Feasibility Report

for

Joint Treatment and Nutrient Removal

Prepared for

City of Bishop

&

Eastern Sierra Community Service District

October 5, 2015

Feasibility Report
for
Joint Treatment and Nutrient Removal

October 5, 2015

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1 Executive Summary

The City of Bishop (City) and Eastern Sierra Community Service District (District, ESCSD) currently collect and treat wastewater from within their respective jurisdictional boundaries that generally encompasses the City of Bishop and nearby subdivisions and developments. Both the City and the District operate separate but adjacent water resource recovery facilities (WRRF) under separate permits from Lahontan Regional Water Quality Control Board (LRWQCB).

Because of the similar function and adjacent location the City and District engaged R.O. Anderson Engineering (ROA) to identify opportunities for cooperation that would be mutually beneficial. ROA verified that groundwater in the area is adversely impacted by nitrogen in the treated wastewater and it is expected that the LRWQCB will require that this impact be reduced or mitigated. Treatment for nutrient (nitrogen) removal would reduce this impact. The benefit to joint treatment is from economies of scale where it is more cost effective to construct and operate one larger nutrient removal facility than two smaller ones.

There were several meetings and discussions of joint treatment for nutrient removal that included the City, District, and LRWQCB and it was determined that joint treatment for nutrient removal should treat effluent to a total nitrogen concentration of 10 mg/L or less and the combined average daily flow to be treated is currently 1.49 Million Gallons per Day (MGD) and will increase to 2.45 MGD in 50 years. With these criterions the City and District engaged ROA to complete this feasibility study of Joint Treatment and Nutrient Removal.

Fifteen potential treatment technologies and process configurations for nitrogen removal (including irrigation) were preliminarily evaluated for implementation at the City's and District's treatment plants. The three most reasonable alternatives were identified for detailed discussion and included two lagoon conversion alternatives as well as a new mechanical plant alternative. The lagoon conversion alternatives include an alternating zones configuration and Modified Ludzack-Ettinger (MLE) configuration. Each lagoon conversion would consist of using a portion of an existing pond and lining it to provide aeration basins. The mechanical plant conversion was based on construction of an oxidation ditch, which would require the smallest footprint and least amount of concrete for a mechanical aeration basin. Secondary clarification and solids handling were considered to be identical regardless of the process configuration and are addressed in detail within the report.

Analyses of the life cycle costs, capital costs, and process ability to remove nitrogen were performed for each alternative and as a result, the alternating zones alternative was selected as the preferred alternative. This alternative has the lowest construction cost of $4,428,400 and the lowest 50-year life cycle cost of $30,539,000.

A discussion on operation and maintenance requirements for each alternative is also included as well as conclusions and recommendations resulting from this feasibility study. This Feasibility Report is to be reviewed by both the City and District and modified as determined appropriate. Then, as appropriate, the City and District should consider implementing the preferred alternative and submitting this Feasibility Study to the LRWQCB for their review.

2 Introduction

The City of Bishop (City, COB) is responsible for sewage collection treatment and disposal within the City. Figures 1, 2, and 3 show the City limits, sewage collection system, treatment works, and disposal areas. The Eastern Sierra Community Service District (District, ESCSD) is responsible for the same within their district. Additionally through a contract for service the District conveys, treats and disposes of sewage from the Bishop Paiute Reservation. The District facilities are also shown on Figures 1, 2, and 3.

Because of the close physical proximity and similar function, the City and the District determined it may be in their best interest to cooperate on sewage collection treatment and disposal and engaged R.O. Anderson Engineering (ROA) to identify areas where cooperation might be mutually beneficial. ROA performed investigations, had discussions and conducted meetings to identify concepts for cooperation. These concepts were further developed and evaluated and it was determined that joint treatment for nutrient removal should be further investigated. This was pursued by first determining the joint design flow and desired nutrient concentrations. The April 8, 2015 Summary of Recommended Flows and Concentrations (1) was tentatively accepted by both the City and District then presented to the Lahontan Regional water Quality Control Board (LRWQCB). They generally concurred with the recommended flows and concentrations in their June 4, 2015 letter (2). Then a meeting with the City, District, ROA, and LRWQCB was held to further discuss joint

CITY OF BISHOP AND EASTERN SIERRA COMMUNITY SERVICE DISTRICT

JOINT TREATMENT & NUTRIENT REMOVAL

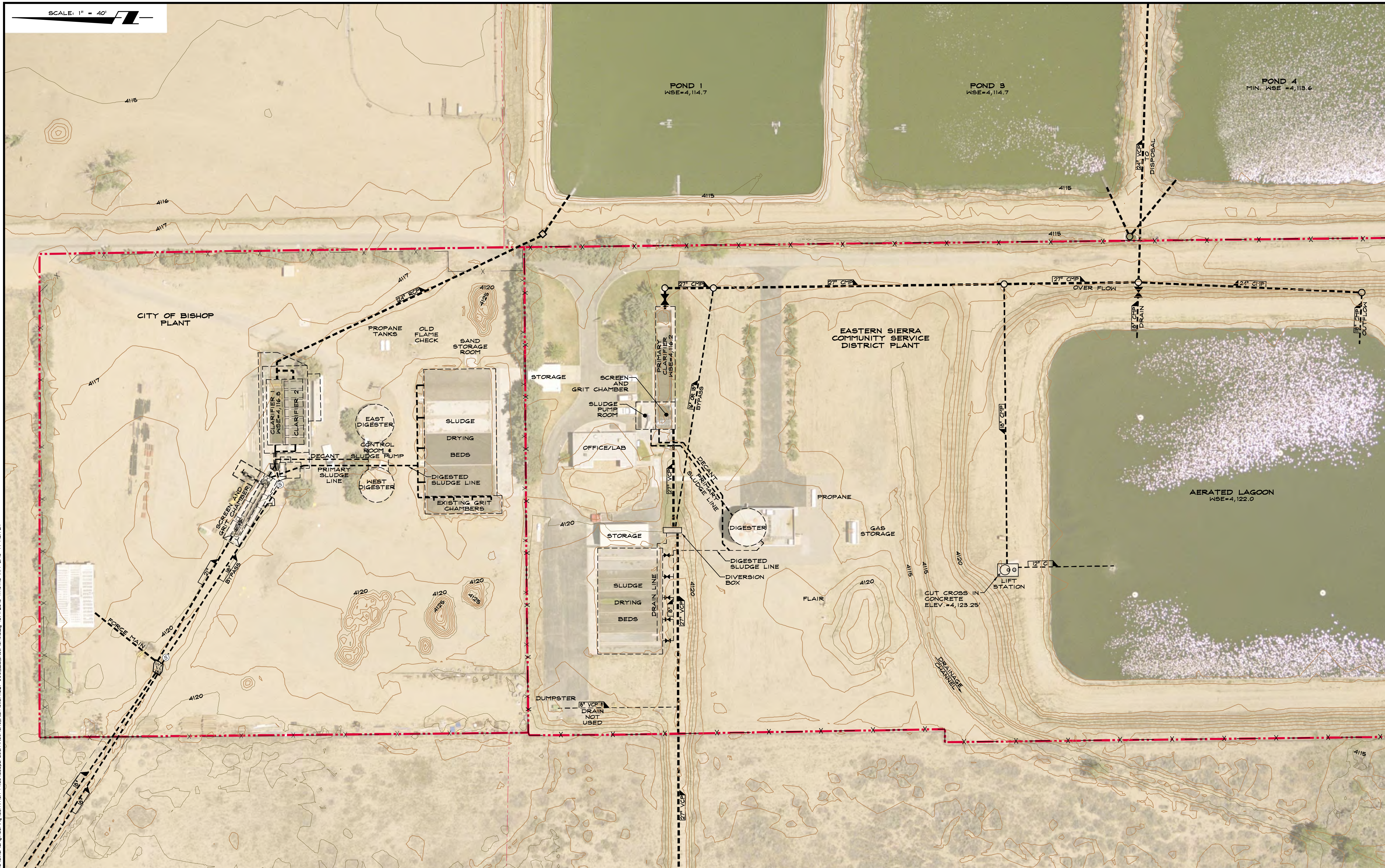
FIGURE 1

RO Anderson

800 400 0 800 1,600 Feet

Scale: 1" = 800'

Print Date:10/01/2015 - - File Name: Bishop_WorkMap02 - - Project Number: 1621-005



NO.	DATE	REVISION BLOCK	BY

SCALE: 1" = 40'

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JOINT TREATMENT & NUTRIENT REMOVAL
CITY OF BISHOP AND ESCSD

PLANT EXISTING FACILITIES
CITY OF BISHOP
AND ESCSD

DRAWN: ETH	JOB: 1621-005
ENGINEER: KRN	DRAWING: SEE PLOT STAMP
SCALE: 1"=40'	SHEET: FIGURE 2
DATE: 10/01/2015	OF: SHEETS

NOTES:

ADD 3.8'± TO PLANT DATUM TO OBTAIN NAVD88 DATUM.

PARCEL LINES SOURCE:	INYO COUNTY G.I.S. JANUARY 2007
CONTOUR SOURCE:	LIDAR COURTESY OF THE CITY OF BISHOP APRIL, 2011 DATUM: NAD83 CA STATE PLANE ZONE 4 ; NAVD88.
AERIAL PHOTO SOURCE:	INYO COUNTY G.I.S. CAPTURE DATE: MAY 5, 2006
ELEVATION SOURCE:	RECORD DRAWINGS ON PLANT ELEVATION DATUM.

NO.	DATE	REVISION BLOCK	BY

SCALE: 1" = 300' (300 0 300 600)

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JOINT TREATMENT & NUTRIENT REMOVAL
CITY OF BISHOP AND ESCSD

OVERALL EXISTING FACILITIES CITY OF BISHOP AND ESCSD

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ENGINEER: KRN	DRAWING: SEE PLOT STAMP
SCALE: 1"=300'	SHEET: FIGURE 3
DATE: 09/30/2015	OF: SHEETS

treatment for nutrient removal in further detail. After this meeting a summary email was prepared by Jehiel Cass of the LRWQCB.

This Feasibility Report is prepared under Task 1B of the July 28, 2014 proposal to the City and District. It is intended to serve as a feasibility assessment of available options for treating the wastewater collected by both entities to a level that produces effluent containing nitrogen concentrations that will not impair the groundwater resource which, as described in more detail below, has been preliminarily determined in reference (1) to be around[1] a total effluent nitrogen concentration of 10 mg/L.

Further this Feasibility Study is intended to address the LRWQCB's request for a feasibility study in their letter (2). However, this Feasibility Study does not address the financing plan or implementation schedule. This information must come from the City and District after consideration of the feasibility of joint treatment for nutrient removal.

2.1 Need for Nitrogen Removal

Nitrogen in wastewater is primarily found in two forms – ammonia and organic nitrogen. Ammonia is toxic to aquatic organisms and as such presents water quality concerns when discharged to receiving waters. Ammonia is readily converted to nitrate in the presence of sufficient amounts of oxygen, which may include oxygenated ground water (3). Since nitrate is toxic to humans (4), the conversion of ammonia to nitrate within the aquifer therefore promotes the degradation of the groundwater resource.

There are 8 monitoring wells placed around the treatment plants and disposal areas operated by both the City and District as shown on Figure 3. Monitoring wells 1 through 5 are for regulatory compliance and monitoring wells 6 through 8 have been installed to better understand groundwater flow and nitrogen concentrations. The details of these wells and monitoring results are presented within the Appendix. The monitoring results show instances where nitrate levels have exceeded the Waste Discharge Requirements (5) (6) of 10 mg/L. Given the

[1] In discussions with LRWQCB Staff varying limits on monthly total nitrogen concentrations may be possible as long as the annual average is 10 mg/L. For example, the January limit might be 12 mg/L and the July limit 8 mg/L.

presence of elevated nitrogen concentrations in the aquifer down gradient of the treatment and disposal areas, the need for nitrogen removal or mitigation is warranted. Further, the required total nitrogen in the effluent from the City and District has been determined to be around 10 (1) and currently the total nitrogen in the City and District effluent varies both seasonally and between treatment plants from approximately 11mg/L to approximately 35 mg/L.

It is noted that this Feasibility Study is focused on joint treatment for nitrogen removal. Concurrently the City is independently investigating and experimenting with nitrogen reduction from the City's existing treatment works.

2.2 Definitions

For the purposes of this report, total nitrogen is composed of two groups of nitrogen classification, viz. *Total Kjeldhal Nitrogen (TKN),* and *total oxidized nitrogen (TON)*. Total Kjeldhal Nitrogen is composed of ammonia (NH_3), and organic nitrogen, while total oxidized nitrogen is composed of nitrate (NO_3^-) and nitrite (NO_2^-). Given these definitions total nitrogen is, in this context, the sum of the following items:

- Ammonia
- Organic Nitrogen

} Total Kjeldhal Nitrogen

- Nitrate
- Nitrite

} Total Oxidized Nitrogen

Total inorganic nitrogen (TIN) includes ammonia nitrogen and TON, while total nitrogen (TN) includes TKN and TON. These definitions are especially important when considering permitting limits as regulators may either include or exclude organic nitrogen from a permit limit. LRWQCB has indicated (2) that the forthcoming discharge permit is expected to include a TN limit of 10 mg/L. Since this limit includes organic nitrogen, the effects of algae and sludge in the effluent can have dramatic effects on total nitrogen concentrations and permit compliance.

3 Existing Conditions

The existing wastewater collected from within both the City and District is conveyed primarily via gravity transmission to each of the respective treatment plants as illustrated in Figure 1. Both plants are equipped with head works to screen large solids and debris followed by grit removal and primary clarification. Primary effluent is then conveyed to aerated lagoons.

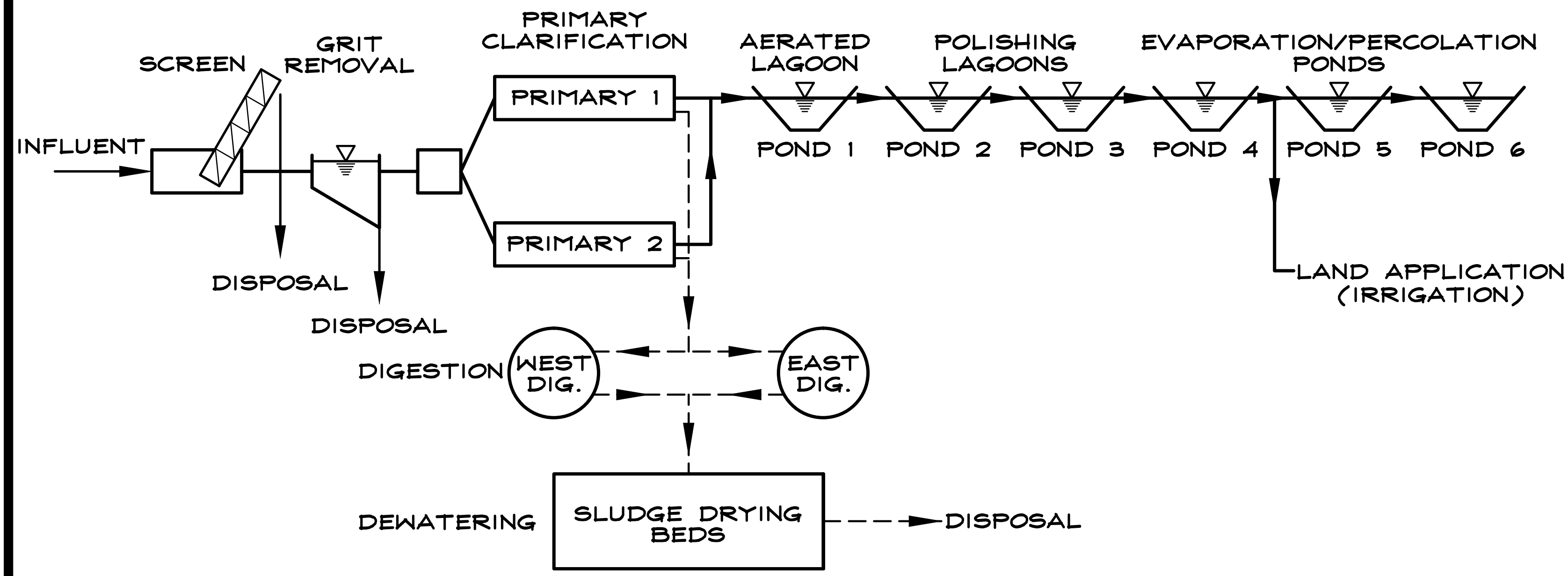
The City has 3 treatment lagoons operated in series while the District has a single lagoon. Following treatment the City's effluent enters their evaporation percolation Pond 4 and then may be conveyed to disposal via irrigation or evaporation/percolation in the City's Ponds 4, 5, and 6.

Following treatment the District's effluent is conveyed via a pipeline to disposal via irrigation or evaporation/percolation in the District's Ponds 1, 2, and 3. The existing process diagram for both the City and the District is presented in Figure 4.

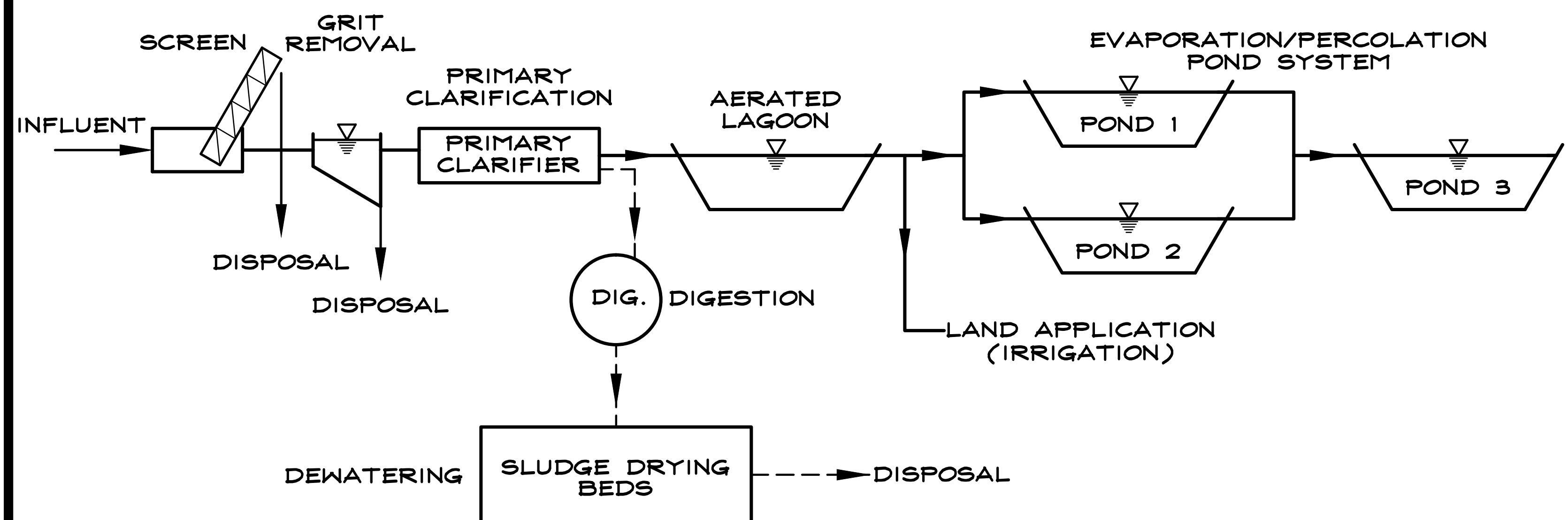
3.1 Flows

The existing flows for both the City and District were previously established in the letter report prepared by R.O. Anderson Engineering, Inc. (ROA) (1). The findings of that report are generally listed below:

1. City of Bishop Existing Flows
    a. Annual Average Flow: 0.697 MGD
    b. Max Daily Flow: 0.866 MGD
    c. Minimum Daily Flow: 0.607 MGD
    d. Instantaneous Peak Daily Flow: 1.21 MGD
    e. Instantaneous Minimum Daily Flow: 0.358 MGD
2. Easter Sierra Community Service District Flows
    a. Annual Average Flow: 0.703 MGD
    b. Max Daily Flow: 0.92 MGD
    c. Minimum Daily Flow: 0.512 MGD
    d. Instantaneous Peak Daily Flow: 1.36 MGD
    e. Instantaneous Minimum Daily Flow: 0.287 MGD



CITY of BISHOP EXISTING PROCESS DIAGRAM



E.S.C.S.D. EXISTING PROCESS DIAGRAM

FIGURE 4
EXISTING PROCESS DIAGRAM
City of Bishop/E.S.C.S.D.

09/29/15

1621-005

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3.2 Influent Characteristics

The influent characteristics of the City and District are generally representative of normal municipal waste water, with respect to concentrations of BOD_5, Total Nitrogen, TKN, Ammonia and alkalinity. A brief summary of each municipality is included below:

City of Bishop:

- BOD_5 = 279 mg/L[2]
- Total Nitrogen = 39 mg/L[3]
- TKN = 39 mg/L[3]
- Ammonia = 26 mg/L[3]
- Alkalinity not determined, assumed to be the same as ESCSD

Eastern Sierra Community Service District:

- BOD_5 = 248 mg/L[2]
- Total Nitrogen = 43.4 mg/L[4]
- TKN = 31.8 mg/L[4]
- Ammonia = 30.9 mg/L[4]
- Alkalinity = 220 mg/L[4]

4 Future Conditions

4.1 Future Flows

Similar to the existing flows, the projected future flows are outlined in the letter report by ROA (1). Projected flows through the next 50 years are estimated to be about 0.95 MGD and 1.17 MGD for COB and ESCSD, respectively. Applying similar peaking factors used in the development of existing flows, the

[2] Average based upon more than 20 samples

[3] Average based upon 2 discrete samples

[4] Average based upon 9 discrete quarterly samples

instantaneous peak flows estimated for a 50 year projection are 1.65 MGD for COB and 2.26 MGD for ESCSD, for a joint (COB and ESCSD combined flow) instantaneous peak flow of 3.91 MGD. The joint average monthly flows, which should be the basis of design for any future expansion of WRRF infrastructure capacity, are projected to be 2.25 MGD. Applying a small factor of safety, incase growth is more than anticipated, a joint monthly average flow of 2.45 MGD is assumed. That is, the design of any WRRF improvements should either provide for or be readily expandable to accommodate the projected average monthly flows. Given the monthly variation in flows for the Bishop area a 2.45 MGD maximum monthly flow is expected to result in a 2.30 MGD annual average flow.

Given that the buildout of both the City and the District will encompass primarily municipal growth and development, the influent characteristics are not expected to deviate much from today's concentrations, including nitrogen. However, should heavy industrial or agricultural development occur, e.g. refining, breweries, or meat and dairy processing, it may be necessary for the City and District to develop pretreatment conditions so as not to create an adverse impact on the future performance of the joint WRRF.

5 Nitrogen Removal Options[5]

5.1 Physical/Chemical Nutrient Removal

There are both physical and chemical nitrogen removal methods for wastewater which have been successfully employed in the past. These methodologies typically are either prohibitively expensive or create additional operational burdens and/or hazardous scenarios. For these reasons most municipalities and other WRRFs no longer rely on these options (7). For completeness, they have been briefly included in this report but are not considered viable options for nitrogen removal for either the City or District.

[5] Design criterion based upon nationally recognized standards. Please refer to the references listed at the end of this report in Works Cited section. Numerical references that occur in the report text, e.g. (1), indicate reference in works cited.

5.1.A Ammonia Stripping

Ammonia stripping involves pH adjustment of the wastewater to around 11.5 which forces nitrogen into the ammonia species. Chemical addition, often at a very high cost, is required in order to drive the pH to sufficient levels for stripping. This process in turn also creates a large sludge volume that must be disposed of. Since the ammonia is stripped with forced air, production of odors is also a concern and may, in some instances, present a conflict with air quality permits (8).

5.1.B Breakpoint Chlorination

As the name implies, breakpoint chlorination involves the addition of chlorine (Cl_2) to the point where ammonia is chemically converted into nitrogen monoxide or nitrogen gas. This requires chlorine concentrations of up to 10 mg/L per mg NH_3. Obviously, chlorine concentrations in this range present safety concerns, and where breakpoint chlorination is administered upstream of receiving waters the potential for large fish kills is possible, as well as the production of odors (4).

5.1.C Ion Exchange

Ion exchange performs nitrogen removal by running wastewater through an exchange media such as Zeolite or other synthetic resins. Ammonium (NH_4^+) is the form of nitrogen removed, and the process requires regeneration of the media during which ammonia stripping may be employed (4). The production of lime sludge, solids, and odors are also operational issue involved with this process.

5.1.D Membrane Filtration

Membrane filtration involves forcing wastewater across a selectively permeable membrane via a pressure differential often provided by booster pumps which can increase energy costs considerably. In addition to energy costs, capital costs associated with filtration equipment can be quite high (4). The contaminant bearing brine that is rejected by the membrane and containing high concentrations of nitrogen must also be disposed of which presents an additional operational burden.

5.2 Biomass Uptake Nutrient Removal (Irrigation)

Biomass uptake is the process by which effluent nitrogen is utilized by plants during cellular growth. Typically, ammonia and nitrate laden effluent is distributed via land application to receiving crops. Nitrogen is converted from ammonia and nitrate into amino acids and ultimately into proteins, however, the biological uptake is limited to about 12 – 14% of the biomass (4). For this reason, biomass uptake often requires substantial amounts of acreage over which effluent must be applied. Since nitrogen uptake is limited to both individual species' growth during the growing season as well as during varying environmental applications, the judicious placement of effluent is necessary to avoid high concentrations of nitrogen entering the groundwater. Since the growing season is limited to certain times of the year, it is necessary to provide for the storage of effluent during the offseason, often at substantial material costs and large footprints for the storage reservoir.

Different conceptual irrigation scenarios have been brought up many times in the numerous meetings, discussions, and investigations prior to this feasibility study. To date all have shown some benefit to nutrient removal but none have been found to be the preferred alternative. It is prudent to complete one more preliminary evaluation of irrigation as a means of nutrient removal. This is presented below with the conclusion that Biomass Uptake Nutrient Removal (Irrigation) will have a capital cost of $8,525,000. Because of this very high cost and the uncertainty of obtaining the required 500 fertile acres of land for long term irrigation this alternative is dismissed and does not justify a more detailed analysis.

The simplified preliminary analysis is presented below.

5.2.A Area

Irrigation must be at the agronomic rate meaning that virtually all irrigation water applied must be used by the crop. The water used by a crop is called the evapotranspiration. In the Bishop area evapotranspiration of crops that can be harvested for animal feed is approximately 61 inches of water per year (5.08 feet). This varies from approximately 1.6 inches in December to approximately 8.2 inches in July. A future combined annual average flow of 2.30 MGD will generate 2,576 acre feet of effluent per year. Applying this at the agronomic rate of 5.08 feet per acre per year requires approximately 500 acres of crop.

5.2.B Storage

The water used by the crop or evapotranspiration varies with the temperature, wind, and growth of the plant. As discussed above for the Bishop area this is expected to be a minimum of 1.6 inches in December and a maximum of 8.2 inches of water in July for an average of 5.1 inches per month. One inch of water over 500 acres is 13,600,000 gallons. Therefore, in December 1.6 inches of effluent, or 21,700,000 gallons, would be applied and the equivalent of 3.5 inches, or 47,500,000 gallons, would be diverted to storage. In July 8.2 inches would be applied. Of this amount 5.1 inches, or 69,200,000 gallons, would come from the treatment and 3.1 inches, or 42,100,000 gallons, from storage. Considering the varying application rates for each month, a storage volume of 161,000,000 gallons, or 495 acre feet, is required. The storage reservoir(s) would be full in April of each year and eventually drain during the summer until they are empty in September.

Because the effluent used for irrigation would contain nutrients it could not be allowed to infiltrate and the storage would have to be lined. Additionally the existing treatment lagoons may be leaking and may have to be lined.

5.2.C Nutrient Removal

The nutrients in the effluent would be taken up by the crop and in order to remove the nutrients from the area the crop would have to be harvested and removed. The crop could not be grazed because the foraging animals would only remove a portion of the nutrients and the majority of the nitrogen would be returned to the area as urine and fecal matter.

Further, the crop would have to use virtually all of the nutrients supplied in the effluent. This would require crops that produce significant biomass that is high in nitrogen. Assuming an effluent total nitrogen concentration of 30 mg/L and 61 inches of effluent applied results in 414 pounds of nitrogen applied per acre.

Alfalfa harvested as hay typically removes 65 pounds of nitrogen per ton (9). Therefore, 6.4 tons per acre of alfalfa would need to be produced. This is a very high rate of production for the Bishop area and may not be reasonably possible. Other crops such as reed canary grass cultivated on fertile soil under ideal conditions and harvested as hay could potentially remove 414 pounds per acre of nitrogen.

5.2.D Required Improvements and Approximate Costs

To implement a biomass uptake nutrient removal system capable of removing practically all nutrients would require 500 acres of fertile land that is fenced for public exclusion. The area would require an efficient irrigation system, most probably consisting of a sprinkler system and the land would have to be leveled or smoothed to allow harvesting and removing the crop, probably as hay. It is estimated that the fencing, irrigation system, leveling, and cultivation would cost approximately $5,000 per acre for a total capital cost of $2,500,000. Approximately 500 acre feet of lined winter storage is required. This could be two 15 foot deep ponds covering a total of 1,450,000 square feet at a capital cost of approximately $2.50 per square foot or $3,625,000. Additionally, approximately 800,000 square feet of existing treatment must be lined at a capital cost of approximately $2.50 per square foot or $2,000,000. Finally, there would be incidental capital costs for pumping systems, earth work, monitoring wells, and instrumentation of say $400,000 for a total capital cost of $8,525,000. This does not include any land costs.

5.3 Biological Nutrient Removal

Biological nutrient removal (BNR) is the process by which nitrogen is converted from ammonia to nitrate and then ultimately removed as nitrogen gas through bio-chemical reactions within the wastewater environment. BNR is generally the cheapest, safest, and most effective process for the removal of nitrogen from wastewater (4). As described above, nitrogen is primarily found in municipal wastewater in two primary forms: ammonia (and ammonium as determined by pH) and organic nitrogen. Oxidized forms of nitrogen – nitrate and nitrite, may also be present at relatively low amounts. In a simplified overview of the BNR process ammonia is first converted to nitrate in the presence of oxygen through a two-step process called nitrification and then ultimately removed from the water as nitrogen gas during denitrification, which takes place in the absence or near absence of oxygen. The BNR process is strongly influenced by several factors, including pH, temperature, alkalinity, dissolved oxygen, as well as hydraulic detention and biological (mean cell) residence times within the treatment system (4) (7).

5.3.A Nitrification, Denitrification, at a Glance

Nitrification is, generally, the two step process by which ammonia is converted to nitrate. This conversion is completed by two distinct groups of autotrophic organisms – ammonia-oxidizing bacteria (AOB) which convert ammonia to nitrite, and nitrite-oxidizing bacteria (NOB) which convert nitrite to nitrate (4). The process of nitrification is completed under aerobic conditions and approximately 4.6 pounds of oxygen are consumed for every one pound of ammonia converted, which requires that aeration of the wastewater be provided. An additional 7.1 pounds of alkalinity are likewise consumed for every one pound of ammonia converted to nitrate and as such, the nitrification process requires a lot of oxygen and potentially the addition of alkalinity in order to complete (7). The excessive consumption of alkalinity can result in low pH (acidic) levels in the wastewater (4). Further, nitrification only converts nitrogen; it does not remove nitrogen from the wastewater. While converting ammonia to nitrate reduces the aquatic toxicity potential as described in Section 2, the presence of nitrates and/or nitrites degrades the groundwater resource because both are toxic to humans (4).

Denitrification completes the process of nitrogen removal. During denitrification nitrates are converted under anoxic conditions into nitrogen gas and released to the atmosphere. Denitrification requires a readily available organic carbon source in order to complete, which is either provided within the wastewater itself or, depending on the process configuration in place, additional carbon must be added. Denitrification consumes approximately 2.9 pounds of BOD per pound of nitrate reduced (removed as nitrogen gas) (4). The process of denitrification in turn yields 3.6 pounds of alkalinity per pound of nitrate removed, as well as 2.86 pounds of oxygen – or in other words, denitrification returns about 50% of the alkalinity and 60% of the oxygen consumed during nitrification (4). For these reasons, it is possible that no additional alkalinity is required to complete the nitrification process and the nitrification oxygen required is reduced when denitrification is occurring. More importantly, denitrification reduces the potential for groundwater resource degradation caused by discharging of nitrogen in the effluent.

6 BNR Process Configurations

6.1 Lagoon Conversion

Conversion of the existing lagoon systems is an attractive alternative because of the obvious capital savings over constructing a completely new mechanical treatment plant. Lagoon conversions may have plastic lined sloping walls and a plastic lined bottom.

As mentioned previously, the BNR process is strongly influenced by many parameters, several of which can be controlled operationally if the process configuration is designed well. Traditional lagoons afford very little control to WRRF operators and as such are subject to effluent of dramatically varying quality, even month to month. Some problems that result from a lack of operational control include the buildup of sludge and the inability to mix and recycle as well as waste from the aeration zones which make consistent nitrogen removal all but impossible. A lagoon may well provide nitrification during parts of the year, however, nitrification only converts nitrogen, it does not remove it and as such nitrified effluent still poses a degradation risk to the groundwater resource.

There are two primary process configurations that are attractive for conversion of a lagoon system, viz. alternating zones or the MLE configurations. Both of these process configurations are readily adaptable to the existing aerated lagoons located at either the COB or ESCSD treatment plants. Both configurations require that new aeration equipment be installed as well as new secondary clarification infrastructure. Piping for return activated sludge (RAS) and waste sludge will be required, which will necessitate additional pumping equipment as well.

6.1.A Alternating Zones

In an alternating zone configuration, influent enters the aeration basin and is passed through a series of aerobic and anoxic zones. In the aerobic zones, ammonia is converted into nitrate via nitrification, and as the nitrified mixed liquor passes into the subsequent anoxic zone nitrates are converted to nitrogen gas and removed from the water. Each zone may also be alternated individually from aerobic to anaerobic conditions by the addition or removal of aeration which promotes mixing in addition to nitrogen conversion and removal.

After passing through the aeration basin, the mixed liquor is delivered to a secondary clarifier where activated sludge settles out to either return or waste destinations. RAS is delivered to the influent line at the head of the aeration basin to seed the mixed liquor with a diverse bacteriological community. Waste sludge is then delivered to solids handling which may first include advanced digestion or continue directly to dewatering and disposal. Typical effluent total inorganic nitrogen (TIN) can be expected in the range of 5 to 10 mg/L under the alternating zone configuration (4).

Because this process is readily retrofitted to existing lagoon systems, and the potential nitrogen concentrations are commonly below 10 mg/L, this process configuration is recommended as a preferred alternative as discussed later in this report.

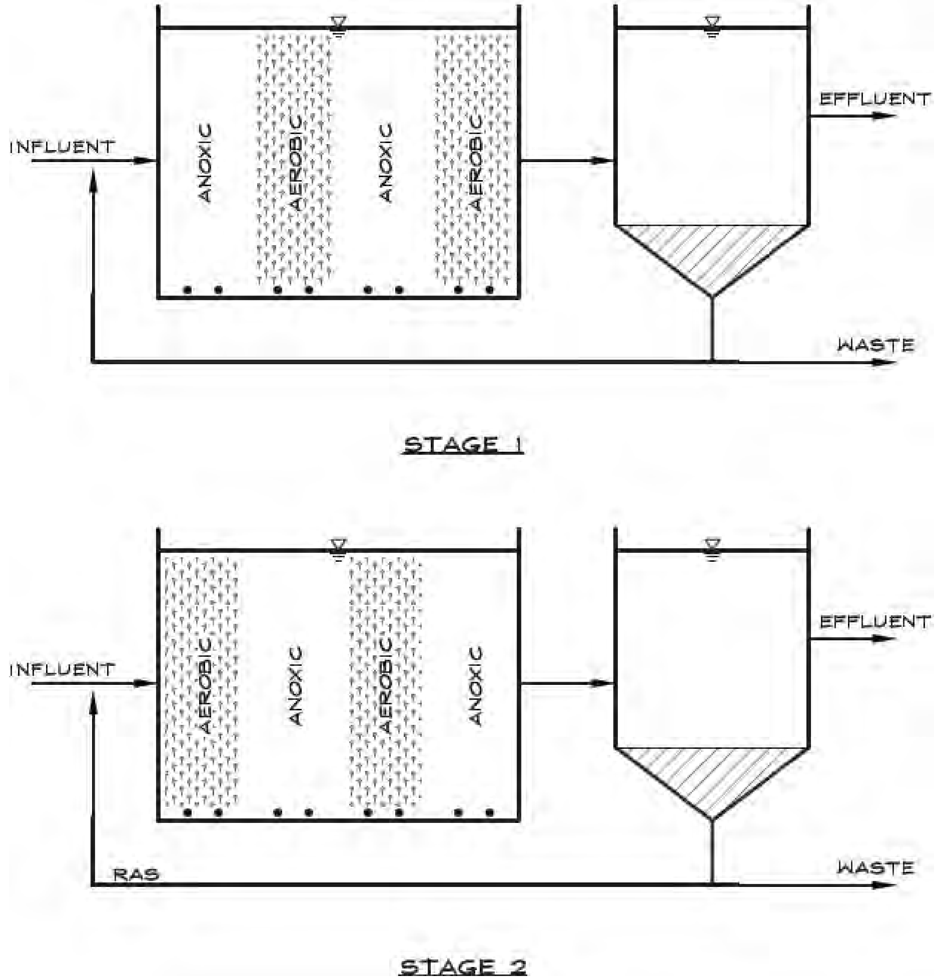

Figure 5: Alternating Zones

6.1.B MLE

The MLE, or Modified Ludzack-Ettinger process, is the traditional BNR process configuration consisting of a standard aeration basin followed by secondary clarification with both RAS and waste lines. Nitrogen conversion and removal is accomplished by providing a preceding anoxic zone (or anoxic basin) just upstream of the aeration basin and providing for recycle of the mixed liquor from the end of the aeration basin to the anoxic zone at a rate of between two (2Q) and four (4Q) times the influent flow rate. Recycling of the mixed liquor in this fashion provides well nitrified liquor directly to the anoxic zone where nitrates are then denitrified. Additional non-aerated mixing of the anoxic zone is required and typically provided by either a mechanical mixer (turbine) or submersible pump.

Denitrification is limited to nitrates in the recycle and the process is limited to a potential nitrogen removal of 82% at a recycle rate of five (5Q) times influent flow, which may be impractical (7). Effluent TIN concentrations under the MLE configuration will generally be in the 6-10 mg/L range (4). The MLE process is considered further in Section 7.

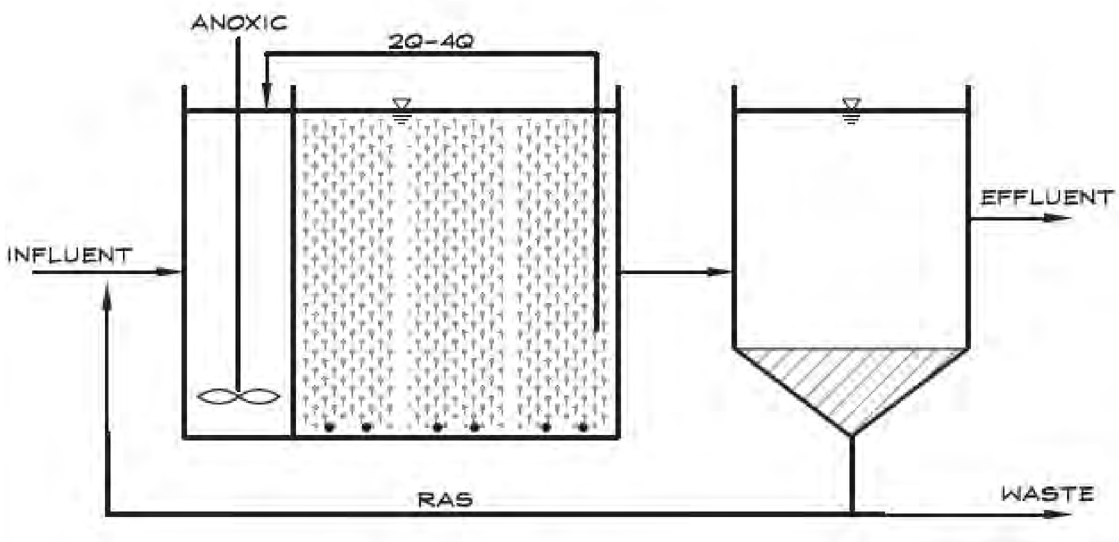

Figure 6: MLE Reactor

6.2 Mechanical Plant Conversion

Converting to a mechanical plant provides for the most robust solution to nitrogen removal as a mechanical plant affords the greatest operational control and flexibility. Process configurations can be designed for more efficiency and ease of operation as a new plant is not limited to the physical constraints experienced when retrofitting an existing system. Mechanical plants will, in general, occupy a substantially smaller footprint than the existing lagoon system. The tradeoff for increased functionality and control with a mechanical plant conversion is, however, capital costs which are often much higher than conversion of the existing lagoon system.

6.2.A Sequencing Batch Reactor

Sequencing batch reactors, or SBR's, are often referred to as package plants because they perform the entire BNR process within a single, almost completely pre-manufactured basin. The SBR process begins with influent filling the basin while being mixed in anoxic conditions and allowed to react. Next, the basin is aerated and the mixed liquor reacts under aerobic conditions. Following the reaction period, the sludge is allowed to settle while a portion of the clear effluent is decanted from the top of the water column. Finally, a portion of the settled sludge is wasted from the basin to solids handling and refilling begins. Typical TN levels are around 4 to 5 mg/L, however, TIN levels as low as 1.6 mg/L are possible (7). SBRs are typically run in parallel configurations with two basins each performing opposite functions simultaneously – e.g. one basin is filling while the other is decanting. It is possible to run single basin SBR's so long as flow equalization and storage are provided to retain incoming wastewater flows while the SBR is between filling cycles.

SBR package plants are readily manufactured to treat flows of between 0.01 and 0.25 MGD, and may be designed for flows as high as 0.5 MGD (10). While an SBR plant can be constructed in place for flowrates in excess of 0.5 MGD, they generally will not offer cost savings over other mechanical plant configurations. Therefore, because the present combined flow and especially the future design combined flows are much greater than what a package SBR plant could treat, the SBR process configuration is not recommended as a preferred alternative.

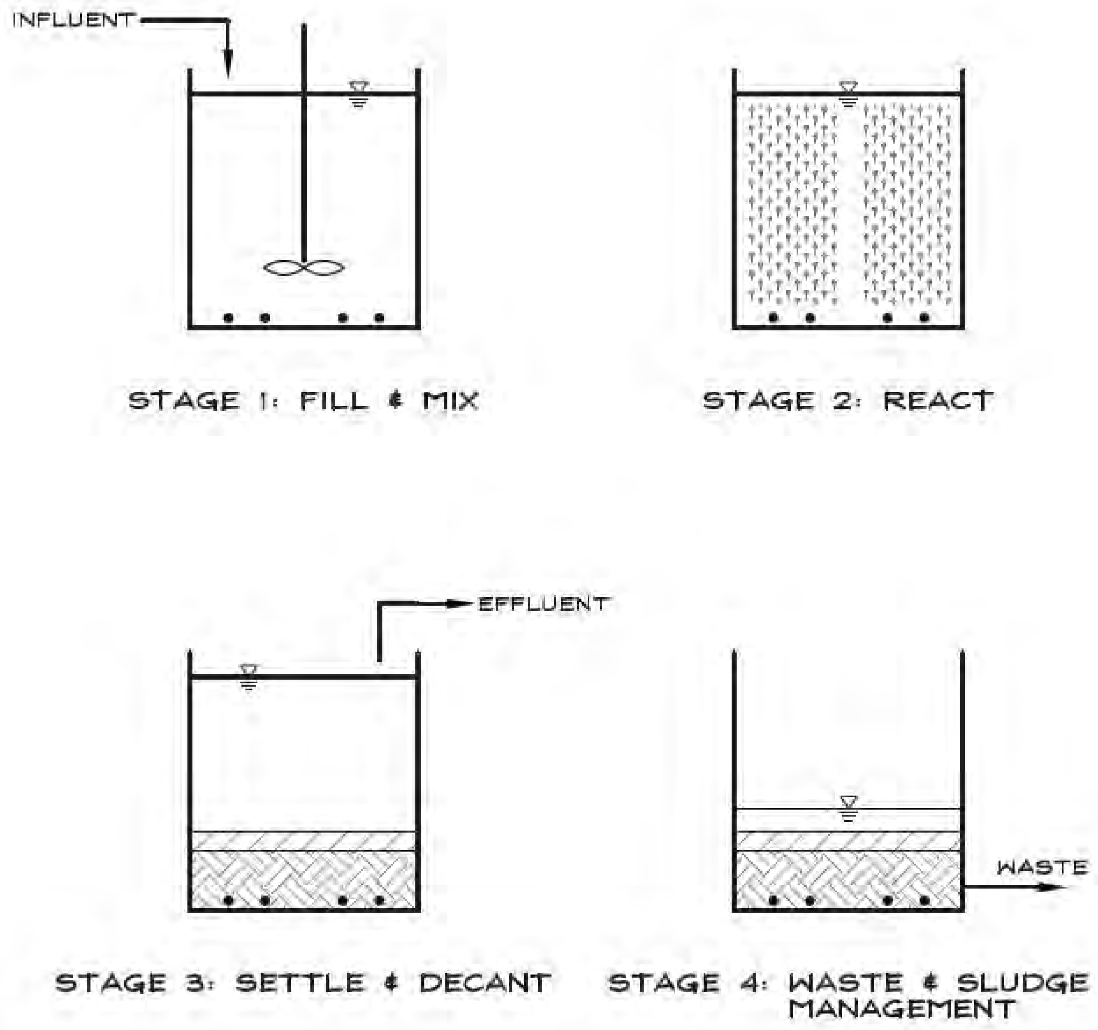

Figure 7: SBR

6.2.B Oxidation Ditch

The oxidation ditch utilizes a "racetrack" or circulative configuration to accomplish the BNR process. Mixing is provided on each end of an elliptical basin that has an intermediate baffle wall located in the center. Primary influent enters the basin near one end of the reactor and flows continuously around the basin until exiting towards secondary clarification near the point of entry, which is typically accomplished by spilling of the effluent over a side weir into a collection channel. On average, the wastewater within the reaction basin may make 200 revolutions before exiting to secondary clarification (7). No additional piping or pumping is necessary to provide for internal recycle as this is accomplished by the basin mixers (7), and at recycle rates of two hundred (200 Q) to three hundred (300 Q) times the influent, significant removal of nitrogen can be achieved (4).

The oxidation ditch is a fairly versatile configuration owing to the long circumferential length that wastewater must travel through the reaction basin. The reaction basin can be further configured for alternating zones (4), on/off aeration (7), or potentially even a low DO configuration by providing for the automatic control and proper location of aerators. Oxidation ditches are good candidates for new mechanical plant construction and for flowrates of 2 to 10 MGD. Typical TIN concentrations may range from 2 – 5 mg/L (4). An oxidation ditch process is considered further in Section 7.

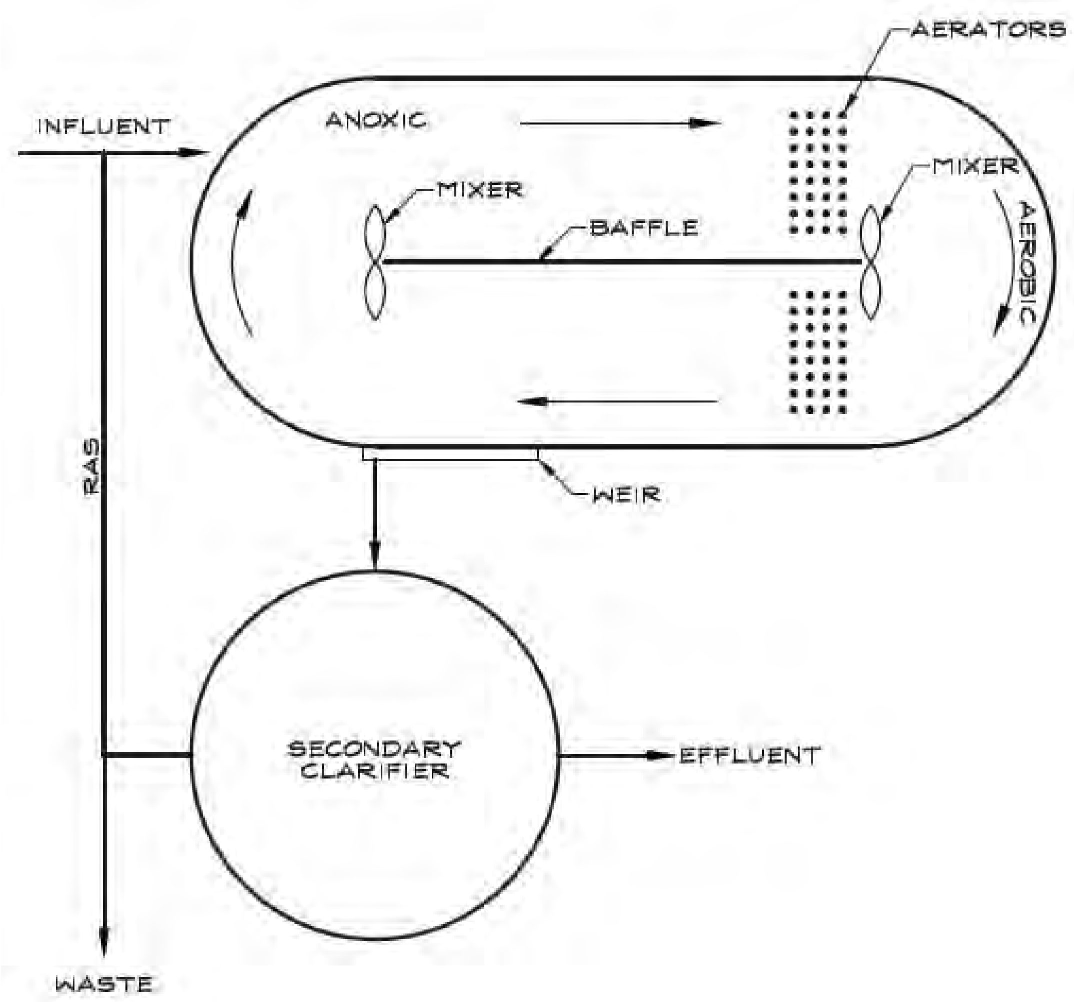

Figure 8: Oxidation Ditch

6.2.C MLE

A mechanical plant MLE would essentially be the same as that described for the lagoon conversion, except it would include a concrete structure and the aeration basin would occupy a smaller, but deeper, footprint. Effluent TN concentrations will also be similar, and recycle lines, RAS, and waste facilities, along with secondary clarification will be required (Refer to Figure 2). A mechanical plant MLE is not considered further because of additional costs over a lagoon conversion MLE plant.

6.2.D Low DO (Simultaneous Nit/Denit)

In a low dissolved oxygen (DO) process, aeration is applied continuously to the aeration basin at concentrations of between 0.1 to 0.5 mg/L, with a typical concentration of 0.2 mg/L. DO concentrations in this range provide conditions within the aeration basin to simultaneously nitrify and denitrify, as the ML encounters aerobic and anoxic conditions provided by the mixing effort of the aerators which are typically provided by fine bubble diffusers. Because the residence time may be longer than other BNR configurations, a larger or oversized aeration tank can be required, often at additional capital expense. The advantage of the low DO configuration is that aeration expense may be cut by 20% over other configurations that target higher DO concentrations (4). Additionally, no recycle is required as both zones are occurring simultaneously within the basin. Typical effluent TIN will average in the 5 – 10 mg/L range (4). A low DO plant is not considered further because it is readily converted from the alternating zones configuration if desired and is expected to achieve similar effluent nitrogen concentrations.

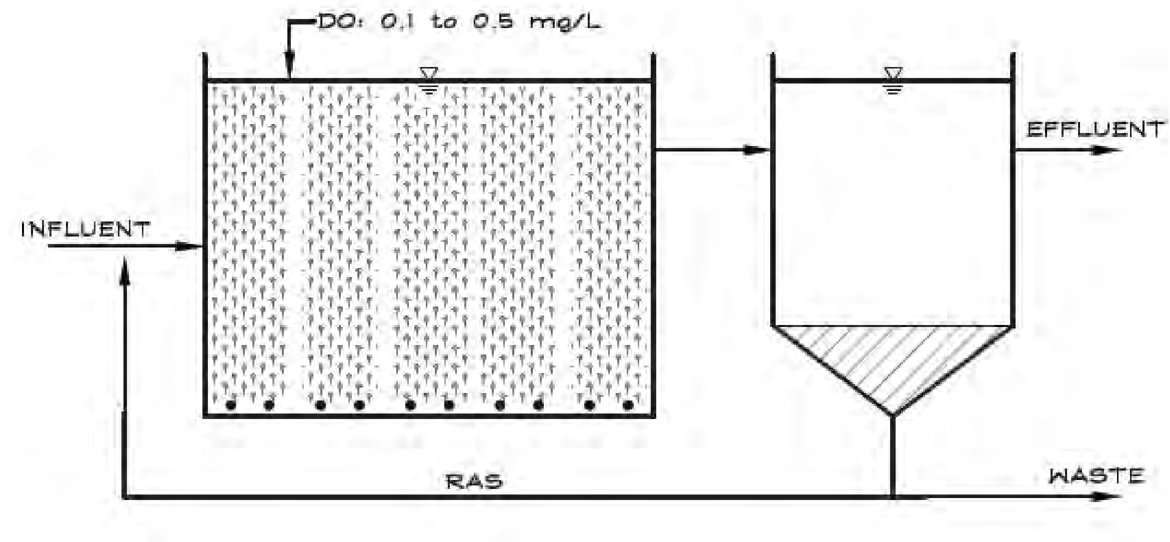

Figure 9: Low DO

6.2.E Step Feed

In the step feed configuration, raw wastewater influent is metered into the aeration basin through a series of feed inlets, allowing for the distribution of nutrients to various stages of the BNR process. Typically, the aeration basin is baffled or otherwise separated into alternating zones of aerobic and anoxic conditions, with influent added to each of the anoxic locations. This configuration capitalizes upon the MLE concept by providing nitrified ML from the preceding aerobic zones into subsequent anoxic zone for denitrification without the need for internal recycle. The addition of raw influent provides the necessary carbon source (BOD) for the denitrification process. Typical TN concentrations will range from 1 mg/L to 14 mg/L (7), with an expected annual average concentration of 7 mg/L. Step feed is not considered further because of additional piping and capital costs necessary to distribute influent to each anoxic section. However, the step feed process can be readily converted from the alternating zones configuration if desired.

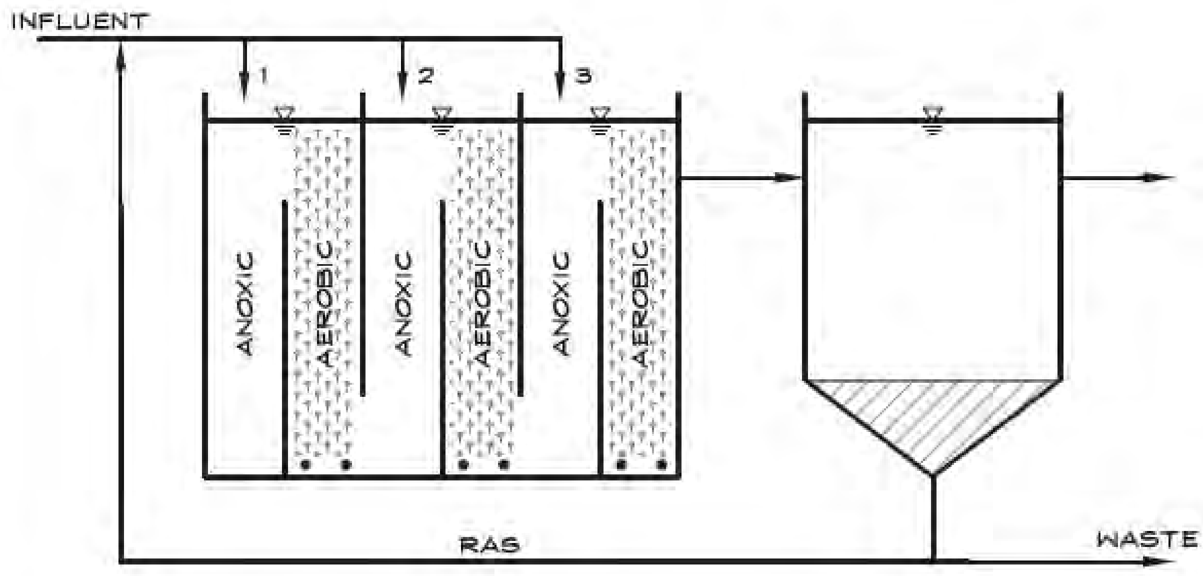

Figure 10: Step Feed

6.2.F On/Off Aeration

On/off aeration is another variation of the anoxic/aerobic conditions configuration, only the entire aeration basin is converted between these two zones. Mixing during the anoxic conditions must be provided. This configuration is a good candidate for existing plants that must be retrofitted to a BNR system and also have existing capacity sufficient to achieve the necessary residence times for nitrification to occur (7). TN concentrations in the effluent can range from 3 – 10 mg/L (7). On/off aeration is not considered in greater detail because it is readily converted from the alternating zones configuration.

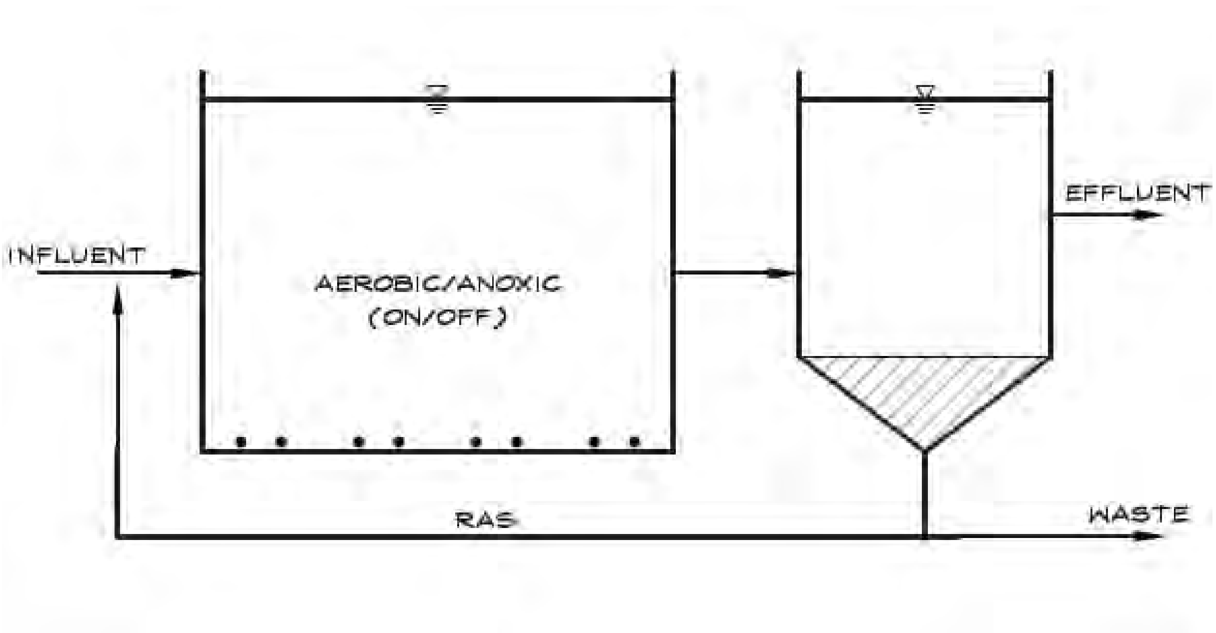

Figure 11: On/Off Aeration

6.2.G Denitrifying Filters

Denitrifying filters are often used in effluent polishing scenarios following secondary wastewater treatment. Nitrified effluent from the secondary clarifier (or aeration basin, if no secondary clarifier exists) is passed through the filter, which may consist of granular or synthetic materials, under anoxic conditions where denitrification can occur. The secondary effluent typically contains very little carbon and consequently carbon addition is often required for the filter process, resulting in additional sludge production (7). Depending upon the filter and existing topography, additional pumping capacity may be required to provide flow through the denitrifying filter. Denitrifying filters are good candidates for existing mechanical plants that need to be retrofitted into BNR systems or where effluent polishing is required. Average effluent TN concentrations less than 4 mg/L are typical of denitrifying filters (7).

The Triple Point system that the City is trying on a pilot scale includes a denitrifying filter. Denitrifying filters are not considered further because of increased capital costs as well as the need for a supplemental carbon source which increases chemical costs and sludge production, as well as additional O&M costs.

6.2.H IFAS & MBBR

Both the integrated fixed film activated sludge (IFAS) and moving bed bio-reactor (MBBR) utilize increased surface area via suspended buoyant media to provide for increased bacteriological attachment sites to host both nitrification and denitrification processes. While additional energy is required to mix the ML with the media in place, this configuration allows for generally a smaller plant footprint which is the only real advantage to using this technology. The IFAS and MBBR are good candidates for retrofitting existing mechanical plants into BNR systems where the existing plant is at or near capacity already. Typical TN concentrations are similar to an MLE configuration (4). IFAS & MBBRs are not considered further because similar effluent nitrogen concentrations can be achieved with less intensive mixing energy that is needed to keep the media mixed.

6.2.I Membrane Bioreactors

Membrane bioreactors generally replace secondary clarifiers and are typically located at the end of the aeration basin in an MLE or alternating zone configuration or may also be installed as a separate facility downstream of the aeration basin. Depending on the particular membrane system in place filtered wastewater either exits the membrane or enters the membrane while filtered solids remain on the opposite side. Typically pumping effort is required to pass wastewater through the membrane, which can increase operational costs (7). Biofouling of the membrane presents an increased operational burden and the membrane must either be cleaned (descaled) or replaced (7), with typical membrane lifespans averaging about 10 years (4). The primary advantages of the membrane bioreactor are that tank volume can be reduced and the need for secondary clarification is eliminated (4). Membrane bioreactors are good candidates for plants with limited footprints or very low TN permit limits. Typical effluent TN concentrations may be as low as 3 mg/L or less (7).

The membrane bioreactor's major benefit is that a secondary clarifier is not required for this configuration. Further, filamentous bacteria, which can cause sludge settling and operational problems, are not an issue since they are retained by the membrane (4). However, the need for chemical cleaning and additional

O&M costs, as well as the relatively short (10 year) design life of most membranes, this configuration is not recommended as a preferred alternative.

6.3 A Note on Sludge

Sludge is at the heart of the BNR process. More commonly referred to as *activated sludge*, it is used to describe both the treatment process that occurs within as well as the composition of the ML, which generally consists of water (environment), food (BOD, ammonia), and various microorganisms (residents). Microorganisms consume BOD and ammonia in wastewater to produce the energy which is required for cellular growth and reproduction. Energy comes from the microorganism's ability to move electrons, which requires both an electron source, e.g. – reduced carbon or ammonia, as well as an electron acceptor like oxygen or nitrates (4). The process of reproduction and cellular growth, while reducing contaminants in the water, also produces excess material – sludge, which must ultimately be removed from the system in order to maintain a mass balance and effective treatment, especially for nutrient removal.

The sludge to be removed from the system is often referred to as waste activated sludge (WAS), or simply "waste." Waste sludge is generally delivered to solids handling which, at a minimum, consists of dewatering and consolidation, and ultimately disposal in some fashion. Depending on the sludge quality, disposal may consist of placement in landfills or, if additional sludge digestion is provided, potentially land application for agricultural use. It is important to note that all of the BNR processes described above include waste production which necessitates that any WRRF utilizing a BNR process include facilities for and operation of a solids handling process. Wherever supplemental carbon is required in the BNR process – e.g. denitrifying filters, and other effluent 'polishing' configurations, there will be excess production of waste in addition to the normal waste sludge production from the aeration basin, which must also be accounted for in both capital and operational resources. To the extent possible, if an effluent polishing configuration is employed, effort should be made to use raw influent in lieu of a supplemental carbon source (e.g. methanol), to reduce material costs and excess sludge production.

6.3.A Clarification

Near the end of the treatment process it is necessary to provide for the separation of the sludge and effluent via clarification. Clarification involves allowing the sludge to settle under gravity thereby allowing for clear effluent to be decanted from the system. This process is most commonly afforded by a secondary clarifier, which acts as a stilling basing with a central baffle to promote the settling of sludge. Settled sludge can then be removed from the bottom of the clarifier via pumps (or in some instances by gravity) and distributed either as RAS or waste. With the exception of the SBR configuration, in which a single basin provides both reaction and clarification, and membrane bioreactors that perform clarification, all of the above described BRN process configurations require secondary clarification, including those configurations for lagoon conversion.

7 Alternatives

Each of the alternatives described below have been preliminarily designed as two parallel treatment trains, with each train sized to treat one-half of the combined future flows of the District and the City, or 1.23 MGD. Having two parallel trains will allow one treatment train to be offline for standby conditions or to facilitate maintenance. When one train is offline the remaining train may be overloaded but will still provide substantial treatment. The use of two treatment trains is generally in accordance with standards for redundancy. At current flow rates with both treatment trains in service there will be excess capacity.

It is worth noting that the cost estimates prepared for each alternative are inclusive of constructing two parallel treatment trains. The cost to construct one larger treatment train would be less than two smaller treatment trains but may not provide for adequate redundancy.

Lagoon conversion is an attractive option because it allows use of the existing ponds at sizable economic savings over constructing new reinforced concrete basins. Generally, capital cost savings for construction of a lagoon conversion may be as high as 40±% over an equivalent sized mechanical plant. As it is suspected that the existing clay liner in the ponds may be leaking or prone to future leakage and the aeration and mixing could cause erosion of the clay bottom; it is prudent that any lagoon conversion include lining the aeration basins with HDPE liners. The detailed design would identify preferred locations for the aeration basin, either in the District's existing pond or the City's existing Pond 1, as preliminarily chosen in this feasibility report

A new mechanical plant, at the tradeoff of greater capital expense, will generally offer better operational control, ease of maintenance, and potentially a longer design life. Secondary clarification, being necessary for all of the alternatives presented, is described in detail at the end of the three alternatives. The preliminary design for secondary clarifiers is the same for all three alternatives. The proposed process diagram is similar for all three alternatives and is shown on Figure 12.

7.1 Alternative A – Lagoon Conversion via Alternating Zones

The alternating zones configuration is essentially a modified version of the extended aeration activated sludge process. Influent is fed to the front of the aeration basin and travels through multiple zones of aerobic and anoxic conditions created by alternating each successive set of submerged aerators between on and off. Mixing, which keeps solids in suspension, is also provided by the aerators in addition to achieving desired dissolved oxygen concentrations. As the mixed liquor passes through an aerobic zone, nitrification takes place converting ammonia into nitrates. As the nitrified mixed liquor enters an anoxic zone, denitrification is allowed to occur. In this fashion, it is possible to avoid the need for mixed liquor recirculation, which provides both capital as well as operational and maintenance cost savings over other extended aeration processes designed for nitrogen reduction.

The geometry and position of the City of Bishop's Pond No.1 make this a good location for a new aeration basin. However, because of high ground water in the area, the required depth of the new basins, and the existing elevations of upstream and downstream components; the new basins must be elevated and a pump station is required.

The alternating zones configuration approximates a plug flow reactor with good mixing. This allows for a higher suspended solids concentration in the mixed liquor (MLSS) which serves to reduce the necessary footprint by a significant amount. Two identical aeration basins consisting of 1.2 million gallons of volume, having three to one side slopes, a ten foot side water depth with two to three feet of freeboard, and top water

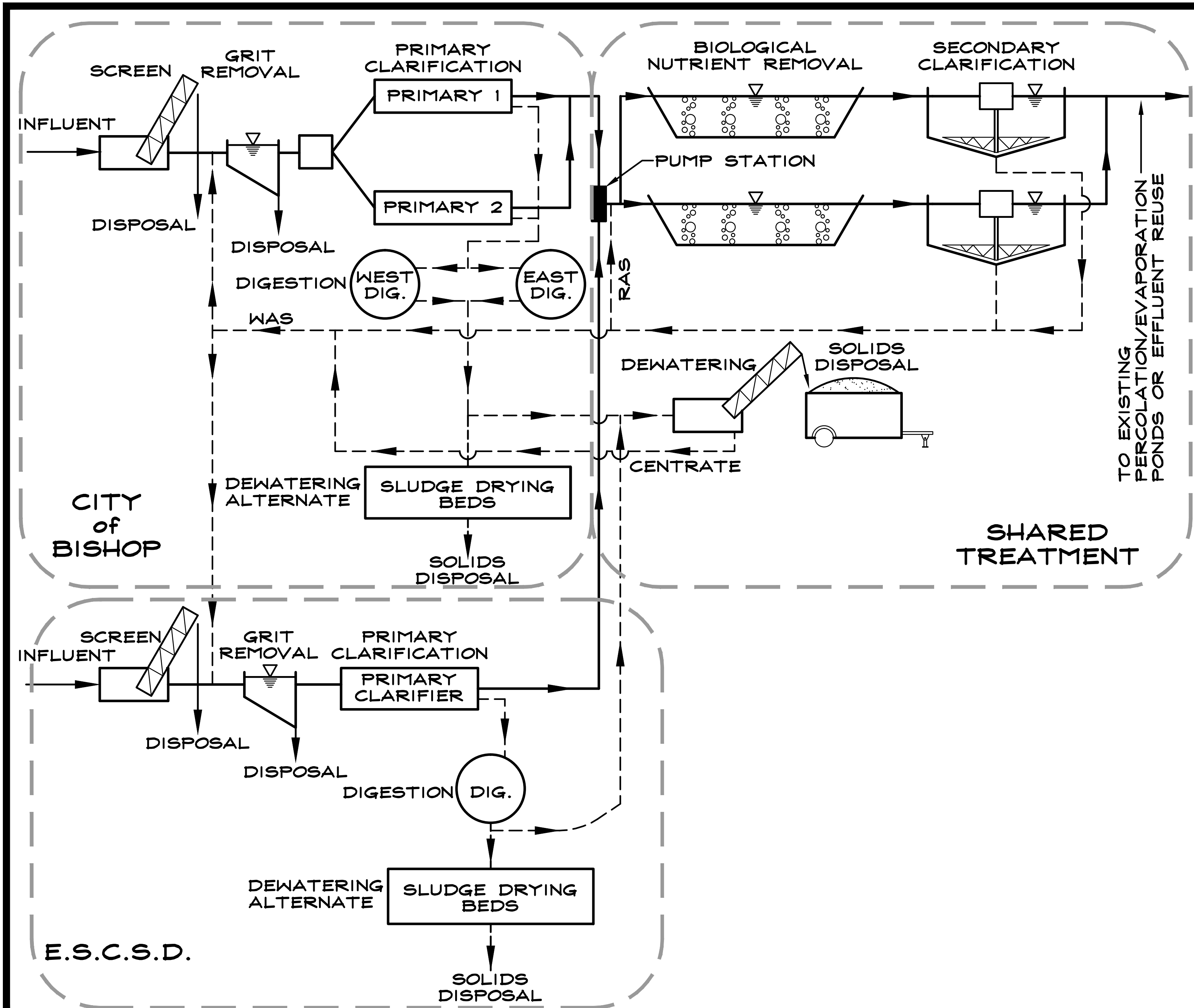

FIGURE 12

PROPOSED PROCESS DIAGRAM

City of Bishop/E.S.C.S.D.

1621-005

09/29/15

R|O|Anderson
WWW.ROANDERSON.COM

NEVADA
1603 Esmeralda Ave
P.O. Box 2229
Minden, NV 89423
p 775.782.2322

CALIFORNIA
595 Tahoe Keys Blvd
Suite A-2
South Lake Tahoe, CA 96150
p 530.600.1660

dimensions of 260 feet by 100 feet will readily accommodate the future design flow of 2.45 MGD, as well as offer capacity for peak flows. The current combined average daily flow of 1.45 MGD may be treated in either a single basin or evenly split between the two basins, with the latter approach more readily achieving the target SRT values.

A design MLSS concentration of 3,000 mg/L will result in a food to mass (FM) ratio of about 0.06; however, MLSS concentrations of up to 6000 mg/L can be accommodated. A target solids retention time (SRT) of 20 days will readily be achieved and allow for thorough nitrification and denitrification. Providing this longer SRT, in addition to a hydraulic residence time (HRT) of around 31 hours also allows for large fluctuations in flow and solids loading, including both diurnal loadings typical of municipal wastewater collection as well as less frequent times of storm water inflow and infiltration. Effluent quality of BOD_5 and TSS less than 20mg/L each, along with a total nitrogen of 10 mg/L or less should also be readily achievable.

RAS flow rates for extended aeration typically range between 50% and 150% of influent flows, and a design RAS flow rate of 100% of average daily influent flows (1,000 GPM) has been preliminarily selected. Sludge wasting from the secondary clarifiers will generally occur at a flow rate of up to 3% of average daily flow as necessary to maintain desired sludge blanket depths in the clarifier and high mean cell residence times (MCRT).

The maximum air flow required has been conservatively estimated at about 2,500 standard cubic feet per minute (SCFM), which will require about 85 to 90 HP of blower capacity. This capacity can be met by providing three each 30 HP blowers to be operated in a lead, lag, lag-lag configuration. An average airflow requirement of 1,865 SCFM has been estimated. The average and maximum oxygen requirements are estimated at 6,700 and 8,900 lbs/day, respectively.

The preliminary estimate of probable construction costs for this alternative is presented in Table 1. It is noted that this estimate is only for construction and the design, construction administration, and inspection is expected to be approximately 25% of the construction cost. The layout presented in Figure 13.

Table 1 - Alternating Zones Preliminary Estimate of Probable Costs

ENGINEER'S PRELIMINARY ESTIMATE OF PROBABLE CONSTRUCTION COSTS

Client:	**CITY OF BISHOP & ESCSD**			**Estimated:**		**JEL**
Project:	**Joint Treatment Feasibility Assessment**			**Checked:**		
Description:	**Alternating Zones Cost Estimate**			**Date:**		**5-Oct-15**
File:	Y:\Client Files\1621\1621-005\Documents\Joint Treatment Feasibility Study\[Engineering Design.xlsx]MLE Life Cycle Costs					
DIVISION 1 - GENERAL REQUIREMENTS						
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		**TOTAL**
1	Mobilization, Demobilization, BMPs (7.5% of construction costs)	1	Lump Sum	7.5%	/LS	$268,658
				SUB TOTAL		**$268,658**
DIVISION 2 - SITE CONSTRUCTION						
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		**TOTAL**
1	Dewatering & Sludge Removal (west portion) of Existing COB Pond No. 1	1	Lump Sum	$20,000.00	/LS	$20,000
2	Earthwork for Aeration Basins and Clarifiers	23,000	Cubic Yards	$10.00	/CY	$230,000
3	Demolition & Abandonment	1	Lump Sum	$10,000.00	/LS	$10,000
				SUB TOTAL		**$260,000**
DIVISION 3 - CONCRETE						
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		**TOTAL**
1	Clarifier Concrete	340	Cubic Yards	$800.00	/CY	$272,000
2	Pump Station Concrete	55	Cubic Yards	$800.00	/CY	$44,000
				SUB TOTAL		**$316,000**
DIVISION 4 MASONRY						
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		**TOTAL**
1	CMU Blower/Controls Building	1200	Square Feet	$150.00	/SF	$180,000
				SUB TOTAL		**$180,000**
DIVISION 5 - METALS						
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		**TOTAL**
1	Misc. Metals (Railings, Fences, Covers, Hatches, Etc.)	1	Lump Sum	$50,000.00	/LS	$50,000
2	Metal Centrifuge Building	1000	Square Feet	$100.00	/SF	$100,000
				SUB TOTAL		**$150,000**
DIVISION 6 WOOD & PLASTICS						
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		**TOTAL**
1	HDPE Top Liner	68,300	Square Feet	$1.00	/SF	$68,300
2	HDPE Bottom Liner & System	68,300	Square Feet	$1.50	/SF	$102,450
3	Air Supply Piping	800	Lineal Feet	$40.00	LF	$32,000
4	RAS Piping	800	Lineal Feet	$60.00	LF	$48,000
5	WAS Piping	1,100	Lineal Feet	$60.00	LF	$66,000
6	Primary & Clarified Effluent Piping	350	Lineal Feet	$100.00	LF	$35,000
				SUB TOTAL		**$351,750**
DIVISION 9 - FINISHES						
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		**TOTAL**
1	High Performance Coatings	1	Lump Sum	$25,000.00	/LS	$25,000
				SUB TOTAL		**$25,000**
DIVISION 11 - EQUIPMENT						
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		**TOTAL**
1	Clarifier Equipment	2	Each	$250,000.00	/EA	$500,000
2	Aeration Equipment (Blowers, Diffusers, Etc.)	2	Each	$350,000.00	/EA	$700,000
3	Pump Station Pumps	3	Each	$30,000.00	/EA	$90,000
4	Centrifuge	1	Each	$340,000.00	/EA	$340,000
				SUB TOTAL		**$1,630,000**
DIVISION 13 - SPECIAL CONSTRUCTION						
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		**TOTAL**
1	Bypass During Construction	1	Lump Sum	$15,000.00	/LS	$15,000
				SUB TOTAL		**$15,000**
DIVISION 15 - MECHANICAL						
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		**TOTAL**
2	Misc. Valves & Connections	1	Lump Sum	$60,000.00	/LS	$60,000
				SUB TOTAL		**$60,000**
DIVISION 16 - ELECTRICAL						
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		**TOTAL**
1	10% of Construction Costs	1	Lump Sum	10%	/LS	$298,775
2	Emergency Generator (125KW +/-)	1	Lump Sum	$125,000	/LS	$125,000
				SUB TOTAL		**$423,775**
DIVISION 17 - CONTROLS						
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		**TOTAL**
1	5% of Construction Costs	1	Lump Sum	5.0%	/LS	$170,576
				SUB TOTAL		**$170,576**
				CONSTRUCTION SUB TOTAL		**$3,850,800**
				CONTINGENCY AT 15%[1]		**$577,600**
ENGINEERS PRELIMINARY ESTIMATE OF PROBABLE COSTS						**$4,428,400**

[1] Contingency is for uncertainties as a full design has not yet been completed.

NO.	DATE	REVISION BLOCK	BY

SCALE: 1" = 40'

R|O|Anderson
1603 ESMERALDA AVENUE / POST OFFICE BOX 2229
MINDEN, NEVADA 89423
PHONE: (775) 782-2322 / FAX: (775) 782-7084
WEB SITE: WWW.ROANDERSON.COM

JOINT TREATMENT & NUTRIENT REMOVAL
CITY OF BISHOP AND ESCSD

ALTERNATIVE A
ALTERNATING ZONES
FIGURE 13

DRAWN: MAB	JOB: 1621-005
ENGINEER: JEL	DRAWING: SEE PLOT STAMP
SCALE: 1"=40'	SHEET: ALT A
DATE: 10/01/2015	OF: SHEETS

7.2 Alternative B –Lagoon Conversion via MLE

The Modified Ludzack-Ettinger method for lagoon conversion is very similar to the alternating zones method of lagoon conversion. The geometry, size, and capacity of the new aeration basins will be the same as the alternating zones alternative. The primary difference with the MLE is the process configuration and approach for providing nitrified mixed liquor to the anoxic zone where denitrification can occur. In the MLE, an anoxic zone is located at the front of the aeration basin, with aerobic zones following. A baffle or curtain may be used to separate the two zones to increase the effectiveness of the anoxic zone. It is necessary to ensure mixing of the anoxic zone to maintain suspension of the MLSS, which has been similarly designed for a concentration of 3,000 mg/L. Mixing is most often achieved by submersible mixers or recirculating pumps and it is important that the mixers do not promote aeration of the anoxic zone. A target SRT of 20 days and FM ratio of 0.5 has also been preliminarily selected for sizing of the reactor, and oxygen and blower capacity requirements will be similar to that of the alternating zones alternative.

The other major difference between the MLE and the alternating zones is the need for recycling of the mixed liquor from near the end of the aeration basin back to the head of the aeration basin (the anoxic zone). This is because well nitrified mixed liquor is only found near the end of the aeration basin after having been sufficiently aerated. Once nitrification has occurred, denitrification can only begin in the absence of oxygen, which is only provided for at the head of the aeration basin in the anoxic zone. Therefore, recycling of the well nitrified mixed liquor to the anoxic zone must be provided for. A well-documented correlation between the recycle rate of flow, often listed as a multiple of the influent flow – e.g. 2Q to 4Q, and the amount of corresponding denitrification of the mixed liquor, indicates that denitrification is generally limited to about 82% occurring at a rate of five (5) times the influent flow, or 5Q. However, a recycle rate of 4Q is most often recommended as it will generally yield a denitrification rate about 80% and represents the approximate economic limit of recycle flow rate. Recycle must be accomplished via pumping using high capacity low head pumps. Two pumps, one per treatment train, having an output of up to approximately 4,000 GPM, are proposed for this design.

The advantage of the MLE over the alternating zones configuration is that recycling of the mixed liquor affords some additional control over the operation of the process. Fine tuning of the recycle can provide for the least amount of recycle pumping (reduced pumping cost) that will still achieve permit compliance and effluent quality. The recycle pumping also provides for some additional mixing of the aeration basin which would otherwise be accomplished solely by the diffused aerators in the alternating zones configuration. The obvious disadvantage of the MLE, when compared to the alternating zones alternative, is that the recycle pumping and mixing requirements will have additional operational and maintenance costs throughout the life of the project, in addition to the higher capital cost associated with more equipment.

The preliminary estimate of probable construction costs for this alternative is presented in Table 2. It is noted that this estimate is only for construction and the design, construction administration, and inspection is expected to be approximately 25% of the construction cost. The layout presented in Figure 14.

Table 2 - MLE Reactor Preliminary Estimate of Probable Costs

ENGINEER'S PRELIMINARY ESTIMATE OF PROBABLE CONSTRUCTION COSTS ROA

Client:	**CITY OF BISHOP & ESCSD**			**Estimated:**	**JEL**	
Project:	**Joint Treatment Feasibility Assessment**			**Checked:**		
Description:	**MLE Cost Estimate**			**Date:**		**5-Oct-15**
File:	Y:\Client Files\1621\1621-005\Documents\Joint Treatment Feasibility Study\[Engineering Design.xlsx]MLE Life Cycle Costs					
DIVISION 1 - GENERAL REQUIREMENTS						
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		**TOTAL**
1	Mobilization, Demobilization, BMPs (7.5% of construction costs)	1	Lump Sum	7.5%	/LS	$299,626.03
				SUB TOTAL		**$299,626**
DIVISION 2 - SITE CONSTRUCTION						
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		**TOTAL**
1	Dewatering & Sludge Removal (west portion) of Existing COB Pond No. 1	1	Lump Sum	$20,000.00	/LS	$20,000
2	Earthwork for Aeration Basins and Clarifiers	23,000	Cubic Yards	$10.00	/CY	$230,000
3	Demolition & Abandonment	1	Lump Sum	$10,000.00	/LS	$10,000
				SUB TOTAL		**$260,000**
DIVISION 3 - CONCRETE						
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		**TOTAL**
1	Clarifier Concrete	340	Cubic Yards	$800.00	/CY	$272,000
2	Pump Station Concrete	55	Cubic Yards	$800.00	/CY	$44,000
				SUB TOTAL		**$316,000**
DIVISION 4 MASONRY						
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		**TOTAL**
1	CMU Blower Building	1200	Square Feet	$150.00	/SF	$180,000
				SUB TOTAL		**$180,000**
DIVISION 5 - METALS						
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		**TOTAL**
1	Misc. Metals (Railings, Fences, Covers, Hatches, Etc.)	1	Lump Sum	$50,000.00	/LS	$50,000
2	Metal Centrifuge Building	1000	Square Feet	$100.00	/SF	$100,000
				SUB TOTAL		**$150,000**
DIVISION 6 WOOD & PLASTICS						
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		**TOTAL**
1	HDPE Top Liner	68,300	Square Feet	$1.00	/SF	$68,300
2	HDPE Bottom Liner & System	68,300	Square Feet	$1.50	/SF	$102,450
3	Recirculation Return Piping	900	Lineal Feet	$125.00	LF	$112,500
4	Air Supply Piping	800	Lineal Feet	$40.00	LF	$32,000
5	RAS Piping	800	Lineal Feet	$60.00	LF	$48,000
6	WAS Piping	1,100	Lineal Feet	$60.00	LF	$66,000
7	Primary & Clarified Effluent Piping	350	Lineal Feet	$100.00	LF	$35,000
				SUB TOTAL		**$464,250**
DIVISION 9 - FINISHES						
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		**TOTAL**
1	High Performance Coatings	1	Lump Sum	$25,000.00	/LS	$25,000
				SUB TOTAL		**$25,000**
DIVISION 11 - EQUIPMENT						
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		**TOTAL**
1	Clarifier Equipment	2	Each	$250,000.00	/EA	$500,000
2	Aeration Equipment (Blowers, Diffusers, Etc.)	2	Each	$350,000.00	/EA	$700,000
3	Recirculation Return Manifold	2	Each	$30,000.00	/EA	$60,000
4	Recirculation Mixers for Anoxic Zone	4	Each	$15,000.00	/EA	$60,000
5	Pump Station Pumps	3	Each	$30,000.00	/EA	$90,000
6	Centrifuge	1	Each	$340,000.00	/EA	$340,000
				SUB TOTAL		**$1,750,000**
DIVISION 13 - SPECIAL CONSTRUCTION						
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		**TOTAL**
1	Bypass During Construction	1	Lump Sum	$50,000.00	/LS	$50,000
				SUB TOTAL		**$50,000**
DIVISION 15 - MECHANICAL						
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		**TOTAL**
1	Hi-Rate Recirculation Submersible Pumps	2	Each	$45,000.00	/EA	$90,000
2	Misc. Valves & Connections	1	Lump Sum	$60,000.00	/LS	$60,000
				SUB TOTAL		**$150,000**
DIVISION 16 - ELECTRICAL						
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		**TOTAL**
1	10% of Construction Costs	1	Lump Sum	10%	/LS	$334,525
2	Emergency Generator (125KW +/-)	1	Lump Sum	$125,000	/LS	$125,000
				SUB TOTAL		**$459,525**
DIVISION 17 - CONTROLS						
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		**TOTAL**
1	5% of Construction Costs	1	Lump Sum	5%	/LS	$190,238.75
				SUB TOTAL		**$190,239**
				CONSTRUCTION SUB TOTAL		**$4,294,600**
				CONTINGENCY AT 15%[1]		**$644,200**
ENGINEERS PRELIMINARY ESTIMATE OF PROBABLE COSTS						**$4,938,800**

[1] Contingency is for uncertainties as a full design has not yet been completed.

NO.	DATE	REVISION BLOCK	BY

SCALE: 1" = 40'

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JOINT TREATMENT & NUTRIENT REMOVAL
CITY OF BISHOP AND ESCSD

ALTERNATIVE B
MLE REACTOR
FIGURE 14

DRAWN: MAB	JOB: 1621-005
ENGINEER: JEL	DRAWING: SEE PLOT STAMP
SCALE: 1"=40'	SHEET: ALT B
DATE: 10/01/2015	OF: SHEETS

7.3 Alternative C – Mechanical Plant via Oxidation Ditch

The oxidation ditch is also an extended aeration process, with a target SRT of 20 days. The oxidation ditch combines the favorable aspects of both the MLE and alternating zones alternatives in that extremely high rates of internal recycle insure complete mixing and reaction times, and the ability to alternate zones internally within the ditch between anoxic and aerobic conditions can achieve very high levels of nutrient removal including nearly complete nitrogen removal (4). Stream velocities of 0.8 to 1.2 feet per second are necessary to maintain suspension of the MLSS, and this is accomplished via submersible mixers on both ends of the ditch. On average, the mixed liquor may make up to 200 revolutions before exiting the basin. Effluent leaves by spilling over an adjustable weir prior to secondary clarification. Diffused aerators are situated upon the bottom of the ditch and can be turned on and off to provide the necessary metabolic conditions.

The oxidation ditch will have a much smaller footprint, being only 170 feet long and approximately 70 feet wide, while providing 1.05 million gallons of volume per each oxidation ditch. Similar to the other alternatives, two treatment trains in parallel are proposed. A deeper side water depth of 13 feet will allow for more efficient oxygen transfer, however, this comes at the cost of increased blower HP requirements. An estimated 105 HP of blower capacity is necessary to provide the estimated maximum airflow of 2,500 SCFM under existing combined ADF. The deeper and more efficient reactor has also been preliminarily designed for an MLSS concentration of 3,500 mg/L, and FM ration of 0.50.

The oxidation ditch will provide the greatest operational control over the entire process. The deep reactor will be more resistant to cold weather impacts, and high quality effluent is anticipated to be readily achievable year round. The primary disadvantage to the oxidation ditch is the substantially higher cost to construct, which is driven by much larger quantities of reinforced concrete, potential for ground water dewatering during construction, and increased equipment costs.

The preliminary estimate of probable construction costs for this alternative is presented in Table 3. It is noted that this estimate is only for construction and the design, construction administration, and inspection is expected to be approximately 25% of the construction cost. The layout presented in Figure 15.

Table 3 - Oxidation Ditch Preliminary Estimate of Probable Costs

ENGINEER'S PRELIMINARY ESTIMATE OF PROBABLE CONSTRUCTION COSTS

Client:	**CITY OF BISHOP & ESCSD**	**Estimated:**	**JEL**	
Project:	**Joint Treatment Feasibility Assessment**	**Checked:**		
Description:	**Oxidation Ditch Cost Estimate**	**Date:**	**5-Oct-15**	
File:	Y:\Client Files\1621\1621-005\Documents\Joint Treatment Feasibility Study\[Engineering Design.xlsx]MLE Life Cycle Costs			

ITEM	DESCRIPTION	QUANTITY		UNIT COST		TOTAL
DIVISION 1 - GENERAL REQUIREMENTS						
1	Mobilization, Demobilization, BMPs (7.5% of construction costs)	1	Lump Sum	7.5%	/LS	$315,574.88
				SUB TOTAL		**$315,575**
DIVISION 2 - SITE CONSTRUCTION						
1	Dewatering & Sludge Removal (west portion) of Existing COB Pond No. 1	1	Lump Sum	$20,000.00	/LS	$20,000
2	Earthwork for Oxidation Ditches and Clarifiers	31,000	Cubic Yards	$10.00	/CY	$310,000
3	Demolition & Abandonment	1	Lump Sum	$10,000.00	/LS	$10,000
				SUB TOTAL		**$340,000**
DIVISION 3 - CONCRETE						
1	Clarifier Concrete	340	Cubic Yards	$800.00	/CY	$272,000
2	Pump Station Concrete	55	Cubic Yards	$800.00	/CY	$44,000
3	Oxidation Ditch Concrete	475	Cubic Yards	$800.00	/CY	$380,000
				SUB TOTAL		**$696,000**
DIVISION 4 MASONRY						
1	CMU Blower Building	1200	Square Feet	$150.00	/SF	$180,000
				SUB TOTAL		**$180,000**
DIVISION 5 - METALS						
1	Misc. Metals (Railings, Fences, Covers, Hatches, Ditch Weirs, Etc.)	1	Lump Sum	$70,000.00	/LS	$70,000
2	Metal Centrifuge Building	1000	Square Feet	$100.00	/SF	$100,000
				SUB TOTAL		**$170,000**
DIVISION 6 WOOD & PLASTICS						
1	Air Supply Piping	600	Lineal Feet	$40.00	LF	$24,000
2	RAS Piping	300	Lineal Feet	$60.00	LF	$18,000
3	WAS Piping	1,000	Lineal Feet	$60.00	LF	$60,000
4	Primary & Clarified Effluent Piping	250	Lineal Feet	$100.00	LF	$25,000
				SUB TOTAL		**$127,000**
DIVISION 9 - FINISHES						
1	High Performance Coatings	1	Lump Sum	$25,000.00	/LS	$25,000
				SUB TOTAL		**$25,000**
DIVISION 11 - EQUIPMENT						
1	Clarifier Equipment	2	Each	$250,000.00	/EA	$500,000
2	Aeration Equipment and Mixers	2	Each	$550,000.00	/EA	$1,100,000
3	Pump Station Pumps	3	Each	$30,000.00	/EA	$90,000
4	Centrifuge	1	Each	$340,000.00	/EA	$340,000
				SUB TOTAL		**$2,030,000**
DIVISION 13 - SPECIAL CONSTRUCTION						
1	Bypass During Construction	1	Lump Sum	$15,000.00	/LS	$15,000
				SUB TOTAL		**$15,000**
DIVISION 15 - MECHANICAL						
1	Misc. Valves & Connections	1	Lump Sum	$60,000.00	/LS	$60,000
				SUB TOTAL		**$60,000**
DIVISION 16 - ELECTRICAL						
1	10% of Construction Costs	1	Lump Sum	10%	/LS	$364,300
2	Emergency Generator (150KW +/-)	1	Lump Sum	$150,000	/LS	$150,000
				SUB TOTAL		**$364,300**
DIVISION 17 - CONTROLS						
1	5% of Construction Costs	1	Lump Sum	5%	/LS	$200,365.00
				SUB TOTAL		**$200,365**
				CONSTRUCTION SUB TOTAL		**$4,523,200**
				CONTINGENCY AT 15%[1]		**$678,500**
ENGINEERS PRELIMINARY ESTIMATE OF PROBABLE COSTS						**$5,201,700**

[1] Contingency is for uncertainties as a full design has not yet been completed.

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SCALE: 1" = 40'

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ALTERNATIVE C
OXIDATION DITCH
FIGURE 15

DRAWN: MAB	JOB: 1621-005
ENGINEER: JEL	DRAWING: SEE PLOT STAMP
SCALE: 1"=40'	SHEET: ALT C
DATE: 10/01/2015	OF: SHEETS

7.4 Clarifier Design

Two circular clarifiers of 70 foot diameters operated in parallel are proposed for each of the alternatives above. Each clarifier has been preliminarily sized to accommodate half of the future combined peak daily flows such that both clarifiers operating together will be able to handle the future combined average daily flow as well as peak daily flows. Similar to the treatment trains, two smaller clarifiers are proposed instead of one large clarifier to allow for reasonable redundancy. If one clarifier is out of service the other may be used and although overloaded will still provide substantial clarification.

Circular clarifiers are proposed over other configurations as the concentric design typically provides easier operation and maintenance due to less equipment. A clarifier surface loading of 400 gal/day-ft^2 during average daily flows and peak daily flows, along with a maximum of 600 gal/day-ft^2 overflow rate during instantaneous peak flows were assumed, with peak daily flow loading controlling the selection of clarifier diameter. A side water depth of 6 feet has been assumed which will provide up to 3.9 hours of hydraulic detention time at the future average daily flow, and over 6.6 hours of hydraulic detention at existing average daily combined flow. At 4,000 mg/L MLSS entering the clarifier under peak flow conditions, the maximum future peak daily solids loading to the clarifiers is estimated at 181,478 lbs/day, or approximately 24 lbs/day-ft^2. Peak weir loading, occurring during instantaneous peak flow conditions, will be around 9,750 gal/day-ft. A 12:1 bottom side slope will make the clarifier 9 feet deep in the middle, providing each clarifier with an effective volume of approximately 201,500 gallons.

7.5 Solids Handling

As described above, the BNR process is intertwined with both the consumption and production of sludge. Sludge is produced in the process by both the generation of new cells, as well as the accumulation of inert solids, both of which are settled out of the mixed liquor during the clarification process. This excess sludge must be removed from the system to maintain hydraulic capacity and biological health of the activated sludge. Sludge that is removed from the system, once it is dewatered, is generally referred to as biosolids. The production, treatment and disposal of biosolids are aptly termed *solids handling*. It is anticipated that, at least initially, solids generated from the treatment process will be digested, dewatered, and hauled to a landfill for disposal.

Future uses may include, at the discretion of ESCSD and COB, beneficial land application for soil amendments, agricultural use, or even landscaping, depending on the quality of treatment and characteristics of the biosolids produced from the process.

Beneficial use of biosolids is regulated under 40 CFR 503, while biosolids disposal in landfills is separately regulated under Part 258 (11). Landfill applications of biosolids mixed with municipal solid waste is referred to as co-disposal (12). 40 CFR 258 delineates the regulatory requirements for landfill operation where co-disposal occurs, including pollutant limits as well as pathogen and vector control considerations (12). In order for biosolids to be applied at a landfill they generally must have a solids concentration of 18% or greater typically determined through a paint filter test (refer to EPA SW-846), and they must also pass the federal Toxicity Characterization Leaching Procedure (TCLP) and California WET test to determine hazardous waste potential. However, biosolids generally do not have an issue meeting the non-hazardous waste criteria (12).

It is necessary to estimate the amount of solids that will be produced during the BNR process, however, the actual amount of biosolids produced varies greatly depending on any number of process-specific variables and treatment plant idiosyncrasies, such as temperature, influent characteristics, SRT, chemical addition, and recycle streams. As a result, conservative values are used for facility planning and often include large factors of safety. As a general rule, it is expected that the treatment process will produce about 1 dry ton of biosolids per million gallons of water treated. This means that, at a combined average daily flow (ADF) of 1.45 MGD, a yield of 1.45 dry tons per day of biosolids is would be reasonably expected (11). While a detailed mass balance of the entire treatment system will produce a more accurate estimate of the biosolids produced, it is beyond the scope of this report and such an investigation should be undertaken during final design. The estimate may be further refined however, using the following relationships:

$$Primary\ Solids\ Production = Pant\ Flow\ x\ Influent\ TSS\ x\ Removal\ Rate\ (11)$$

Where removal rate (in percent removed) is defined as: $T/(a+bT)$; and

$$T = detention\ time\ (\text{min})$$

$$a = constant, 0.406$$

$$b = constant, 0.0152$$

Using a combined average daily flow of 1.45 MGD, and an estimated influent TSS, the solids produced from primary clarification is expected to be about 1,775 pounds.

Solids produced from secondary treatment are similarly estimated by the kinetic relationship:

$$Y_{obs} = \frac{Y}{1+k_{d}(\theta_{c})} \quad (11)$$

Where Y_{obs} is the observed yield as (lbs. biomass/lbs. of substrate); and

$$Y = yield\ \left(\frac{g - biomass}{g - substrate}\right) estimated\ at\ 0.8, typ.\ 0.4 - 0.8$$

$$k_d = endogenous\ decay\ rate\ \left(\frac{g - biomass}{g - biomass - d^{-1}}\right) estimated\ at\ 0.04, typ.\ 0.04 - 0.075$$

$$\theta_c = SRT\ or\ MCRT\ (days), estimated\ at\ 20, avg.$$

Using the estimated values above, which conservatively produce higher yield estimates, the secondary treatment solids production at 1.45 MGD ADF is estimated to be 1,236 pounds. This estimate can be generally verified by a quick mass balance approach, which is the product of flow and substrate removed. Using an estimated overall BOD_5 removal rate of 91%, including a 30% reduction within the primary clarifier and 20 mg/L effluent concentration, the solids produced during secondary treatment are estimated at 0.44 g-biomass per g-BOD_5 removed, or approximately 836 pounds of biosolids. Since the process will also be removing nitrogen, the solids production must also include the yield for both ammonia removal (nitrification) and nitrate removal (denitrification). The estimated yield rates for nitrification and denitrification are 0.17 g-Biomass per g-NH_3 and 0.8 g-biomass per g-$N0_3$; respectively. Assuming a 98% nitrification rate and influent concentration of 30.0 mg/L NH_3, and an 88% denitrification rate, the corresponding solids yield is estimated at 61 pounds for nitrification and 336 pounds for denitrification, which results in an estimated overall secondary treatment solids yield of 1,233 pounds, which very closely agrees with the estimate produced by the kinetic relationship described above.

Combining both primary and secondary solids production estimates, an overall treatment yield of 3,008 pounds (1.504 tons) of solids per day can reasonably be utilized for the purposes of this report including general facility capacity sizing at current combined ADF. Future daily solids production at buildout, based upon an ADF of 2.45 MGD, is estimated at approximately 4,900 pounds (2.45 tons).

7.5.A Digestion & Dewatering

It is necessary to dewater the sludge removed from the system in order to produce the required solids concentration so that the bioslids may then be co-disposed of in a landfill. Dewatering may also increase the quality of the biosolids, and anaerobic digestion can, under the appropriate loading rates and digester operation, produce Class B biosolids which would be eligible for certain restricted land application and beneficial usage (11).

The combined sludge from the primary clarifiers and secondary treatment process is assumed to have a minimum solids concentration of 4%. Converting the 1.5 dry tons per day expected yield into wet tons, the expected wet yield is 37.5 tons per day, or approximately 1,190 cubic feet per day, assuming a sludge specific gravity of 1.01. Assuming the digesters are operating as complete mix flow through reactors (flow in is equal to flow out), the operational mean cell residence time (MCRT) for the digesters is equivalent to the digester hydraulic residence time, or volume divided by flow. Assuming a 50/50 flow split to the District and the City, with an applied factor of safety of 2.5, the MCRTs are 12 and 13 days for the District and City, respectively. At these residence times, the digesters need to be maintained at a minimum temperature of 18° C (64.4° F) (13). It is expected that, given a higher percentage of volatile suspended solids being contributed to the digesters from the secondary sludge, digester performance and methane gas production will be improved and may offset the need to heat the digesters with supplemental propane or other fuels.

Digested sludge is expected to have a minimum solids content of 6%, with a potential range of up to 8% solids. Digested sludge will need to be further dewatered to a minimum solids concentration of 18%. This additional dewatering can occur in the existing sludge drying beds, which offer a low energy, efficient means of increasing the solids concentration, with solids concentrations of up to 45% being possible (13). Presently, the District's sludge drying beds are operating at capacity (14), while the City is operating at approximately 82% of capacity based upon an estimated sludge drying volume of 8,500 cubic feet. Given the near capacity of sludge drying bed volume, it is recommended that an additional method for dewatering be incorporated, such as a centrifuge or screw press, which could handle the solids loading for the entire system. This is preferred over expanding the sludge drying beds as the dewatering equipment is not subject to wet weather inhibition, would be capable of dewatering un-digested sludge should the digesters fail, and occupies a much smaller footprint.

Modern dewatering equipment is capable of producing biosolids with solids concentrations well in excess of the 18% minimum from a wide range of influent sludge solids concentrations. Further, this equipment can be used to dewater either digested sludge after passing through the digesters or raw sludge.

8 Operations and Maintenance

Operation of an activated sludge BNR system will generally consist of the same process regardless of the system configuration. Since nitrification is the first part of the BNR process, it is necessary to provide Mean Cell Residence Times (MCRT), or "sludge age" that will allow for nitrification to occur. MCRT is typically the number one problem with achieving nitrification (4). Sludge age is critical in order to achieve sufficient communities of organisms that are capable of nitrifying since nitrifiers are generally slow growing organisms. MCRTs will need to be calculated often, likely on a daily basis, and the calculation consists of determining the mass of solids within the aeration basin and clarifier, and dividing by the mass of solids wasted per day. MLSS samples from the activated sludge will be used to determine the mass in the aeration basin by multiplying the concentration by the volume of the basin and converting to pounds, while MLSS samples from the waste line will determine the solids being removed by multiplying the waste solids concentration by the waste flow rate and converting to pounds per day. Solids in the clarifier will be determined by sampling for the sludge blanket depth in the bottom of the clarifier and multiplying the volume of sludge (as determined by blanket depth) by the waste solids concentration and converting to pounds. This will yield the MCRT in days, which can then be used to determine the appropriate amount of solids to be wasted from the system in order to achieve the target MCRT developed during plant operation.

Testing for and maintaining proper pH is also critical for nitrification, and a pH of 7.2 is ideal for the aeration basin (4). As pH decreases, ammonia within the mixed liquor will trend toward the ionized form – ammonium (NH_4^+), which is not available to nitrifying bacteria (nitrifiers) that require NH_3 for consumption. It is expected that, when the combined influent alkalinity is joined with alkalinity produced from denitrification, a sufficient alkalinity buffer (>40 mg/L) will remain in the mixed liquor to prevent the loss of the nitrifier community due to a drop in pH. However, testing the pH will be required to insure that an appropriate level is maintained in the system. In addition, it will be necessary to test alkalinity concentrations from within the aeration basin in order to determine that elevated pH levels are not absent of alkalinity, since high pH can mask the absence of alkalinity in the aeration basin. Another problem with low pH is the fostering of fungal growth within the aeration basin which can cause poor sludge settling due to bulking. To prevent the accumulation of fungal growth, it is important that the aeration basin pH does not drop below 6.5. If pH levels should drop drastically the addition of lime or another alkaline substance can be used to increase the pH.

Dissolved oxygen (DO) concentrations should also be routinely observed at various points within the system. Maintaining adequate DO is necessary to achieve nitrification, however, too much DO will inhibit denitrification. A portable DO probe is recommended for evaluating DO concentrations within the system. Where anoxic conditions are supposed to be occurring, DO levels should be quite low, and where aerobic conditions are supposed to be occurring, DO levels should be measurable at the target level that will be field determined under system optimization. Nitrification requires substantially more oxygen than carbonaceous consumption (BOD removal). However, oxygen transfer efficiency actually increases at lower residuals. For instance, a DO of 0.2 mg/L will have a 29% increase in transfer efficiency over a DO of 2 mg/L (4)[6]. Additionally, DO concentrations between 0.5 and 1.0 mg/L will tend to foster the growth of low DO filamentous bacteria that can cause poor sludge settling. Since this will occur in the anoxic zones where BOD is present, it is important to insure that DO levels are quite low in the anoxic zones to prevent filamentous growth as well as promote denitrification.

While temperature is generally out of the operational control for most treatment plants, it is important to know that low temperatures will affect nitrification. Generally, low temperature will cause ammonia to be present as ammonium, and will also require longer solids retention times (SRT) within the aeration basin in order to achieve

[6] Based upon 1,000 feet of elevation and 20° C, and beta=0.95, C_{sw}=8.9 (4).

nitrification. Understanding the effects of temperature on the system's performance will allow for timely operational modifications in order to continue to maintain nitrification.

8.1 Process Control Testing and Suggested Meters

- MLSS/MLVSS – this test determines the biomass available and is necessary for computation of MCRT. A handheld suspended solids meter is recommended for accomplishing this test. Solids concentrations should be periodically obtained from the aeration basin and RAS/WAS lines.
- DO – the residual (excess) dissolved oxygen concentration available. This test should be periodically performed at various points in the system to determine oxygen concentrations. A handheld optical DO meter is recommended. Permanent DO & TSS meters may also be installed in the aeration basin and connected to the plant SCADA for automatic feedback and process control.
- SOUR – Specific Oxygen Uptake Rate will determine the rate of aerobic oxidation and should be run every day.
- ORP – Oxidation Reduction Potential will determine the degree of oxidation and indicate whether nitrification or denitrification is occurring. A handheld ORP meter is recommended for accomplishing this test; however, integrated in-basin ORP probes are also quite helpful for process feedback.
- Multi-Parameter Probe – While ORP can be used as a proxy for Ammonia and Nitrate concentration tests, an analytical analyzer that can determine various concentrations including ammonia and nitrate levels can offer very accurate information about nutrient concentrations at various points in the system. While not mandatory, a multi-parameter test probe is recommended.
- Flow Rate – Flow rates for RAS, WAS, influent and mixed liquor recycle (if part of the process) are all important data for process control. While the influent flow cannot typically be controlled, RAS, WAS, and mixed liquor recycle can all be varied as necessary for fine tuning the process. Automatic flow meters which are integrated with the plant SCADA will offer substantial benefit for operational control. It is expected that RAS flow will

generally be varied by the operators in conjunction with influent flow from 75% to 100% of influent flow, as necessary to achieve MLSS and MCRT targets. Sludge will also be periodically wasted in order to achieve a solids balance within the system and to maintain MLSS and MCRT targets as well. It is expected that WAS flows will at most be 3% of influent flow, however, the exact amount of WAS flow will need to be determined during plant operation. Recycle of the mixed liquor, if included, will vary from 2 to 4 times the influent flow, with higher rates of denitrification occurring near the 4Q flow rate – assuming no DO is present in the recycle.

- Microscopic Analysis – the use of a lab microscope to inspect the content of the activated sludge is recommended. Microscopic analysis will be helpful to determine some sludge settling issues due to the presence of fungi or filamentous bacteria, as well as other indicators of sludge health and biodiversity.

8.2 Secondary Clarification and Dewatering

All of the proposed alternatives contemplate the same design and approach for secondary clarification and dewatering. Operation and maintenance of the secondary clarifiers will include general preventative maintenance and repairs to the clarifier drive and rake assembly, as well as periodic cleaning of the baffle and weirs to prevent the accumulation of algae which can artificially increase effluent nitrogen concentrations. Maintaining a consistent sludge blanket depth will require periodic samples using a sludge depth indicator or an ultrasonic transducer, although the latter is substantially more expensive. Every other year it is expected that one clarifier will be taken offline for inspection of the clarifier walls, submerged baffles and rake assembly, and to accommodate any repairs or modifications determined necessary during those inspections.

Dewatering will, depending on the type of equipment chosen, require varying levels of preventative maintenance and repairs. Specialty dewatering equipment such as a centrifuge will likely require contract maintenance from the manufacturer which can also provide dewatering optimization guidance during maintenance visits. Other equipment, such as a screw press, will require less specialized maintenance and will be similar to other mechanical processes with respect to preventative maintenance and repairs. The addition of polymer will be required to optimize dewatering energy and solids concentrations; however, polymer addition quantities will have to be field determined. It is expected that polymer will be injected from a storage container into the waste sludge upstream of the dewatering equipment automatically via a chemical dosing pump that is flow matched to the WAS flow meter.

8.3 Alternative A – Lagoon Conversion via Alternating Zones O&M

Operation of the alternating zones system will generally consist of the description above, with the exception of recycle of the mixed liquor which is not included in this process configuration. It will be necessary to determine under field conditions the appropriate amount of DO necessary to achieve nitrification within the aerobic zones, while spacing between the aerators will need to be ultimately determined under final design to insure that true anoxic conditions can be achieved for denitrification in each successive anoxic zone. Primary control will be achieved in varying RAS and WAS flow rates, DO concentrations, and which aeration lines will be alternated and the duration of such alternating.

Maintenance will include preventative maintenance and repairs on influent, RAS, and WAS pumps, clarifier drives and rake assemblies, blowers and automated valves. The aerators themselves will likely need to be cleaned on an annual basis which, depending on the manufacturer's recommendations may include air scour with an acid or descaling with an acid bath. Wearable parts on all components will need to be periodically replaced according to manufacturer's recommendations or as determined necessary by field performance.

8.4 Alternative B –Lagoon Conversion via MLE O&M

Operation and maintenance of the MLE will be essentially the same as the alternating zones except that additional preventative maintenance and repairs will be necessary on the recycle pumps. Recycle flow rates will also be included in the primary control strategy and will vary from between 200% and 400% of the influent flow rate.

8.5 Alternative C – Mechanical Plant via Oxidation Ditch O&M

In general, the operation and maintenance of the oxidation ditch will be similar to the two other alternatives. Submersible mixers, rather than mixed liquor recycle pumps, will provide both mixing and recycle of nitrified elements to the anoxic zones, and will be a primary operational control mechanism. The duration and location of aerobic conditions will be alternated by operators as necessary to achieve target DO conditions and insure that nitrification and denitrification are occurring, which will be greatly enhanced by ORP probe readings. In addition to submersible mixers, effluent will be delivered from the aeration basin via an adjustable weir. Period inspection and maintenance of the weir will also be required in this alternative.

9 Life Cycle Costs

It is prudent to compare alternatives based upon life cycle costs which are the present value of the cost of an alternative throughout its useful life. This includes the capital cost as well as future operation, maintenance, rehabilitation and replacement costs that are discounted to present value based upon an assumed discount rate. The life cycle is selected as the expected life of the longest lived components of the system. In this case it is the life of concrete structures, buildings and pipelines that are assumed to be 50 years. The discount rate used in this analysis is 1.4% which is the expected real interest after accounting for inflation[7].

Tables 5, 6, and 7 calculate the life cycle cost of the three alternatives. The unit costs and much of the criterion are estimated from experience, various vendors and references. The estimated electrical energy use is calculated based upon typical operational parameters for the specific process.

Life cycle costs consider only the additional treatment to remove nitrogen and not existing operations such as screening, primary treatment, or collection system maintenance. The estimated cost of the major expenses associated with each alternative is listed then the

[7] From Office of Budget and Management, Circular A-94, Appendix C, Revised December 2014

future expenses are calculated based upon the expected future flow rates and discounted based upon the discount rate.

For example, 35 years from the start of the new treatment the flow is expected to be 2.15 MGD (starting flow of 1.58 MGD with 0.881% annual increase). This is expected to require 12 hours of labor per MGD per day or 9,417 hours per year (12 x 2.15 x 365). At $50 per hour this will be $470,850. Now because the real interest (interest after inflation is considered) is 1.4% the present value of this future amount is 61.5% or $289,437. A similar calculation is done for all expenses for all years of the life cycle and the present values are summed to determine the present value of the alternatives. The life cycle cost of the three alternatives is summarized in Table 4 below.

Table 4 – Summary of Life Cycle Costs

Alternative	**50 Year Life Cycle Cost**
A – Treatment by Alternating Zones	$30,539,339
B – Treatment by MLE Process	$31,737,360
C – Treatment by Oxidation Ditch	$32,706,625

Table 5 - Alternating Zone Life Cycle Costs

ENGINEER'S PRELIMINARY ESTIMATE OF LIFE CYCLE COSTS						
Client:	**CITY OF BISHOP & ESCSD**			**Estimated:**		**KRN**
Project:	**Joint Treatment Feasibility Assessment**			**Checked:**		
Description:	**Alternating Zones Life Cycle Cost Estimate**			**Date:**		**5-Oct-15**
File:	**Y:\Client Files\1621\1621-005\Documents\Joint Treatment Feasibility Study\[Engineering Design.xlsx]MLE Life Cycle Costs**					
GENERAL						
Life Cycle		50	Years			
ADF Year 0		1.58	MGD			
ADF Year 50		2.45	MGD			
Annual Increase in Flows		0.881%				
Real Interest Rate (from Office of Management and Budget)		1.40%				
UNIT COSTS						
Detailed Design, Contract Administration, Inspection		25.00%	Of Construction Costs			
Energy Cost		$ 0.14	/kWh			
Labor, Including Benefits		$ 50	/Hr.			
Sludge Disposal Including Transportation and Polymer for Thickening		$ 130	/Dry Ton			
Laboratory Compliance Testing		$ 400	/Test			
CRITERION / BASIS						
Additional Labor For Operation		12	Hours Per MGD Per Day			
Sludge Production (Dry Ton)		1.0	Tons/MGD/Day			
Maintenance and Repairs		1%	of Construction Cost per Year			
Electrical Energy		113	kWh/MGD/Day			
Laboratory Testing		1	Test Per Month			
LIFE CYCLE COSTS						

ITEM	DESCRIPTION	CRITERION / BASIS: QUANTITY		UNIT COST		PRESENT VALUE OF 50 YEAR LIFE
1	Construction	1	Lump Sum	$ 4,428,400	LS	$ 4,428,400
2	Detailed Design, Contract Administration, Inspection	25%	Const. Cost	$ 4,428,400	LS	$ 1,107,100
3	Labor	12	Hours Per MGD Per Day	$ 50	/Hour	$ 15,567,336
4	Maintenance and Repairs	1%	Const. Cost	$ 4,428,400	LS	$ 1,584,728
5	Electricity	113	kWh/MGD/Day	$ 0.14	/kWh	$ 410,459
6	Sludge Disposal and Polymer	1	Tons/MGD/Day	$ 130.00	/Dry Ton	$ 3,372,923
7	Laboratory Testing	12	/Year	$ 400.00	/Test	$ 171,771
ITEM	**DESCRIPTION**	**USEFUL LIFE**		**TOTAL COST**	**ANNUAL COST**	**PRESENT VALUE OF 50 YEAR LIFE**
8	Replace: Coatings; Controls	15	Years	$ 195,576	$ 13,038	$ 466,587
9	Replace: Centrifuge; Pumps; Blowers; Valves	20	Years	$ 1,190,000	$ 59,500	$ 2,129,242
10	Replace: Electrical; Emergency Generator; Upper Liner	25	Years	$ 492,075	$ 19,683	$ 704,367
11	Replace: Clarifier Components	30	Years	$ 500,000	$ 16,667	$ 596,426
50 YEAR LIFE CYCLE COST ESTIMATE						**$30,539,339**

Table 6 - MLE Life Cycle Costs

ENGINEER'S PRELIMINARY ESTIMATE OF LIFE CYCLE COSTS				ROA		
Client:	**CITY OF BISHOP & ESCSD**			**Estimated:**		**KRN**
Project:	**Joint Treatment Feasibility Assessment**			**Checked:**		
Description:	**MLE Life Cycle Cost Estimate**			**Date:**		**5-Oct-15**
File:	**Y:\Client Files\1621\1621-005\Documents\Joint Treatment Feasibility Study\[Engineering Design.xlsx]MLE Life Cycle Costs**					
GENERAL						
Life Cycle		50	Years			
ADF Year 0		1.58	MGD			
ADF Year 50		2.45	MGD			
Annual Increase in Flows		0.881%				
Real Interest Rate (from Office of Management and Budget)		1.40%				
UNIT COSTS						
Detailed Design, Contract Administration, Inspection		25.00%	Of Construction Costs			
Energy Cost		$ 0.14	/kWh			
Labor, Including Benefits		$ 50	/Hr.			
Sludge Disposal Including Transportation and Polymer for Thickening		$ 130	/Dry Ton			
Laboratory Compliance Testing		$ 400	/Test			
CRITERION / BASIS						
Additional Labor For Operation		12	Hours Per MGD Per Day			
Sludge Production (Dry Ton)		1.0	Tons/MGD/Day			
Maintenance and Repairs		1%	of Construction Cost per Year			
Electrical Energy		116	kWh/MGD/Day			
Laboratory Testing		1	Test Per Month			
LIFE CYCLE COSTS						
		CRITERION / BASIS				**PRESENT VALUE OF 50 YEAR LIFE**
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		
1	Construction	1	Lump Sum	$ 4,938,800	LS	$ 4,938,800
2	Detailed Design, Contract Administration, Inspection	25%	Const. Cost	$ 4,938,800	LS	$ 1,234,700
3	Labor	12	Hours Per MGD Per Day	$ 50	/Hour	$ 15,567,336
4	Maintenance and Repairs	1%	Const. Cost	$ 4,938,800	LS	$ 1,767,378
5	Electricity	116	kWh/MGD/Day	$ 0.14	/kWh	$ 421,356
6	Sludge Disposal and Polymer	1	Tons/MGD/Day	$ 130.00	/Dry Ton	$ 3,372,923
7	Laboratory Testing	12	/Year	$ 400.00	/Test	$ 171,771
ITEM	**DESCRIPTION**	**USEFUL LIFE**		**TOTAL COST**	**ANNUAL COST**	**PRESENT VALUE OF 50 YEAR LIFE**
8	Replace: Coatings; Controls	15	Years	$ 215,239	$ 14,349	$ 513,496
9	Replace: Centrifuge; Pumps; Blowers; Valves; Mixers	20	Years	$ 1,340,000	$ 67,000	$ 2,397,633
10	Replace: Electrical; Emergency Generator; Upper Liner	25	Years	$ 527,825	$ 21,113	$ 755,541
11	Replace: Clarifier Components	30	Years	$ 500,000	$ 16,667	$ 596,426
50 YEAR LIFE CYCLE COST ESTIMATE						**$31,737,360**

Table 7 - Oxidation Ditch Life Cycle Costs

ENGINEER'S PRELIMINARY ESTIMATE OF LIFE CYCLE COSTS						
Client:	**CITY OF BISHOP & ESCSD**			**Estimated:**		**KRN**
Project:	**Joint Treatment Feasibility Assessment**			**Checked:**		
Description:	**Oxidation Ditch Life Cycle Cost Estimate**			**Date:**		**5-Oct-15**
File:	**Y:\Client Files\1621\1621-005\Documents\Joint Treatment Feasibility Study\[Engineering Design.xlsx]MLE Life Cycle Costs**					
GENERAL						
Life Cycle		50	Years			
ADF Year 0		1.58	MGD			
ADF Year 50		2.45	MGD			
Annual Increase in Flows		0.881%				
Real Interest Rate (from Office of Management and Budget)		1.40%				
UNIT COSTS						
Detailed Design, Contract Administration, Inspection		25.00%	Of Construction Costs			
Energy Cost		$ 0.14	/kWh			
Labor, Including Benefits		$ 50	/Hr.			
Sludge Disposal Including Transportation and Polymer for Thickening		$ 130	/Dry Ton			
Laboratory Compliance Testing		$ 400	/Test			
CRITERION / BASIS						
Additional Labor For Operation		12	Hours Per MGD Per Day			
Sludge Production (Dry Ton)		1.0	Tons/MGD/Day			
Maintenance and Repairs		1%	of Construction Cost per Year			
Electrical Energy		142	kWh/MGD/Day			
Laboratory Testing		1	Test Per Month			
LIFE CYCLE COSTS						
		CRITERION / BASIS				**PRESENT VALUE OF 50 YEAR LIFE**
ITEM	**DESCRIPTION**	**QUANTITY**		**UNIT COST**		
1	Construction	1	Lump Sum	$ 5,201,700	LS	$ 5,201,700
2	Detailed Design, Contract Administration, Inspection	25%	Const. Cost	$ 5,201,700	LS	$ 1,300,425
3	Labor	12	Hours Per MGD Per Day	$ 50	/Hour	$ 15,567,336
4	Maintenance and Repairs	1%	Const. Cost	$ 5,201,700	LS	$ 1,861,458
5	Electricity	142	kWh/MGD/Day	$ 0.14	/kWh	$ 515,798
6	Sludge Disposal and Polymer	1	Tons/MGD/Day	$ 130.00	/Dry Ton	$ 3,372,923
7	Laboratory Testing	12	/Year	$ 400.00	/Test	$ 171,771
ITEM	**DESCRIPTION**	**USEFUL LIFE**		**TOTAL COST**	**ANNUAL COST**	**PRESENT VALUE OF 50 YEAR LIFE**
8	Replace: Coatings; Controls	15	Years	$ 225,365	$ 15,024	$ 537,654
9	Replace: Centrifuge; Pumps; Blowers; Valves; Aerators; Mixers	20	Years	$ 1,590,000	$ 79,500	$ 2,844,953
10	Replace: Electrical; Emergency Generator	25	Years	$ 514,300	$ 20,572	$ 736,181
11	Replace: Clarifier Components	30	Years	$ 500,000	$ 16,667	$ 596,426
50 YEAR LIFE CYCLE COST ESTIMATE						**$32,706,625**

10 Conclusions and Recommendations

It is determined that joint treatment for nutrient removal is possible and the preferred alternative is treatment by the Alternating Zone process. This is selected as the preferred process because of its ability to produce effluent that will satisfy the anticipated new nitrogen limits, general ease of operation, and its lower capital and life cycle costs over other alternatives considered. Additionally it is well suited for conversion of a lagoon system and has been successfully implemented in other locations where an existing lagoon system was required to meet low effluent nitrogen limits. Finally, the alternating zones configuration is also readily converted to other process configurations described above, including the Low DO (Simultaneous Nit/Denit), Step Feed, On/Off Aeration, and even the MLE process. Given the ability of the alternating zones to achieve treatment objectives, cost advantages, and applicability for conversion of the existing lagoon systems, this is the preferred alternative identified in this feasibility report.

This Feasibility Report is to be critically reviewed by both the City and District and revised as determined appropriate. Then, as appropriate, the City and District should consider implementing the preferred alternative and submitting this Feasibility Study to the LRWQCB for their review along with conceptual agreements between the City and District and preliminary funding options.

Should the City and District desire to implement Joint Treatment the next steps will generally be:

1. Preliminary agreements between the City and District
2. Preliminary funding alternatives
3. Preliminary regulatory approval
4. 30% design
5. Value engineering
6. CEQA and NEPA (if applicable) compliance
7. Final design
8. Final agreements between the City and District
9. Final regulatory approval
10. Final funding
11. Bidding
12. Contract award
13. Construction
14. Startup

11 Works Cited

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12 Appendices

Appendix 1: Monitoring Well Information