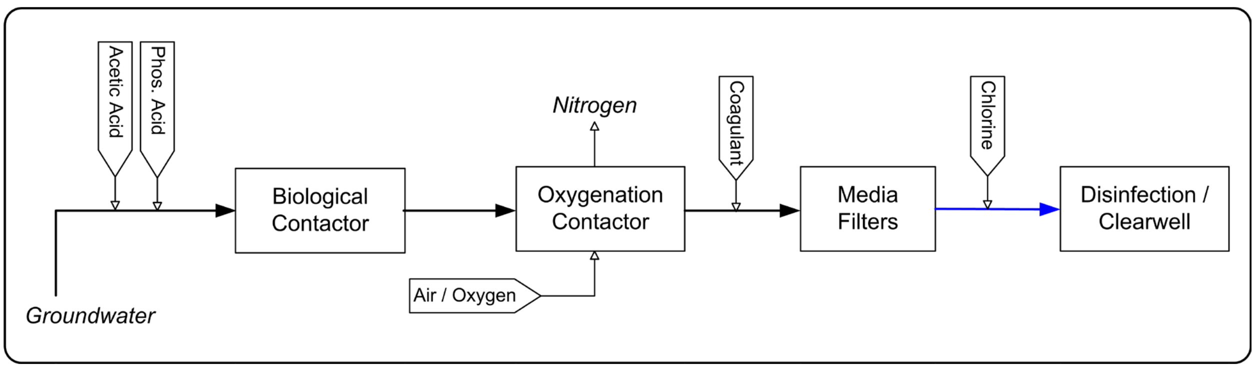

Figure ES-1 – Schematic Representation of a BDN Treatment Process Train

The biological contactor and the filter require frequent backwashing (once every 24 to 48 hrs). While the waste backwash water can be discharged to the sewer, one of the objectives of this pilot testing effort is to evaluate the feasibility of treating the waste backwash water and recovering the majority of it. The results would be a reduction in the amount of water wasted from a range of 5% to 10% to a range of 0.1% to 0.5%.

PILOT TESTING RESULTS

A pilot-scale BDN treatment system was installed inside the chlorine room at Well 37-01. WQTS staff operates the pilot equipment in accordance with a detailed test plan that was implemented at the start of the project. The pilot-scale treatment system mirrors the configuration of a full-scale treatment plant, including washwater treatment and recovery. Data gathering began in mid April 2013 and will continue through mid August 2013. The report includes a detailed discussion of the pilot testing results. An abbreviated version of the analysis is presented herein.

Figure ES-2 shows the profile of nitrate in the treated water from the pilot plant. The average nitrate level in the groundwater was measured at approximately 8.5 mg/L as N. Within five days of startup, nitrate was reduced to less than 4 mg/L as N, and after 10 days of operation, the nitrate reached its target treated-water level of 1 to 2 mg/L as N. Figure ES-3 shows a profile of the concentration of sulfide in the raw, filtered, and chlorinated treated water. Elevated sulfide levels result in objectionable taste-and-odor in the treated water. Since the start of the testing effort, the sulfide level in the treated water has remained at or below the detection limit of 0.01 mg/L.

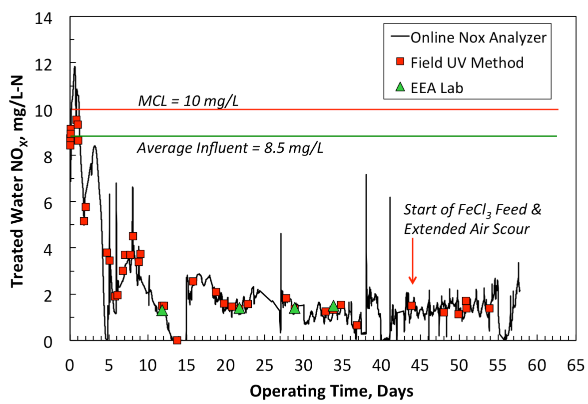

Figure ES-2 – Nitrate+Nitrite Concentration in the Treated Water

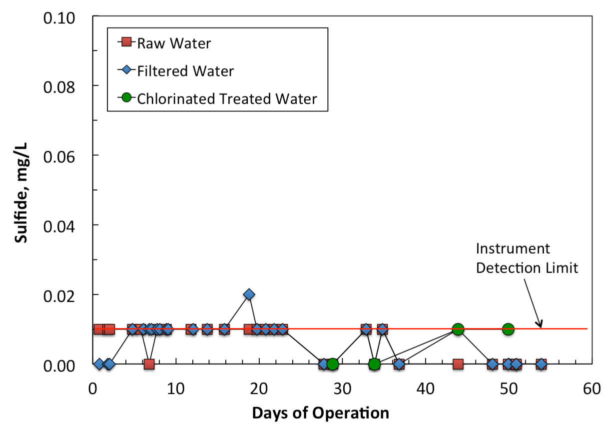

Figure ES-3 – Levels of Sulfide in the Raw, Filtered, and Chlorinated Treated Water

The challenging aspect of a BDN treatment system is maintaining low filtered water turbidity. California DPH has set a criterion of maintaining filtered water turbidity below 0.3 NTU in no less than 95% of all the turbidity values recorded with the online instruments. Under certain operational conditions, the turbidity was maintained below 0.1 NTU throughout the filter run. However, frequent challenges continue to be encountered with turbidity removal requiring further fine-tuning of the operational conditions.

Pilot testing results to-date show that the waste backwash water can be clarified and returned to the head of the treatment plant. With clarification and recovery, the wastage rate from the pilot plant was reduced from 8% without washwater recovery to approximately 0.33% with washwater recovery.

FULL-SCALE SYSTEM CONFIGURATION & PROBABLE COST

Based on the pilot-testing results obtained to-date, WQTS developed general design criteria, layout, and estimates of probable capital and annual cost for a 1,000-gpm BDN treatment system. Details of the system are presented in the report. Figure ES-4 presents a potential layout of the BDN treatment plant, including all process components and chemical feed systems. The plant's footprint is large by groundwater treatment standards. It includes a 40'x66' building that houses the chemical feed systems, aerator, booster pump, air blower, and clarifiers. The plant also includes a 66'x90' uncovered slab area containing seven pressure vessels, one waste washwater tank, on sludge tank, and the clearwell. This amounts to a total of approximately 8,600 ft^2, which yet does not include an area for delivery truck parking and other onsite space requirements.

Table ES-1 summarizes the probable capital cost, probable annual O&M cost, and resulting unit water cost for a 1,000-gpm BDN treatment system that is operated at maximum capacity for 11 months of the year. The probable capital cost for engineering, permitting, construction, and startup support services for the treatment plant is projected to range from $5.8M to $12.5M, with a most probable value of $8.3M. The probable annual O&M cost is projected to range from $345,000/yr to $740,000/yr. If the capital cost is amortized over 20 years at an interest rate of 5%, the total annualized probable cost may range from $0.81M/yr to $1.74M/yr.

If the amortized capital cost is spread over the production volume, the water cost from capital payments would amount to a probable range of $322/AF to $690/AF, with a most probable value of $460/AF. If both capital and annual O&M costs are spread over the production volume, the total unit water cost would amount to a probable range of $560/AF to $1,200/AF, with a most probable value of $800/AF.

Table ES-1 – Summary of Probable Capital and Annual O&M Costs for a 1,000 gpm BDN Treatment System

Item	Probable Range	Most Probable Value
Probable Capital Cost	$5.8M – $12.5M	$8.3M
Probable Annual O&M Cost	$345,000/yr – $740,000/yr	$493,000/yr
Amortized Capital Cost(1)	$466,000/yr – $999,000/yr	$666,000/yr
Total Annualized Cost	$0.81M/yr – $1.74M/yr	$1.1M/yr
Water Cost(2)		
Water Cost from Capital Payments	$322/AF – $690/AF	$460/AF
Water Cost from Annual O&M Cost	$238/AF – $510/AF	$340/AF
Totalized Water Cost	$560/AF – $1,200/AF	$800/AF

(1) Assuming 20 yr payment period at 5% interest rate
(2) Based on a water production rate of 1,443 AF/yr

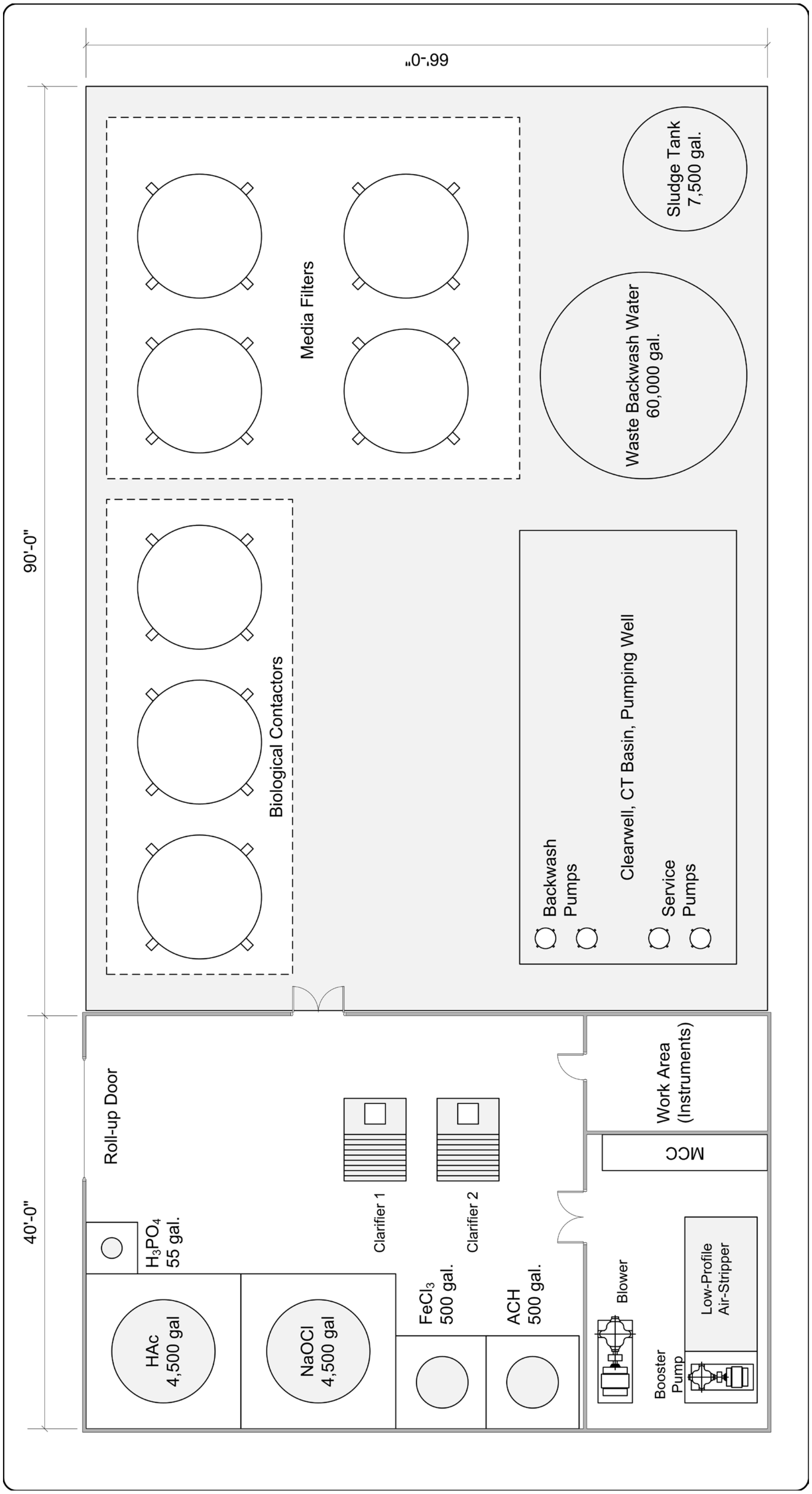

Figure ES-1 – Possible Layout of a 1,000 gpm BDN Treatment System

SECTION 1.0 – INTRODUCTION

Many groundwater sources throughout North America have nitrate levels near or above the maximum contaminant level (MCL) of 10 mg-N/L. The health concern over the presence of nitrate in water is related to its reduction to nitrite in the digestive system. When absorbed into the bloodstream, nitrite hinders the blood's ability to transport oxygen to the cells. Infants are most susceptible to nitrite poisoning because their digestive systems contain high levels of bacteria that convert nitrate to nitrite. In a major national survey of approximately 2,100 domestic wells conducted by the United States Geological Survey (USGS), nitrate was measured at levels above the MCL in 4.4% of wells sampled (DeSimone, 2009). The study observed that nitrate levels are highest in agricultural areas, primarily due to fertilizer application.

Nitrate is an integral component of the natural nitrogen cycle in which organic nitrogen present in soils is degraded to ammonia and then to nitrate by naturally occurring bacteria. Most of the nitrate generated is then taken up by new plant growth. However, excess nitrate in the soil can leach into groundwater. Human activities can increase the nitrate level in the soil, and thus increase nitrate leaching into groundwater. Such activities include fertilizer application, both on crops and residential lawns, animal waste from concentrated animal facilities (e.g., dairies), as well as improperly maintained septic systems.

1.1 NITRATE IN LA COUNTY DEPARTMENT OF PUBLIC WORKS' DISTRICT 37 WELLS

Los Angeles County Department of Public Works (County) operates several groundwater systems within unincorporated portions of Los Angeles County. Some of these groundwater basins contain nitrate close to or above the primary MCL of 10 mg/L as N. These include District 37 Wells 37-01, 37-03, and 37-04, located in Acton. Figure 1 shows profiles of nitrate concentrations in all three wells measured between 2008 and 2011. Over the reporting period, Well 37-01 had the highest nitrate concentration ranging from 6.5 mg/L as N to as high as 9.9 mg/L as N. Well 37-03 had the lowest nitrate concentration ranging between 2 and 4 mg/L as N, while Well 37-04 contained nitrate between 4 and 7 mg/L as N. Average values of general water quality parameters in all three wells measured in 2010 and 2011 are listed in Table 1. The County currently relies on blending between the three wells to maintain nitrate at a comfortable level below the drinking water limit in the water served to its customers. In order to maximize the use of its groundwater sources, the County can benefit from reducing its reliance on blending. In addition, the County is interested in minimizing the overall level of nitrate in its drinking water. This requires the construction and operation of water treatment systems that can reliably remove nitrate from groundwater.

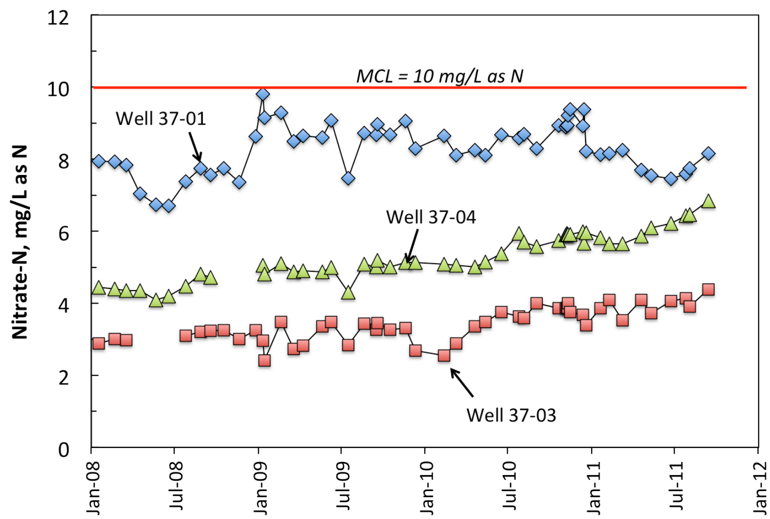

Figure 1 – Nitrate Concentrations in Wells 37-01, 37-03, and 37-04 (2008 – 2011)

Table 1 – General Water Quality of District 37 Wells (2010 – 2011)

Parameter	Unit	MCL or Goal	Well 37-01	Well 37-03	Well 37-04
pH	--	6.5 – 8.5	7.5	7.3	7.2
Nitrate-N	mg/L as N	10	8.5	3.7	5.8
Alkalinity	mg/L as $CaCO_3$	--	179	137	154
Calcium	mg/L	--	96	64	68
Magnesium	mg/L	--	26	14.6	12.6
Hardness, Total	mg/L as $CaCO_3$	--	345	220	222
Turbidity	NTU	--	0.30	0.44	0.16
TDS	mg/L	1,000	552	324	424
Specific Conductance	µS	--	852	478	722
Sodium	mg/L	--	62	24	36
Chloride	mg/L	--	100	34	70
Sulfate	mg/L	--	72	46	64
Arsenic	µg/L	10	2.8	2.1	2.4
Barium	µg/L	--	171	112	131
Fluoride	mg/L	2.0	0.34	0.27	0.23

Physical/chemical nitrate removal technologies are well developed and are currently in use for nitrate removal. These technologies include ion exchange (IX) and reverse osmosis (RO). While these technologies are reliable, they generate a high-salt waste stream that cannot be disposed in a sanitary sewer. The one technology that does not generate a high-salinity waste stream is biological denitrification (BDN), which makes its application at District 37 wells plausible. Biological denitrification relies on naturally occurring bacteria to convert nitrate nitrogen (NO_3-N) to nitrogen gas (N_2). Since this reaction is part of the natural nitrogen cycle, denitrifying bacteria are ubiquitous in the natural environment. Under environmental denitrification conditions, the bacteria utilize naturally occurring organic carbon to slowly degrade nitrate. In engineered systems, such as a BDN treatment system, external sources of carbon and nutrients are added to promote the growth of these natural bacteria and expedite the process of nitrate consumption.

The possibility of using the BDN treatment system to remove nitrate from District 37 groundwater prompted the County to enter into an agreement with the Water Research Foundation (WaterRF) in 2012 to co-fund a pilot-testing program to evaluate the applicability of a BDN treatment system for nitrate removal from District 37 groundwater. The pilot-testing program is being performed by Water Quality & Treatment Solutions, Inc. (WQTS).

1.2 BIOLOGICAL DENITRIFICATION

BDN treatment is not new to the environmental engineering field. It is widely applied in wastewater treatment as a means of removing inorganic nitrogen from secondary-treated wastewater before the water is returned to the environment. As a drinking water treatment technology, BDN has been implemented at numerous full-scale plants in Europe (Roennefahrt, 1986; Bockle et al., 1986; Soares, 2000). Dördelmann (2009) presented data on four denitrification plants in Germany, Austria, and Poland, with capacities ranging from 760 gpm to 7,000 gpm. Different plants had different configurations of the biological reactor (i.e., fixed bed vs. fluidized bed), and one plant used acetic acid while others used ethanol as the carbon source. The plant in Austria was installed in 1997 and remains in operation. Two similar plants were also constructed in Italy (IWA, 2007), one with a capacity of 220 gpm, which began operation in 1997, and another with a capacity of 2,400 gpm that began operation in 2004. These plants also utilize acetic acid as the carbon source. The larger plant includes three parallel treatment trains and treats groundwater containing approximately 18 mg/L of nitrate-N to a treated water concentration of approximately 1.1 mg/L as N. A number of other biological denitrification plants have been constructed in France (Richard, 1989) and Belgium (Liessens et al., 1993).

In spite of the significant experience with biological denitrification in Europe, the technology has not achieved popularity in the US. One US system was installed in Coyle, OK in 2000, but has since been shut down (Oklahoma DEQ, 2007). The US resistance to biological drinking water treatment stems from concerns over the potential introduction of unknown pathogens into the drinking water supply. It is also true that the US regulatory philosophy considers the presence of any bacteria to be a sign of insufficient disinfection. As a result, biological dentirification was not seriously considered in the US for decades. Nonetheless, a number of studies have been conducted to evaluate various biological denitrification technologies for nitrate removal from groundwater. A recent example is a pilot-scale study funded by the Water Research Foundation in Glendale, AZ that compared autotrophic and heterotrophic biological denitrification for nitrate removal from groundwater (Meyer et al., 2010). The autotrophic process utilized hydrogen gas as the energy source, while the heterotrophic process used ethanol as the energy and carbon sources. Both processes removed nitrate from an influent of 12 mg/L as N to an effluent of <0.5 mg/L as N. The heterotrophic process was configured to operate in an upflow configuration with plastic media for bacterial support.

In early 2012, WQTS completed a pilot-scale study of biological denitrification for the City of Glendale, California. The study showed that a downflow fixed-bed process followed by oxygenation and media filtration reliably achieved sufficient nitrate removal and produced water that met all the regulatory requirements for drinking water. Acetic acid was used as the energy and carbon source.

In addition, since the discovery of perchlorate contamination in several US groundwaters, there has been a resurgence in the interest in biological denitrification because of its ability to also remove perchlorate. Over the last 10 years, a large number of biological denitrification bench-scale and pilot-scale studies have been conducted, most of which were focused on perchlorate removal (e.g., Nerenberg et al., 2002; Evans & Logan, 2004; Logan et al., 2004; Adham et al., 2004; Brown et al., 2008; Najm et al., 2009; Webster et al., 2009). These projects evaluated various configurations of biological systems including upflow fluidized bed, upflow fixed bed, downflow fixed bed, and membrane-based systems. The focus of these projects was on demonstrating performace of the systems for perchlorate removal and identifying optimum design and operational conditions. One project resulted in the construction of the first full-scale drinking water biological denitrification plant in California designed for perchlorate removal from groundwater. This 2,000 gpm plant, which is owned and operated by the East Valley Municipal Water District, will utilize a proprietary upflow fluidized-bed biological reactor followed by flocculation, sedimentation, and media filtration. Start-up is planned for the summer of 2013.

SECTION 2.0 – BIOLOGICAL DENITRIFICATION FOR GROUNDWATER TREATMENT

2.1 FUNDAMENTALS OF BIOLOGICAL DENITRIFICATION

This section presents a brief discussion of the fundamentals of biological denitrification as it applies to drinking water treatment. From a general perspective, biological denitrification is a bacterially-mediated process in which nitrogen present as nitrate (NO_3^-) or nitrite (NO_2^-) is reduced to nitrogen gas (N_2). Denitrifying bacteria can be heterotrophic (i.e., utilize organic carbon as their carbon source) or autotrophic (i.e., utilize inorganic carbon as their carbon source). The denitrification reaction occurs as an electron transfer process in which electrons are extracted from an *electron donor* and taken up by nitrate or nitrite as the *electron acceptor*. Denitrifying bacteria are *facultative aerobic* bacteria, which means that they can degrade nitrate when oxygen becomes limiting. In fact, dissolved oxygen inhibits the ability of denitrifying bacteria to degrade nitrate. For this reason, denitrifying bacteria will first utilize oxygen as the electron acceptor, and only when the dissolved oxygen is below a certain threshold can they shift to utilizing nitrate as the electron acceptor.

There are three primary electron donors used in engineered biological denitrification systems. They include two organic electron donors: methanol (CH_3OH) and acetic acid (CH_3COOH), as well as one inorganic electron donor: hydrogen gas (H_2). Engineered systems for the utilization of hydrogen gas as an electron donor are still in the development stage, and thus were not considered for District 37 groundwater treatment. Methanol is commonly used in wastewater denitrification because of its wide availability and ease of use. However, due to its toxicity to humans, it is not used in drinking water denitrification systems. Acetic acid (or sodium acetate) has emerged as the more common electron donor for drinking water denitrification applications for a number of reasons: First, there are no health concerns associated with acetate as there are with methanol. Second, there are no safety concerns associated with its handling and use as there are with hydrogen gas. Third, acetic acid or sodium acetate is already available with ANSI/NSF60 certification for drinking water treatment. For these reasons, acetic acid was used as the electron donor for District 37 groundwater denitrification treatment evaluation.

There are many fundamental biochemical reactions that take place during the process of biological denitrification. Rittmann & McCarty (2001) present an excellent discussion of the fundamentals of biological denitrifying reactions and the basis behind them. Under typical biological denitrification treatment conditions, the total reaction for the reduction of oxygen with denitrifying bacteria is as follows:

$$0.16457\, O_2 + 0.0122\, NO_3^- + \ 0.125\, CH_3COOH \\ \rightarrow 0.0640\, CO_2 + 0.1250\, HCO_3^- + 0.1128\, H^+ + 0.0122\, C_5H_7O_2N + 0.0884\, H_2O$$

while the total reaction for the reduction of nitrate with denitrifying bacteria is as follows:

$$0.14386\ NO_3^- + 0.01886\ H^+ + 0.125\ CH_3COOH \rightarrow 0.0640\ CO_2 + 0.1250\ HCO_3^- + 0.0122\ C_5H_7O_2N + 0.06583\ N_2 + 0.1542\ H_2O$$

Based on the above two reactions, Table 2 summarizes the stoichiometric demand of acetic acid by denitrification reactions. The table shows that 1.42 mg of acetic acid is required for every mg of oxygen to be consumed, and 3.72 mg of acetic acid is required for every mg of nitrate-nitrogen to be consumed.

Table 2 – Summary of Theoretical Demand of Acetate for Denitrification Reactions

Reaction	Parameter	Value
Consumption of Oxygen	mass of acetic acid required per mass of oxygen consumed, HAc:O_2	1.42 mg/mg
Consumption of Nitrate	mass of acetic acid required per mass of nitrate consumed, HAc:NO_3-N	3.72 mg/mg

The following is an example of how the factors in Table 2 can be used to estimate the acetic acid dose required under any combination of oxygen and nitrate concentrations.

If a groundwater contains 7 mg/L of dissolved oxygen and 9 mg/L of nitrate-N, and approximately 8 mg/L of the nitrate-N are to be removed, the acetic acid dose required is calculated using the factors in Table 2 as follows:

1. The acetic acid dose required to completely degrade the oxygen is $7 \times 1.42 \approx 10$ mg/L.
2. The acetic acid dose required to degrade the specified amount of nitrate is $8 \times 3.72 \approx 30$ mg/L.
3. The total acetic acid dose required is calculated as: 10 + 30 = **40 mg/L**.

A similar approach can be used for estimating the theoretical acetic acid dose under any combination of oxygen and nitrate concentrations. It is important to note that complete oxygen consumption must be attained before nitrate consumption can be achieved.

Finally, bacteria require phosphorus as a nutrient for cell growth. Since most groundwaters do not contain sufficient levels of phosphorus, it needs to be added to the water, typically as phosphoric acid (H_3PO_4). Literature suggests that the theoretical phosphorus requirement of denitrifying bacteria is approximately 0.022 mg PO_4-P per mg of biomass generated (deBarbadillo et al, 2006). In the above example, the amount of biomass generated is estimated at approximately 9 mg/L, and therefore the phosphorus dose required is estimated at 9 mg/L × 0.022 = 0.2 mg/L as P.

2.2 CONFIGURATION OF A GROUNDWATER BDN TREATMENT SYSTEM

Figure 2 shows a line schematic of a groundwater BDN treatment system. The main process train includes four unit processes: (1) a biological contactor, (2) an oxygenation contactor, (3) a media filter, and (4) a disinfection contactor. Other components include a backwash system and chemical feed systems. The backwash system includes a backwash water supply tank and a waste backwash water tank to capture the waste backwash water from the media filter and the biological contactor. A total of four chemical feed systems are required for the addition of acetic acid, phosphoric acid, coagulant, and chlorine. This section discusses the role of each component of this treatment system, as well as its general engineering configuration.

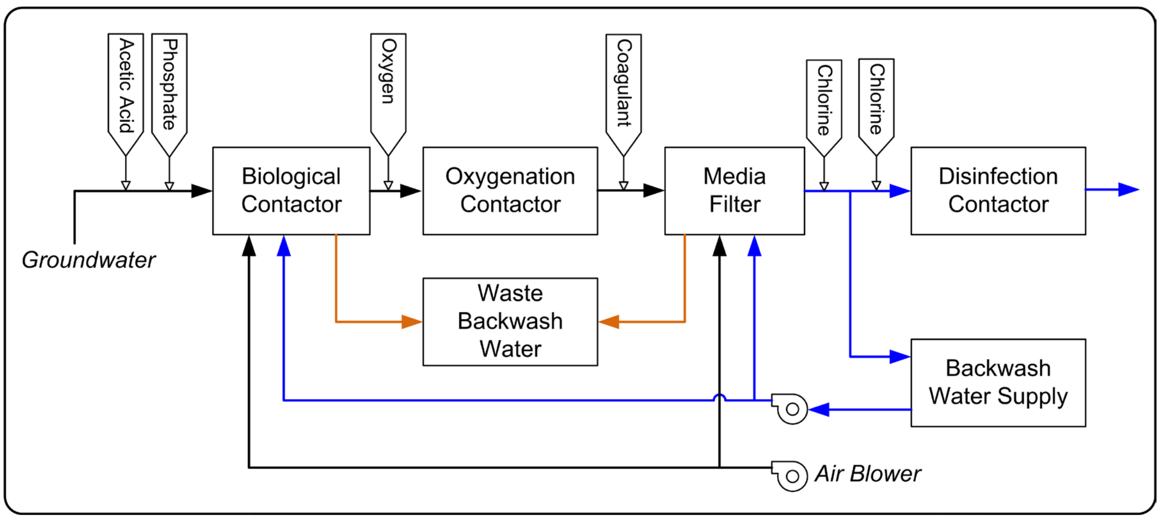

Figure 2 – Line Schematic of a Biological Denitrification (BDN) Treatment System

Biological Contactor

After dosing the water with acetic acid and phosphoric acid, the water enters the biological contactor. The contactor is designed to maximize the concentration of biomass per unit volume of contactor so as to improve process efficiency and reduce its size. This is best achieved with a fixed film contactor where natural bacteria are cultivated on granular media with a high specific surface area (i.e., high surface area per unit volume). The higher the surface area of the media on which bacteria can grow, the higher is the process efficiency and the smaller is the required total process volume and footprint. Typical media used include sand, anthracite, or granular activated carbon (GAC). Since GAC has a much higher specific area compared to sand or anthracite, it is the preferred media in biological contactors. GAC also offers the benefit of an irregular surface with many crevices in which bacteria can protect themselves from the shear effects of backwashing.

The key design parameter for the biological contactor is the Empty Bed Contact Time (EBCT). The higher the EBCT, the larger is the treatment system required. Based on studies conducted by WQTS and others, an EBCT of 10 minutes is sufficient for an acetate-based biological contactor used for the denitrification of groundwater.

There are two typical configurations of the biological contactor in the BDN process: 1) a downflow packed-bed configuration, or 2) an upflow fluidized-bed configuration. Figure 3 shows a schematic comparison between the two configurations. In a downflow configuration, the water enters the contactor from top, flows through the packed bed of media, and exits from the bottom. At a certain frequency, and depending on the system needs, the contactor is taken out of service and backwashed using a clean water source. A backwash pump is used to push water up through the media at a high rate. As the media fluidizes, inert particles trapped in the media, as well as dead biomass, are washed out of the contactor. The waste backwash water is then either treated on site or discharged to the sewer untreated.

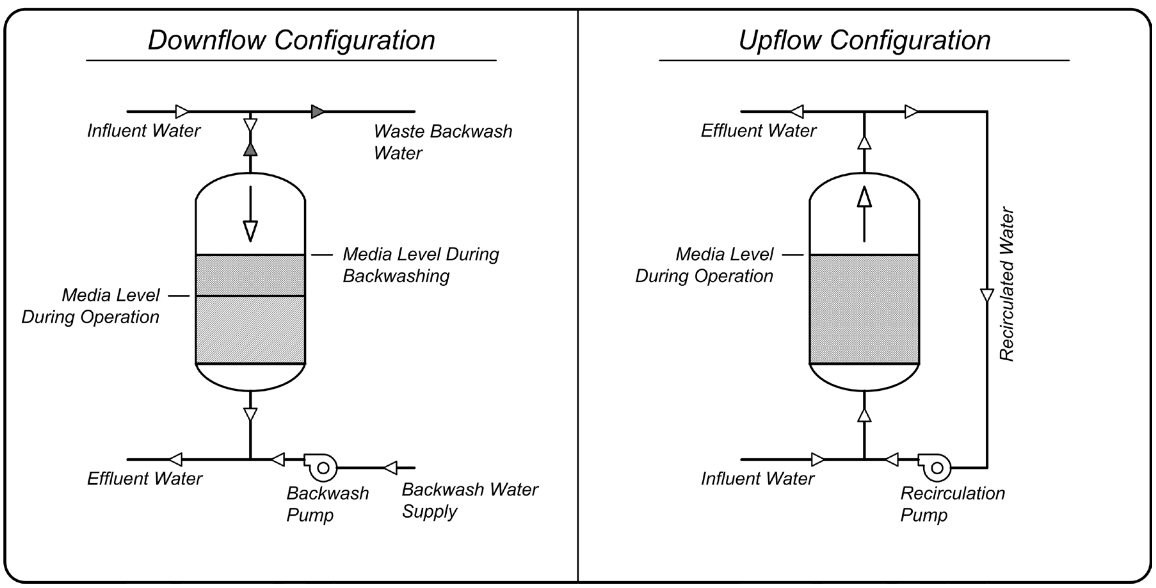

Figure 3 – Schematic Illustrations of a Downflow Configuration and an Upflow Configuration of the Biological Contactor

In an upflow configuration, the water enters the contactor from the bottom, flows up through the media, and then exits the contactor from the top. A recirculation pump is used to draw water from the top of the contactor and blend it back with the contactor influent on a continuous basis. The recirculation rate is selected to be high enough to maintain fluidization of the media. In this configuration, inert material is not trapped in the bed, and most of the dead biomass is removed from the contactor on a continuous basis. Therefore, the contactor does not need to be taken out of service for backwashing. While this appears to favor the fluidized-bed configuration over the packed-bed configuration, there are several disadvantages to the fluidized-bed configuration:

1. The process relies on the continuous operation of a recirculation pump. Not only does this consume additional energy, it reduces the reliability of the system by introducing a critical mechanical component. Failure of the recirculation pump will force the shutdown of the system.
2. The recirculation system results in significant mixing of the bed. Since the kinetics of the reactions are proportional to the concentrations of biomass, nitrate, and acetate, the efficiency of a continuously mixed process is expected to be lower than that of a plug-flow process (i.e., packed-bed configuration).
3. In the fluidized-bed configuration, all the sloughed biomass is carried over to the downstream process. This means that the downstream filter is burdened with the entire solids load generated. Work by Webster et al. (2009) showed that a clarifier is required upstream of the filter in order to reduce the solids load on the media filter. However, in a packed-bed configuration, only part of the biomass that sloughs off the media exits the contactor and the rest is retained in the contactor. This allows the media to filter to operate reliably without the need for a clarifier.

For the above reasons, WQTS recommends the use of a downflow packed-bed configuration instead of the upflow fluidized-bed configuration.

Oxygenation

Oxygen and nitrate are consumed as the water passes through the biological contactor, resulting in very low oxygen levels in the denitrified water. The water should then be oxygenated before it is filtered. The oxygenation process is needed for a number of reasons:

First, since dissolved oxygen levels impact metal corrosion in the distribution system, it is important that the oxygen level in the water be restored to that experienced by the distribution system. This may range from 6 to 9 mg/L.

Second, replenishing the oxygen level in the water allows the downstream media filter to achieve aerobic biodegradation of any residual acetate that may be present in the effluent of the biological contactor, such that the finished water contains little to no additional biodegradable organic carbon compared to the untreated raw water.

Third, with the conversion of nitrate to nitrogen gas in the biological contactor, the effluent will be supersaturated with nitrogen gas. This poses a challenge for the downstream media filter as the excess nitrogen gas may cause air-binding in the filter, resulting in increased headloss and early breakthrough of turbidity. If oxygenation is implemented by aerating the water in an open vessel, the excess nitrogen can be removed from the water upstream of filtration.

One approach to oxygenating the water while simultaneously removing the excess nitrogen gas is to utilize a bubble oxygenation process. In this approach, the biologically treated water enters the top of a contactor that is open to the atmosphere. Pure oxygen or air is bubbled through the water column in a counter-current mode. The water exiting the contactor from the bottom will have sufficient oxygen and will not be supersaturated with nitrogen gas. The downside to this approach is that re-pumping is necessary, which requires a dedicated booster pump system and increases the energy cost of the treatment system.

Media Filtration

After biological treatment and oxygenation, the water will undergo filtration, which has two objectives: The first is to capture biomass that may slough off the biological contactor and prevent it from entering the distribution system. The second objective is to serve as an aerobic biological barrier to remove any excess acetate that may be present in the effluent of the biological contactor. The California Department of Public Health (CDPH) requires that media filtration in a BDN treatment system maintain a treated-water turbidity less than 0.3 NTU in 95% of the samples collected. This criterion was adapted from surface water treatment requirements, and merely serves as an indicator of overall filter performance. A coagulant and/or a coagulant aid polymer may be added to the influent of the filtration process to aid in particle removal and extend filter runtime. Once turbidity breakthrough takes place, or the filter headloss exceeds the terminal headloss, the filter is backwashed with treated water and put back in service.

Disinfection Contactor

The CDPH requires BDN treatment systems to maintain a minimum 4-log virus inactivation with a disinfectant. With chlorine as the disinfectant, the CT required for 4-log virus inactivation is a function of water temperature, and is outlined in Table 3 below for pH values between 6 and 9 (USEPA, 1991). As an example, if a system maintains a residual of 1 mg/L in the effluent of the disinfection contactor, and the minimum water temperature is 10 °C, then the minimum required T_{10} through the contactor is 6 minutes.

Table 3 – Disinfection CT Values for 4-log Virus Inactivation with Chlorine (pH 6 - 9)

Temperature, °C	Chlorine CT for 4-log virus Inactivation, mg-min/L
5	8
10	6
15	4
20	3
25	2

The CT values listed in Table 3 will require a dedicated disinfection contactor located after the filters. This contactor can serve the dual purpose of disinfection and treated water wet-well for the treated water pumps.

Backwash Water System

A backwash water system is required for backwashing the biological contactor and the granular media filter. WQTS experience suggests that backwashing both unit processes with chlorinated water helps control bacterial growth without hindering the performance of the biological contactor. As shown in Figure 2, chlorine is added to the filtered water immediately before and after water is diverted to the backwash water supply tank. Alternatively, an inline clearwell can be used as the backwash water supply tank.

A backwash water pump is used to convey water from the backwash water supply tank to the media filter and the biological contactor. Based on WQTS' experience, due to the high amount of biomass generated, both in the biological contactor and in the media filter, air scour is incorporated into the backwash cycle to improve backwash efficiency. As shown in Figure 2, an air blower is used to deliver air to each unit process during backwashing.

The waste backwash water volume could constitute between 5% and 10% of the volume of water treated depending on the backwash frequency required to maintain proper operation of the BDN process and the media filter. For groundwater wells located away from a sewer system, this is a large volume that is difficult to dispose of. Recovery of the waste backwash water could greatly reduce the burden of disposing of the waste backwash water.

SECTION 3.0 – PILOT TESTING

As noted in Section 1, the County and the Water Research Foundation are co-funding a pilot-scale evaluation of BDN treatment to remove nitrate from District 37 groundwater. As of the date of preparation of this report (June 30, 2013), the testing is ongoing and will be completed by the middle of August 2013. However, sufficient data and information have been collected to date to allow for the development of this report. This section describes the pilot testing program, and presents its results to date.

3.1 PILOT-PLANT CONFIGURATION AND OPERATION

The pilot testing equipment was set up at Well 37-01 located at 3318 W. Soledad Canyon Road in Acton. This site was ideal for several reasons: First, Well 37-01 has the highest nitrate level compared to the other District wells. Second, the water from the other two wells comes to the Well 37-01 site and blends with the water from Well 37-01 water before the blend is chlorinated. This meant that tapping into the discharge pipe from Well 37-01 to feed the pilot plant would ensure that the pilot-plant can still receive water from the other wells if Well 37-01 had to be shut down during the pilot testing period. Third, the site is fenced and secured, yet provides WQTS with easy access to the pilot equipment. Fourth, the site has an empty room capable of housing all the pilot equipment and thus protect it from the elements, and from vandalism.

Figure 4 shows a schematic outline of the pilot-scale treatment process train, while Tables 4 and 5 list the design and operational parameters of the pilot plant unit processes. The water is drawn from the discharge pipe of Well 37-01 at a flowrate of approximately 2.1 gpm, and directed into the pilot plant building. The influent water is dosed with acetic acid and phosphoric acid before it goes through the biological removal process. This process includes two pressure filters containing 48 inches of Granular Activated Carbon (GAC).

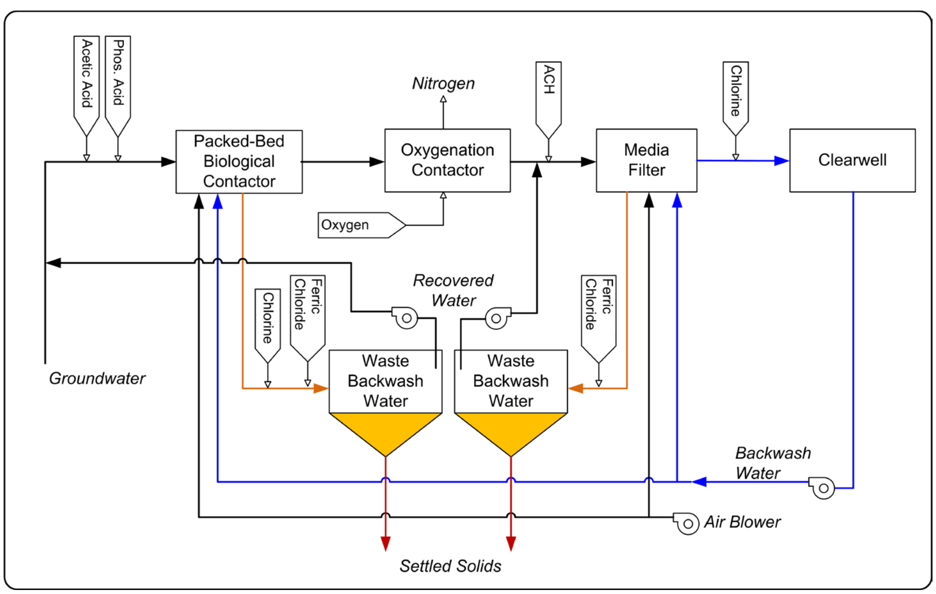

Figure 4 – Schematic Outline of the Pilot-Scale BDN Treatment Plant Implemented at Well 37-01

Table 4 – Design and Operational Criteria of the Pilot-Scale BDN and Oxygenation Processes

Parameter	Unit	Value
Influent Flowrate	gpm	2.1
Acetic Acid Dose Control	--	Automatic Control (Target NOx = 1 to 2 mg/L)(1)
Acetic Acid Dose	mg/L	35 – 40
Phosphoric Acid Dose	mg/L as P	0.2
BDN Maximum Runtime	hrs	24
BDN Empty Bed Contact Time (EBCT)	minutes	10
Oxygen Dose Control	--	Target of 6 to 7 mg/L DO in the Media Filter Influent Water
Ferric Chloride Dose to Waste Backwash Water	mg/L	75
Chlorine Dose to Waste Backwash Water	mg/L	10
Waste Backwash Water Tank Settling Period	minutes	60 to 120
Return Flow of Clarified Waste Backwash Water	gpm	0.21

(1) NOx refers to the sum of nitrate-N (NO_3-N) and nitrite-N (NO_2-N)

Table 5 – Design and Operational Criteria of the Pilot-Scale Media Filtration Process

Parameter	Unit	Value
Influent Flowrate	gpm	1.0
Filtration Rate	gpm/sf	3.0
Coagulant Type	--	Aluminum Chlorohydrate
Coagulant Dose	mg/L	15 to 30
Filter Maximum Runtime	hrs	24
Maximum Filtered Water Turbidity	NTU	0.3
Chlorine Dose to Filter Effluent	mg/L	2 to 4 mg/L (Target 0.5 mg/L in Clearwell)
Ferric Chloride Dose to Waste Backwash Water	mg/L	50
Waste Backwash Water Settling Time	minutes	60 to 120
Return Flowrate from Waste Backwash Water Tank	gpm	0.15

The phosphoric acid dose is set at 0.2 mg/L as P. The automatic control system on the pilot plant is set to modulate the acetic acid dose with the goal of maintaining the nitrate concentration in the treated water between 1 and 2 mg/L as N. This setting results in an acetic acid dose between 35 mg/L and 40 mg/L.

After the BDN process, the water flows through the oxygenation process in which oxygen is added to the water and excess nitrogen gas generated in the BDN process is released from the water. Pure oxygen is used at the pilot plant, but compressed air can be used in the full-scale system to achieve the same objectives. The small size of the aeration process at the pilot plant makes it necessary to use pure oxygen compared to compressed air.

After the aeration process, the water is treated through a dual-media filter containing anthracite and sand. The purpose of the filter is to remove bacterial cells that may slough off the upstream BDN process. A coagulant is added upstream of the filter to help improve its performance. The coagulant used during the pilot testing is an aluminum chlorohydrate (ACH) coagulant labeled SWT 8808A provided by Sterling Water Technologies (Columbia, TN). The ACH dose is typically 15 mg/L. However, at times when the filtered water turbidity increases, the ACH dose is increased to 15 and 30 mg/L to improve filter performance. Chlorine is added to the filter effluent before the water enters the clearwell. The chlorine dose is set to maintain a chlorine residual of approximately 0.5 mg/L in the clearwell. The typical chlorine dose ranges from 2 to 4 mg/L.

Both the BDN process and the media filter require backwashing at a specific frequency. The time between backwashes can vary greatly depending on the amount of bacterial growth achieved, which in turn is driven by the amount of nitrate being removed and amount of acetic acid added. During pilot testing, the runtime for each of the BDN process and the media filter is set at 24 hrs between backwashes.

The treated water clearwell serves as the source of backwash water for both the BDN process and the media filter. The waste backwash water from each process is collected into a waste backwash water tank. A coagulant (ferric chloride, $FeCl_3$) is added to the waste backwash water from each process before it enters its waste backwash water tank. The $FeCl_3$ dose to each waste backwash water was selected based on the results of jar testing. For the BDN process, the $FeCl_3$ dose added to the waste backwash water is 75 mg/L, while the dose added to the waste backwash water from the media filter is 50 mg/L. The coagulated water is allowed to settle for 1 to 2 hrs before the clarified water is returned to the influent of each process, while the settled solids are allowed to accumulate at the bottom of the tanks. A dose of 10 mg/L chlorine is added to the waste backwash water from the BDN process. The purpose of chlorine addition at this location is simply to improve the aesthetic quality of the waste backwash water.

3.2 PILOT TESTING RESULTS TO-DATE

As of the time of preparation of this report, the pilot plant has been in operation for approximately 60 days. The results obtained to date are presented in this section. Table 6 lists the general water quality parameters measured in Well 37-01 water during this period. The nitrate level in the water has averaged 8.3 mg/L as N with no detectable nitrite. The dissolved oxygen (DO) level has averaged 8.2 mg/L. The alkalinity and pH of the water have averaged 170 mg/L as $CaCO_3$ and 7.5, respectively.

Table 6 – General Water Quality of Well 37-01 Water Measured During Pilot Testing To Date

Parameter	Unit	Value
NO_3-N	mg/L	8.5
NO_2-N	mg/L	<0.1
DO	mg/L	8.2
Alkalinity	mg/L as $CaCO_3$	170
pH	--	7.5
TOC	mg/L	0.51
Temperature	°C	20
Total Cr	µg/L	1.9
Cr(VI)	µg/L	1.6
Total Coliforms	MPN/100mL	<1
HPC Bacteria	CFU/mL	496*

* Measured at pilot plant influent tap, not the well tap

Start-up of a BDN treatment system is a critical component of pilot testing. The reason is that the biological activity takes several days or weeks to reach full capacity. During this period, the treated water quality goals are not necessarily met. Therefore, measures are taken to jump-start the biological process as quickly as possible. These measures include: 1) starting the system at low flow in order to increase the contact time and improve the chances of the natural bacteria to colonize the GAC surface, and 2) starting with a high acetic acid dose to provide the bacteria with an unlimited amount of "food" to grow. Once nitrate removal begins, the acetic acid dose can be reduced and the flowrate can be increased. Figure 5 shows the acetic acid dose and flowrate through the pilot plant during this study. Figure 6 shows concentrations of nitrate and nitrite in the treated water. At the start of the study, the acetic acid dose was set at 2 times the theoretical dose required for full removal of the nitrate present in the water. This amounted to approximately 120 mg/L. The flowrate was set at 0.4 gpm, which is 20% of the target flowrate of 2.1 gpm. Within the first five days, the nitrate concentration in the treated water had decreased to less than 4 mg/L as N. At that time, the acetic acid dose was reduced to approximately 35 mg/L. By the 10th day, full nitrate removal was achieved. At that time, the flowrate was gradually increased to 0.8 gpm, and ultimately to 2.1 gpm by the 15th day of operation. From that day on, the treatment system met the treated water nitrate concentration target of 1 to 2 mg/L as N as shown in Figure 6.

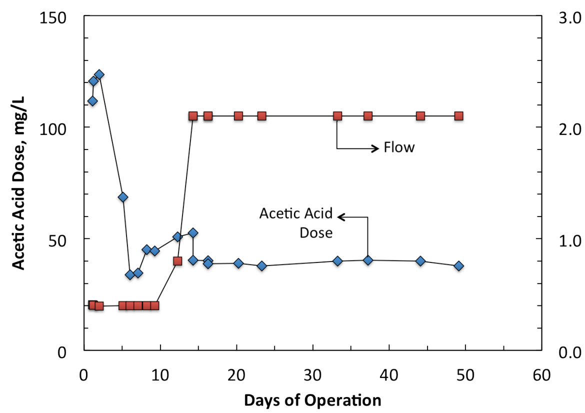

Figure 5 –Acetic Acid Dose and Pilot Plant Flowrate

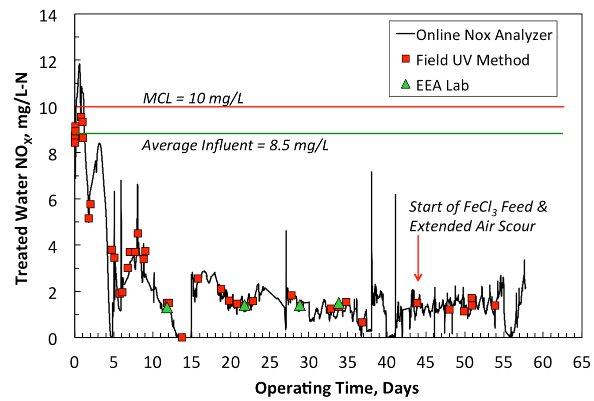

Figure 6 –Nitrate+Nitrite Concentration Throughout the Pilot Study

Figure 6 also shows a comparison between three methods used to analyze for nitrate+nitrite (NOx) during the pilot study. The first method is the online analyzer loaned to the project by the Hach Company (Loveland, CO). The readings from the analyzer are represented by the solid line in Figure 6. The second method is a field method using a modification to Standard Method 4500-NO_3^- B, which employs an ultraviolet (UV) spectrophotometer. The modification developed by WQTS allows for the analysis of nitrate in the presence of nitrite. These results are represented by the red square symbols in Figure 6. These measurements are made by the pilot plant operator as part of the routine monitoring of the system. The third method is performed by EEA Laboratories in Monrovia, CA on samples collected from the pilot plant. The laboratory uses ion chromatography (IC) to measure nitrate and nitrite. These values are represented by the green triangles in Figure 6. The plot in Figure 6 shows that all three methods are in excellent agreement, which validates the applicability of the field UV method for on-site monitoring of the performance of a denitrification system.

The primary challenge with the performance of a BDN process is the ability to maintain low treated water turbidity. CDPH requires that the turbidity in the treated water from a BDN process meet the criterion of <0.3 NTU in 95% of samples collected. Figures 7 and 8 show the turbidity levels in the filtered water from the pilot plant. During the first 43 days of operation, several challenges were encountered that prevented the filter from meeting the turbidity criterion. These challenges included programming problems, clumping of media, and others. On the 43rd day of operation, two operational changes were made that stabilized the performance of the filter. The first was the application of ferric chloride ($FeCl_3$) as the coagulant of choice for clarifying the waste backwash water. The second was increasing the air-scour period during backwashing from 3 to 10 minutes to help break up the clumps formed in the BDN contactors and media filter. Figure 8 shows the turbidity profile since these two modifications were implemented. The turbidity has remained below 0.3 NTU for most of the time with the exception of some spikes during the filter ripening period, which takes place during the first 15 to 30 minutes of a filter run. One high turbidity spike was also experienced during start-up after a power outage at the well that abruptly shut down the system. Nonetheless, the filtered water turbidity has been below the target of 0.3 NTU in more than 95% of the readings taken by the online analyzer.

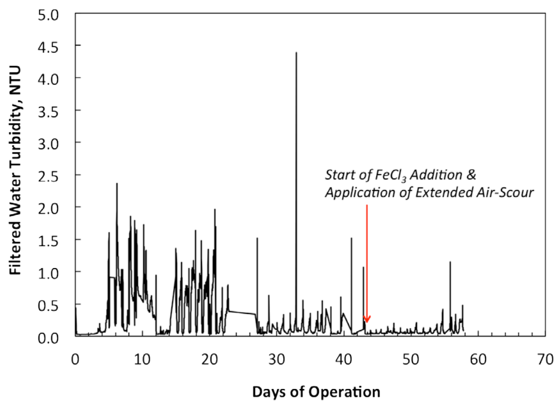

Figure 7 – Filter Effluent Turbidity From Start of Testing

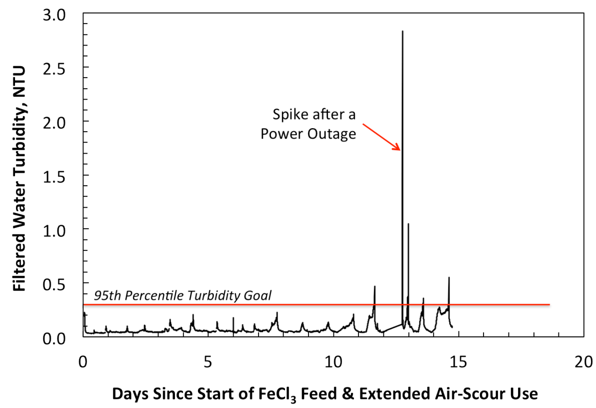

Figure 8 – Filter Effluent Turbidity During Stable Operation

Figure 9 shows of the filter effluent turbidity, headloss, and return washwater flow through the media filter during one of the filter runs. At the start of the filter run, there is a small spike in turbidity due to filter ripening, followed by a relatively stable turbidity level until the end of the 24-hr run period. During the filter run, the headloss across the filter media increases from a low value of <0.5 psi to as high as 14 psi at the end of the run. An increase in headloss translates into an increase in the pumping energy required to push the water through the filter. Finally, Figure 9 shows the return washwater (RWW) flow during this filter run. After backwashing, the waste backwash water is allowed to settle in the waste washwater tank for 1 hour before the return pumps begin bringing the clarified water back to the influent of the media filter. It is noted that the settling period has since been increased to 2 hrs to provide more time for the solids to settle in the tank.

Figure 10 shows the average and range of bacterial levels measured in the raw water, BDN process effluent, media filter effluent, and chlorinated treated water. Bacterial monitoring includes HPC bacteria, total coliform bacteria, and *E. coli*. The results show that HPC bacteria are present in the raw water, as well as the BDN effluent and media filter effluent. This finding is not surprising since HPC bacteria are ubiquitous in the environment and are present in all drinking water sources, including groundwater. Nonetheless, after chlorine addition to the filtered water for a period of 10 minutes, the HPC bacterial counts decreased to a non-detectable level (<1 CFU/mL). Coliform bacteria were absent from the groundwater, but were present in the BDN contactor effluent in one of the four samples collected to date, resulting in an average value of 404 MPN/100mL. However, coliform bacteria were absent in the media filter effluent and the chlorinated treated water. Finally, *E. coli* monitoring continues to demonstrate absence (<1 MPN/100mL) in all samples collected from all locations at the pilot plant.

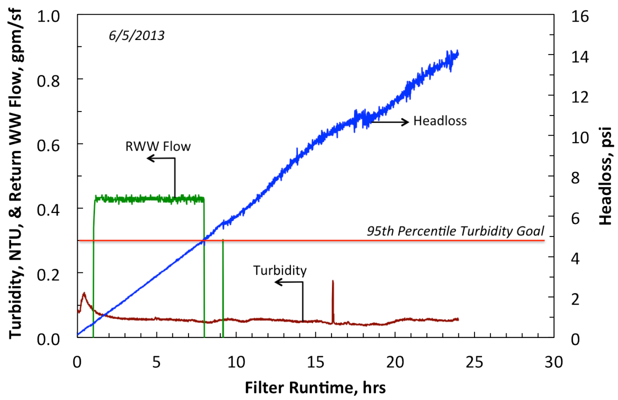

Figure 9 – Turbidity, Headloss, and Return Washwater Flow During an Individual Filter Run

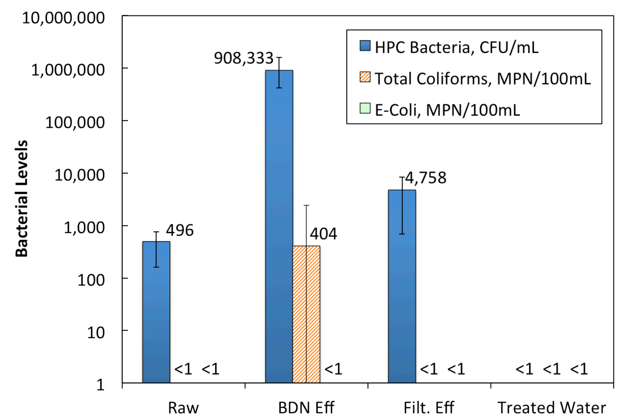

Figure 10 – HPC and Coliform Bacterial Counts Measured during Pilot Operation To Date

With the use of biological treatment, there is always concern about the aesthetic quality of the water, specifically related to the potential formation of sulfide, which gives the water an objectionable taste and odor. The concentration of sulfide is measured at multiple locations in the pilot plant. Figure 11 shows the measured sulfide concentrations in the raw water, filtered water, and the chlorinated treated water. The analysis was conducted using a Hach field instrument with a detection limit of 0.01 mg/L (or 10 µg/L), which is also the resolution of the instruments (i.e., the readings are in 0.01 mg/L increments). The results show that the instrument registered either 0.00 mg/L or 0.01 mg/L on all but one sample from the filter effluent, which was detected at 0.02 mg/L. These levels are very low and suggest that sulfide should not be present in the water generated by the biological denitrification system when operated under the conditions maintained during the pilot study. Figure 12 shows the dissolved oxygen concentrations measured in Well 37-01 water and the filtered water from the pilot plant. During the first 23 days, air was used to provide oxygen to the water and strip the nitrogen gas out of the water. However, due to inefficiencies in the pilot-scale oxygenation contactor, the decision was made to switch from air to pure oxygen. Since then, the filtered water dissolved oxygen has been maintained between 6 and 7 mg/L.

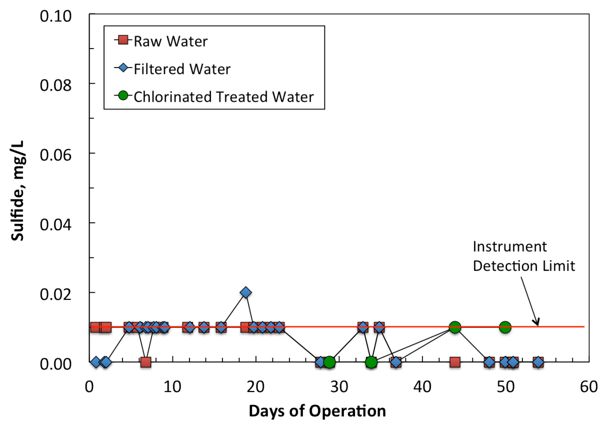

Figure 11 – Sulfide Concentrations Through the Pilot Plant

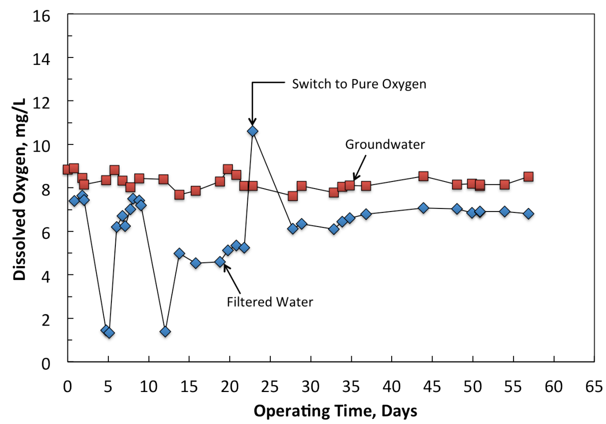

Figure 12 – Dissolved Oxygen Concentrations in Pilot Plant Influent and Filter Effluent

Another important parameter to consider is the level of organic carbon leaving the treatment plant and its potential impact on the formation of disinfection by-products (DBPs) in the distribution system. Weekly samples are collected from the treatment plant influent and effluent and analyzed for Total Organic Carbon (TOC) concentration. The results to date are presented in Figure 13. While the raw water TOC remained consistent at approximately 0.5 mg/L, the treated water TOC level rose to as high as 2.0 mg/L. The only source of TOC added is acetic acid. Every 1 mg/L of added acetic acid represents an addition of 0.4 mg/L of organic carbon. Therefore, if the increase in TOC is caused by the presence of excess acetic acid in the treated water, then an increase of 1.5 mg/L in TOC concentration represents a possible presence of 3.75 mg/L of acetic acid in the treated water. Ongoing effort is evaluating the source of the TOC in the treated water and identifying measures to reduce it.

The formation of DBPs in the treated water was evaluated using samples collected on May 20, which was approximately 34 days from start of pilot operation. The evaluation focused on trihalomethanes (THMs) and haloacetic acids (HAAs) since they are the two chlorination by-products with regulatory limits. The type of testing conducted is referred to as the Simulated Distribution System (SDS) test, which includes the addition of chlorine to the water, holding it in the dark at room temperature for 72 hours to simulate its travel time through a distribution system, and then analyzing the water for THMs and HAAs. Figure 14 shows the results of the test, which was conducted on raw water and filtered water, as well as distilled water as a control. The results show that the THM and HAA5 levels are well below their respective regulatory limits.

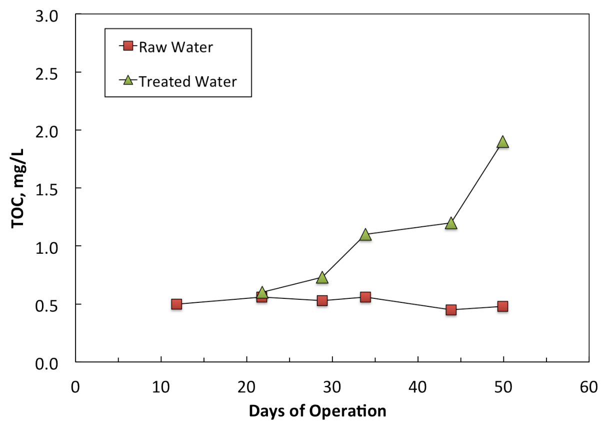

Figure 13 – Total Organic Carbon in Pilot Plant Influent and Treated Water

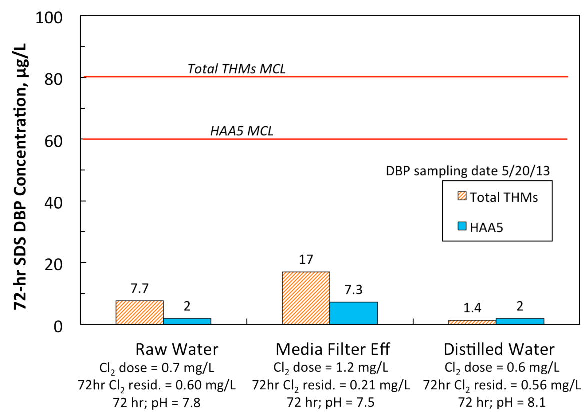

Figure 14 – SDS Disinfection By-Products in Pilot Plant Influent, Filter Effluent, and Control

3.3 WASHWATER RECOVERY

One of the key objectives of this pilot-testing program is to evaluate the applicability of clarifying and recovering the waste backwash water in order to minimize the wastage rate to the sewer. Therefore, during pilot testing, the waste washwater from each process is coagulated with ferric chloride, allowed to settle in a waste backwash water tank, and then returned to the head of the unit process after each backwash. This allowed the solids to accumulate at the bottom of the waste backwash water tank until they are hauled off site for disposal at the County's wastewater treatment facility. Details of the washwater recovery procedures and conditions practiced to date at the pilot plant have been provided throughout this section.

If all the waste backwash water were discharged to the sewer, the operation of the treatment system showed that the water recovery rate is approximately 92% (i.e., the water wastage rate is approximately 8%). With the washwater recovery system operation, the pilot testing to date has showed that the wastage rate can be reduced to 0.33% or lower, a 25-fold decrease in waste production.

SECTION 4.0 – FULL-SCALE SYSTEM CONFIGURATION

Based on the pilot-scale testing results obtained to date, WQTS has developed conceptual design criteria and an overall treatment plant layout for a 1,000 gpm Biological Denitrification Water Treatment Plant (BDNWTP) for District 37. It is important to note that this is not an engineering design, but only a conceptual plan aimed at giving the County a general understanding of the configuration and size of a BDNWTP. It is also noted that no component of this plant is proprietary, which allows the County to use a competitive design-bid-build process for the BDNWTP.

Table 7 lists the design criteria for all of the BDNWTP unit processes and components. Figure 15 presented later in this section shows a conceptual layout of the plant. The treatment plant is sized for a total capacity of 1,000 gpm, and is assumed to have a utilization rate of 90%. This means that the treatment plant will operate at its full capacity approximately 11 months of the year, and will be off for maintenance the remaining one month of the year.

The BDN treatment process will comprise of three 12-ft diameter vertical steel pressure vessels. Each vessel will contain 48 inches of GAC for a total Empty Bed Contact Time (EBCT) of 10 minutes. If one vessel is off-line for backwashing, the EBCT decreases to 6.7 minutes.

For the aeration process, a low-profile, multi-stage aerator was selected for the purpose of this conceptual effort. The advantage of this type of aerator is that it fits inside a typical building, which allows the County to control the outside noise level generated by the air blower. These aerators are available in different sizes to fit different flowrate requirements (e.g., see Lowry Systems® at www.LowryH2O.com). After the aerator, a booster pump is required to re-boost the water pressure for the downstream processes.

After the aeration process, the water is pumped through the media filters. For a 1,000-gpm system operating at a filtration rate of 3 gpm/ft^2 with one filter off-line, four 12-ft diameter vertical steel vessels are required. Each vessel will contain 36 inches of media (24 in anthracite over 12 in sand).

The plant includes five chemical feed systems. The acetic acid feed system was sized to deliver a dose of 38 mg/L. It includes a 5,000-gallon tank and is sized to deliver 66 gallons/day of acetic acid to the plant. At this rate, one 4,500-gallon bulk delivery is required every 68 days of operation. At a dose of 0.2 mg/L as P, phosphoric acid will be fed from a 55-gallon drum at a rate of only 0.7 gallons/day. One 55-gallon drum will be required approximately every 80 days. The ACH feed system is sized to deliver a dose of 15 mg/L, and includes a 500-gallon storage tank with a feed pump system capable of delivering approximately 21 gallons/day of solution. This translates into a partial-load delivery of 500 gallons every 47 days. The ferric chloride system is sized to deliver a dose of 50 mg/L to the waste backwash water, and includes a 500-gallon tank and feed system capable of delivering 9.5 gallons/day of solution. At this rate, the plant would receive a partial-load delivery of 500-gallons every 53 days of operation. Finally, the sodium hypochlorite feed system is sized to deliver a conservative dose of 5 mg/L. The system includes a 5,000-gallon tank and a 60-gallon/day pump system. At this rate, a 4,500-gallon bulk truck delivery will arrive at the plant approximately every 75 days of operation.

Table 7 – Design Criteria for the Unit Processes in a 1,000 gpm BDN Treatment System

Unit Process	Parameter	Value
General	Treatment Capacity	1,000 gpm
	Utilization Rate	90%
Biological Process	Number of Vessels	3
	Vessel Configuration	Vertical
	Vessel Diameter	12 ft
	Media Type	GAC
	Media Depth	48 inches
	EBCT with all Vessels in Operation	10 minutes
	EBCT with One Vessel Off-Line	6.7 minutes
Aeration Process	Source	Ambient Air
	Type	Low-Profile Multi-Stage Aerator
Media Filtration	Number of Vessels	4
	Vessel Configuration	Vertical
	Vessel Diameter	12 ft
	Media Type	Anthracite/Sand
	Media Depth	24-in Anth. over 12-in Sand
	Filt. Rate with One Vessel Off-Line	3.0 gpm/sf
Acetic Acid Feed	Anticipated Dose	38 mg/L
	Storage Volume	5,000 gallons
	Usage Rate	66 gal/day
	Days between Deliveries	68 days
Phosphoric Acid Feed	Anticipated Dose	0.2 mg/L as P
	Storage Volume	55 gallons
	Usage Rate	0.7 gal/day
	Days between Deliveries	79 days
ACH Feed	Anticipated Dose	15 mg/L
	Storage Volume	500 gallons
	Usage Rate	21 gal/day
	Days between Deliveries	47 days
Ferric Chloride Feed	Anticipated Dose	50 mg/L
	Storage Volume	500 gallons
	Usage Rate	9.5 gal/day
	Days between Deliveries	53 days
Hypochlorite Feed	Anticipated General Dose	5 mg/L
	Storage Volume	5,000 gallons
	Usage Rate	60 gal/day
	Days between Deliveries	75 days

Table 7 (Cont'd) – Design Criteria for the Unit Processes in a 1,000 gpm BDN Treatment System

Unit Process	Parameter	Value
Waste Backwash Water	Number of Vessel Backwashes Stored	2 backwashes
	Tank Capacity	60,000 gallons
WBWW Clarifiers	Type	Lamella Plate Settlers
	Number of Settlers	2
	Effective Loading Rate	0.25 gpm/sf
	Flowrate per Settler	77 gpm
	Anticipated Sludge % Solids	0.5%
	Sludge Blowdown Flow	1.9 gpm
Sludge Handling	Days of Sludge Storage	2 days
	Sludge Tank Capacity	7,500 gallons
	Anticipated Flow to Sewer	1.9 gpm
Clearwell	Total Volume	65,000 gallons

The remaining components of the BDNWTP involve the waste backwash water system. A 60,000-gallon waste backwash water tank is included. This tank is sized to store two individual-vessel backwashes. Water will be pumped out of the waste backwash water tank to two lamella-plate settlers sized for a low loading rate of 0.25 gpm/ft^2. If the settled sludge is concentrated to 0.5% solids, the sludge blowdown flowrate is projected at 1.9 gpm. The sludge will be pumped to a sludge tank with a capacity of 7,500 gallons. This is sufficient for two days of sludge storage in the event that discharge to the sewer is temporarily interrupted. Ultimately, the sludge is discharged to the sewer.

Finally, the treatment plant includes a 65,000-gallon clearwell sized to provide sufficient volume for 4-log virus disinfection and two individual-vessel backwashes. Some remaining miscellaneous items are not included in Table 7, including backwash pumps, high-service pumps, and sludge pumps.

Figure 15 shows a conceptual layout of the various treatment plant components. It was assumed that all pressure vessels, waste backwash water tank, sludge tank, and clearwell can be located outside, while all chemical feed systems, aeration process, pumps, and clarifiers are located inside a building. This resulted in a total building footprint of 2,640 ft^2 (40' x 66') and an outdoor equipment slab area of 5,940 ft^2 (90' x 66'). In total, the treatment plant equipment area covers approximately 8,600 ft^2. This does not include any area for operator parking, clearances for truck deliveries, or other miscellaneous facilities that the County may require at the plant.

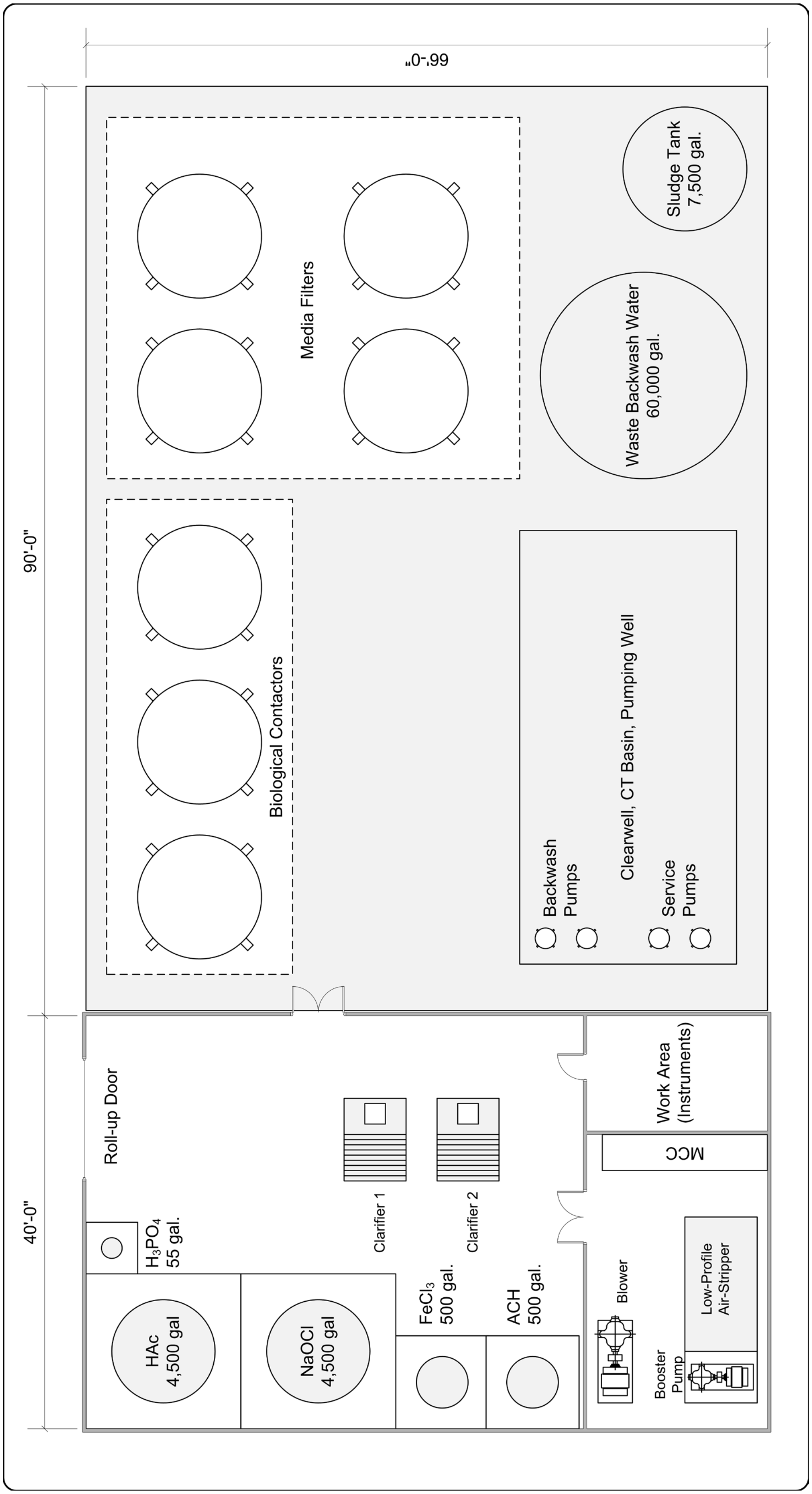

Figure 15 – Possible Layout of a 1,000 gpm BDN Treatment System

SECTION 5.0 – PROBABLE CAPITAL AND ANNUAL O&M COSTS

WQTS has developed a cost model for developing probable capital and annual Operations & Maintenance (O&M) costs for a BDNWTP. This section presents the probable capital and annual O&M costs for the 1,000-gpm BDNWTP detailed in Section 4.0. It is important to note that the probable cost values presented in this section are purely budgetary and have a certainty range between –30% and +50% of estimated values.

Table 8 presents a breakdown of the probable capital cost for the 1,000-gpm BDNWTP. The overall construction cost of the plant is projected to range from $4.6M to $9.8M, with a most probable cost of $6.5M. Professional services required are estimated to range between $1.0M and $2.3M, with a most probable estimate of $1.5M. These services include engineering design, environmental permitting, construction management, administration & legal, and start-up support services. The final component of the capital cost is an estimate of the probable sewer connection fee. This fee was estimated based on the calculation used by the Los Angeles County Sanitation District. However, these calculations are highly dependent on the service area within which the discharge is requested. Therefore, the estimated value of $320,000 is only a placeholder at this time.

With all the components added together, the range of probable capital cost of the 1,000-gpm BDNWTP is projected to be between $5.8M and $12.5M, with a most probable value of $8.3M.

Table 9 lists the probable annual O&M cost for the 1,000-gpm BDNWTP. The total annual O&M cost is projected at $493,000/yr. This includes a total chemical cost of $151,000/yr, which is dominated by the acetic acid cost of approximately $103,000/yr. The energy cost is projected at $143,000/yr, and is dominated by the need for intermediate pumping after aeration. The annual cost estimate also includes $79,000/yr in labor cost, which covers one half-time operator labor cost and overhead. The balance of the annual cost includes $14,000/yr in analytical services, $18,000/yr for sewer disposal, and $88,000/yr in general maintenance cost.

Finally, Table 10 presents a summary of the probable range of capital and annual O&M cost and a projection of the overall probable unit water cost on a $/AF basis. If the amortized capital cost is spread over the production volume, the water cost from capital payments would amount to a probable range of $322/AF to $690/AF, with a most probable value of $460/AF. If both capital and annual O&M costs are spread over the production volume, the total unit water cost would amount to a probable range of $560/AF to $1,200/AF, with a most probable value of $800/AF.

Table 8 – Estimates of Probable Capital Cost for a 1,000 gpm BDN Treatment System
[Certainty is –30% to +50% of Estimate]

Category	Item	Item Cost	Total Cost
CONSTRUCTION			
Equipment	Vessels & Mechanical Equipment	$1,364,000	
	Tanks & Clearwell	$250,000	
	Chemical Feed Equipment	$51,000	
	Misc. Equipment	$250,000	
	Installation Cost	$575,000	
	Total Equipment & Installation		$2,490,000
Building & Slab	Building	$330,000	
	Slab	$239,000	
	Total Building & Slab		$569,000
Contractor Activities	Mobilization	$153,000	
	Site Work & Yard Piping	$306,000	
	Electrical & HVAC	$459,000	
	Instrumentation & Control	$459,000	
	Construction Contingency	$1,109,000	
	Contractor Overhead & Profit	$832,000	
	Initial Media Installation	$121,000	
	Total Contactor Activities		$3,439,000
MOST PROBABLE CONSTRUCTION COST			**$6.5M**
RANGE OF PROBABLE CONSTRUCTION COST			**$4.6M – $9.8M**
PROFESSIONAL SERVICES			
	Engineering Design	$638,000	
	Environmental Permitting	$128,000	
	Construction Management	$638,000	
	Administrative & Legal	$64,000	
	Start-up Services	$64,000	
MOST PROBABLE PROFESSIONAL SERVICES COST			**$1.5M**
RANGE OF PROBABLE PROFESSIONAL SERVICES COST			**$1.0M – $2.3M**
Sewer Connection Fee			$320,000
MOST PROBABLE CAPITAL COST			**$8.3M**
RANGE OF PROBABLE CAPITAL COST			**$5.8M – $12.5M**

Table 9 – Estimates of Probable Annual O&M Cost for a 1,000 gpm BDN Treatment System

Category	Item	Item Cost	Category Cost
Chemicals	Acetic Acid	$102,600/yr	
	Phosphoric Acid	$4,500/yr	
	Ferric Chloride	$4,300/yr	
	Sodium Hypochlorite	$11,800/yr	
	Aluminum Chlorohydrate	$27,800/yr	
Total Chemical Cost			$151,000/yr
Energy			$143,000/yr
Analytical Services			$14,000/yr
Labor			$79,000/yr
Sewer Disposal			$18,000/yr
General Maintenance			$88,000/yr
TOTAL ANNUAL COST			**$493,000/yr**

Table 10 – Summary of Probable Capital and Annual O&M Costs for a 1,000 gpm BDN Treatment System

Item	Probable Range	Most Probable Value
Probable Capital Cost	$5.8M – $12.5M	$8.3M
Probable Annual O&M Cost	$345,000/yr – $740,000/yr	$493,000/yr
Amortized Capital Cost(1)	$466,000/yr – $999,000/yr	$666,000/yr
Total Annualized Cost	$0.81M/yr – $1.74M/yr	$1.1M/yr
Water Cost(2)		
Water Cost from Capital Payments	$322/AF – $690/AF	$460/AF
Water Cost from Annual O&M Cost	$238/AF – $510/AF	$340/AF
Totalized Water Cost	$560/AF – $1,200/AF	$800/AF

(3) Assuming 20 yr payment period at 5% interest rate
(4) Based on a water production rate of 1,443 AF/yr

APPENDIX A – REFERENCES

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