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December 19, 2025

Mr. Adam Teitzman, Clerk
Florida Public Service Commission
2540 Shumard Oak Boulevard
Tallahassee, FL 32399-0850

Re: NC Real Estate Projects, LLC dba Grenelefe Utility
Request for Staff Assisted Rate Increase – Staff's Seventh Data Request
Docket No. 20250023-WS

Dear Mr. Teitzman,

Attached is the Preliminary Design Engineering Report as part of the response to Staff Data Request #7.

Should you or any members of the Commission staff have any questions in this regard, please let us know.

Sincerely,

SUNDSTROM & MINDLIN, LLP

/s/ F. Marshall Deterding

F. Marshall Deterding
Of Counsel

FMD/brf

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PRELIMINARY DESIGN ENGINEERING

REPORT

FOR

Grenelefe Water Utilities

Wastewater Treatment Plant

Polk County, Florida

ID: FLA013016
Permit No.: FLA013016 Expires: November 15, 2027

Prepared For:
NC Real Estate Projects LLC
3425 Turnberry Dr
Lakeland, Florida,33803

October 24, 2024

Prepared By:

McDONALD GROUP INTERNATIONAL, INC.
9030 S. BRITTANY PATH
INVERNESS, FLORIDA 34452

PRELIMINARY DESIGN ENGINEERING

REPORT

FOR

Grenelefe Water Utilities

Wastewater Treatment Plant

Polk County, Florida

The information contained in this report was prepared in accordance with sound engineering principals, and the recommendations contained within have been discussed with the permittee

Digitally signed by George J McDonald
Date: 2024.10.28 13:23:12 -04'00'

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No. 44740
★
STATE OF
FLORIDA
PROFESSIONAL ENGINEER

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This item has been digitally signed and sealed by George J McDonald PE on the date adjacent to the seal. Printed copies of this document are not considered signed and sealed and the signature must be verified on any electronic copies

PRELIMINARY ENGINEERING
Report
FOR
Grenelefe Water Utilities
WASTEWATER TREATMENT PLANT

Section

Appendix

SBR design Calculations
Sidestream Unit Process Design Calculations
Denitrificaton Filter Design Calculations
Parkson Corporation Design Calculations/Modelling

Exhibits

Hydrogeology & Geotechnical Report (Andreyev Engineering)
Permit Drawings

PRELIMINARY ENGINEERING REPORT

1.0 INTRODUCTION

The Florida Department of Environmental Protection(FDEP) requires that a preliminary engineering report be submitted to the Department with a permit application to construct a new or substantially modify a wastewater treatment plant. It should be prepared substantially to conform with the submittal requirements of the guideline document published as a companion to rule 62-620.

This preliminary engineering report is submitted to the FDEP by McDonald Group International, Inc., George J. McDonald, P.E., consultant engineer for NC Real Estate Projects LLC, the owner and operator of the Grenelefe Water Utilities Wastewater Treatment Plant located in Polk County, Florida in order to comply with these requirements.

The facility is located 4501 Abbey Ct. AUSGS quad map are provided in Figures 1.1.

Grenelefe Water Utilities is required by changes in State regulation to increase the level of treatment provided by its wastewater treatment plant located in Polk County Florida. In addition, additional treatment capacity is needed for proposed new development. These upgrades generally concern improvements to meet advanced nitrogen and phosphorus removal.

The regulatory factor driving the needs for these is compliance with the Florida Department of Environmental Protection's Lake Okeechobee Basin Management Action Plan (BMAP). Secondary treated effluent is presently disposed of at the existing rapid rate land application system (infiltration basins). Advanced nitrogen removal is required by the BMAP for all methods of effluent reuse or disposal.

As further described in this report the owner and developer of the Grenelefe Resort has forecasted a short term need of a permitted capacity for treatment and effluent reuse of 0.495 MGD. Longer term, the owner forecasts the need for eventual capacity of 1 MGD.

The existing treatment plant is a 0.680 MGD extended aeration treatment plant with 0.340 MGD in permitted effluent disposal capacity (land application reuse) in four rapid infiltration basins. The existing treatment plant was constructed in three phases beginning in the 1970s. The first two phases are no longer in operation. Only the third phase, constructed in the 1980s is in operation, and is a complete flow train of 0.340 MGD capacity.

This report will document a proposed expansion plan:

- Construction of a Sequencing Batch Reactor (SBR) system incorporating Biological Nitrogen Removal (BNR) along side the existing plant; re-purposing of various existing tanks both in an out of service, to essential side stream unit processes, in phases.

- Expansion and consolidation of the existing four rapid infiltration basins into a system of two enlarged rapid infiltration basins with a combined effluent reuse or disposal capacity of 0.5 MGD.

- The construction of the SBR and expansion/consolidation of the rapid infiltration system are intended to allow a complete permitted treatment and reuse system of 0.495 MGD.

- The report and accompanying exhibits will document how the plant can be expanded to 1 MGD capacity and will request permit approval for future construction. Future expansion of the reuse system to 1 MGD will be a future permit application.

1.1 Authorization

NC Real Estate Projects LLC has retained George J. McDonald, P.E., of McDonald Group International, to study the existing conditions at the Grenelefe Water Utilities Wastewater Treatment Plant in order to prepare the documentation which supports this application.

1.2 Related Reports and Documents

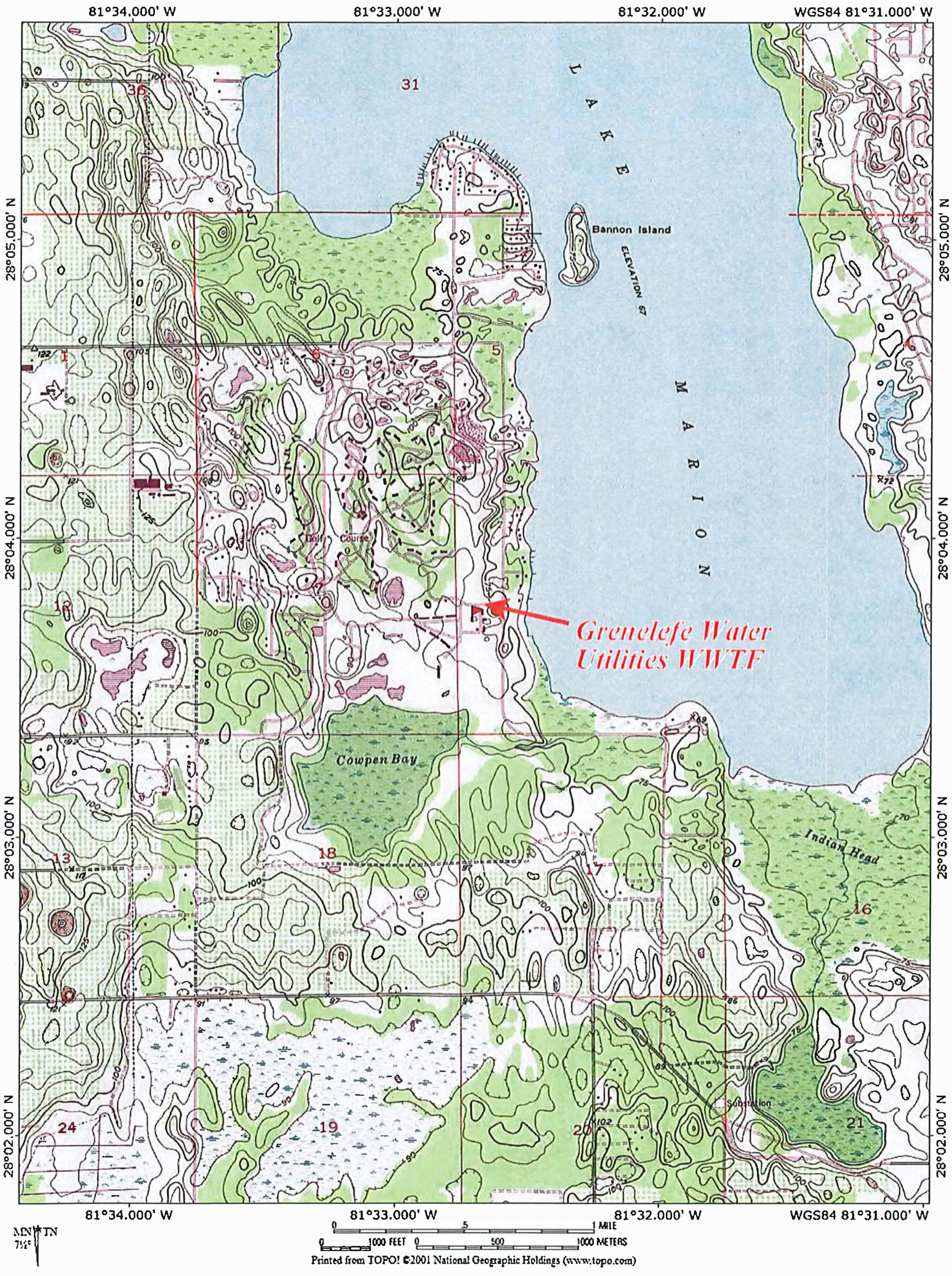
Accompanying this report are :

- FDEP Forms 1 and 2A for a domestic wastewater treatment plant.

- Permit drawings of the proposed facility; the drawings themselves include a hydraulic profile, process diagram, as well as tankage layout, and other illustrative details. Additional information is thus contained in the accompanying plans and documents.

- Accompanying this report also is a geotechnical report on the hydrogeology of the proposed rapid infiltration system modifications and expansion by Andreyev Engineering.

Figure 1.1 Location / USGS Map



1.3 Basin Management Action Plan Requirements

The requirements of the Basin Management Action Plan are implemented through an attachment to the permit the State Department of Environmental Protection issues to the plant owner for operation called an Administrative Order. The order requires the permittee to comply with the new discharge limits and to carry out certain activities per a schedule that is made a part of the facility permit.

The Administrative Order (AO)requires the facility, within a set period of time to comply with the requirements of the Lake Okeechobee BMAP for TN and TP reduction. The specific limits in the AO are:

Total Nitrogen:	Max 10 Annual Average Lake Okeechobee Basin Management Action Plan June 2018
Phosphorus, Total:	Max Report Single Sample mg/L

The required schedule is as follows:

Action Item	Due Date
1) Collect monthly effluent samples and analyze for TN and TP and report as required by this permit and Discharge Monitoring Report.	First day of the second month following the permit issuance until September 31, 2025
2) Submit a proposal with the most feasible option to bring the TN and TP into compliance with the final limits being 10.0 mg/L and of 6.0 mg/L, respectively. If necessary, schedule a meeting with DEP SWD office to discuss the proposal.	Prior to September 31, 2025
3) Submit a proposal with the necessary modifications to the facility required to meet the treatment and disinfection requirements of 62-610.460, F.A.C., giving the facility the option to dispose of the effluent via a Part III Slow-Rate public access reuse system (Irrigation). If necessary, schedule a meeting with DEP SWD office to discuss the proposal.	Prior to September 31, 2025
4) Obtain the Department's approval for the proposal.	Prior to September 31, 2025
5) Implement the proposal.	Within twelve months of DEP approval and after obtaining a permit modification, if required.
6) Comply with the final limit for TN and TP or obtain Department approved regulatory relief	Within three months of completion of any modification if required.
7) Meet the facility classification and operator staffing requirement in accordance to Rule 62-699.310 (2) (a)1., F.A.C as a Category I, Type III, Class C facility.	Upon the date of completion for item 6.

It should be noted there are some differences in the text of the AO and the text of the BMAP with respect to Nitrogen and Phosphorus required reduction. The following table is from the June of 2020 Lake Okeechobee BMAP:

Table 19. TP effluent limits

mgd = Million gallons per day

Permitted Average Daily Flow (mgd)	TP Concentration Limits for Direct Surface Discharge (mg/L)	TP Concentration Limits for RRLA Effluent Disposal System (mg/L)	TP Concentration Limits for All Other Disposal Methods, Including Reuse (mg/L)
Greater than or equal to 0.5	1	1	6
Less than 0.5 and greater than or equal to 0.1	1	3	6
Less than 0.1	6	6	6

Table 20. TN effluent limits

mgd = Million gallons per day

Permitted Average Daily Flow (mgd)	TN Concentration Limits for Direct Surface Discharge (mg/L)	TN Concentration Limits for RRLA Effluent Disposal System (mg/L)	TN Concentration Limits for All Other Disposal Methods, Including Reuse (mg/L)
Greater than or equal to 0.5	3	3	10
Less than 0.5 and greater than or equal to 0.1	3	6	10
Less than 0.1	10	10	10

The facility is currently permitted for a capacity of 0.340 MGD; according to the BMAP the standard is 6 mg/L TN and 3 mg/L TP for a faciity of this size using rapid rate land application, whereas the permit and AO is for 10 mg/L TN and "report" for TP.

From communication with FDEP at the Southwest District in Tampa, it appears the Administrative Order is in error; FDEP is likely to make a Department initiated revision.

The permit or AO itself does not provide guidance as to what the standards would be if the facility was expanded to over 0.5 MGD capacity, but the BMAP indicates it would be 3 mg/L TN and 1 mg/L TP with effluent discharged to rapid rate systems as opposed to reuse systems.

The current treatment plant has been permitted to only a meet a 12 mg/L Nitrate standard, which is but one form of nitrogen of several that can be present in the plant effluent. The current treatment plant was not designed to reduce phosphorus.

1.4 Treatment Plant Historical Background

The treatment plant was constructed through three phases. The first, Phase 1, was constructed around or after 1976 and appears to have had a capacity of 0.170 MGD. A few years Phase 2 was constructed, in which the structure was "mirrored" with the same unit processes and volumes: both parallel plants flow trains would have had a capacity of 0.340 MGD. Around 1986 a Phase 3 flow train similar in process and operation was built next to the first two phases.

Technically the Grenelefe treatment plant consists of three plants, two of 0.170 MGD capacity and one of 0.340 MGD treatment capacity. Each flow train consists of aeration - which was delivered by both mechanical and diffused aeration processes ; settling of process sludge occurs in rectangular settling tanks with waste sludge digesters. Effluent from each flow train is combined in a common sand filter system, and then disinfected in a single chlorine contact tank

Historically the treatment system was permitted for a capacity of 0.680 MGD. In the 1990s effluent was pumped to a golf course pond from which water was withdrawn to irrigate the resort's South golf course.

On September 12, 2000, the reuse of reclaimed water on the golf course was halted by the Florida Department of Environmental Protection owing to the facility lacking a number of the features required of treatment plants that provide reclaimed water for reuse.

Shutting down the reuse system meant all the effluent water had to be directed to existing unlined water storage ponds. Up until 2000 these appear to have been considered holding ponds and did not have a capacity assigned to them. However, once they were placed into use as infiltration basins and appeared to work successfully, the 4 ponds or infiltraton basins that make up that system were given a nominal capacity of 0.340 MGD/

The mechanical equipment in the original 1970s era Phase 1 and Phase 2 plant flow trains deteriorated and both Phase 1 and 2 flow trains were placed out of service by the early 2000s rather than repaired and maintained. Only the phase 3 flow train is presently in operation.

Owing to limitations in the effluent disposal system and with 50% of the plant's flow trains being out of service, capacity is limited to 0.340 MGD. The current permit does recognize that the concrete tankage in place could yield a treatment capacity of 0.680 MGD if it was all mechanically restored. The permit does not recognize any historic reclaimed water reuse capacity.

2.0 DESIGN WASTEWATER FLOW, PHYSICAL AND CHEMICAL PROPERTIES

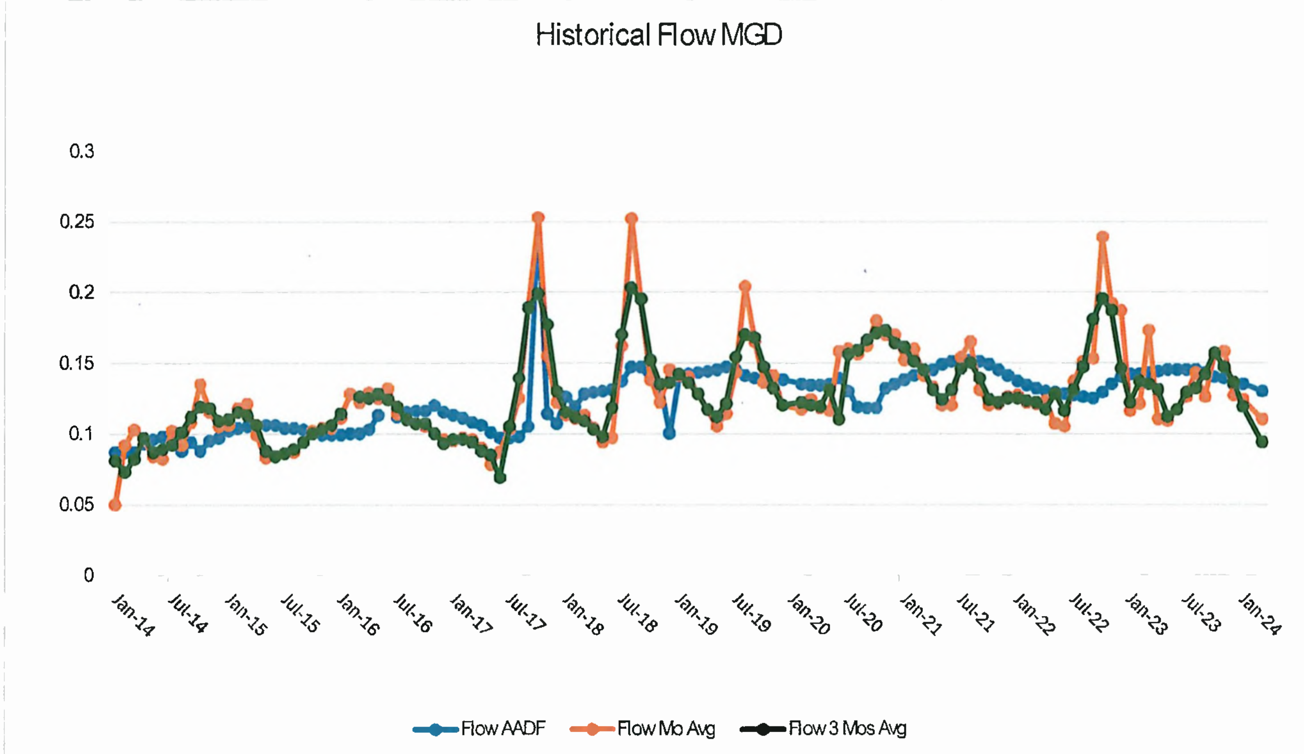
Wastewater historic, present and projected flow, annual patterns, and projected influent loading, is discussed in this section.

2.1 Plant Flow Characteristics

Ten years of flow data from Discharge Monitoring Reports (DMRs) were reviewed to assess the present and historical plant flow characteristics.

Figure 2.2.1 graphically illustrates the month average, rolling three month and annual average flow for the past ten years:

Figure 2.2.1 Wastewater Flow Chart



The flow pattern exhibits typical winter season increases in flow which subside later in the year.

Typically in assessing present used treatment plant capacity, a three year look back is made to assess the flow against permit statistical metrics:

March 2021 to March 2024

Parameter	Result	Unit	Permit Limit
Max Flow AADF	0.153	MGD	0.34
Max Mo Flow	0.239	MGD	report
Max 3 Mos Flow	0.195	MGD	0.34

The plant is permitted on a maximum three month basis for treatment and on an annual average basis for effluent disposal. For treatment, the plant is operating at 57% of capacity based on the highest three month average flow in the past three years. For disposal of effluent, it is operating at 45% of permitted capacity.

2.2 Unit Flow Rates

The service area comprises a mix of family homes, townhouses, and condominiums.

There are a number of commercial accounts, which are associated with the resort conference center and golf course. The resort conference center and golf course are closed. There are no industrial wastewater contributors.

There is presently a total of about 1400 served units. Based on the annual average flow, the flow per unit is about 109 gpd each or 139 gpd on a three month basis.

3.0 FUTURE CONDITIONS - WASTEWATER FLOW PROJECTION

3.1 Smokey Groves

The short term flow projection is based on the proposed development called Smokey Groves. This is a single family home addition of approximately 426 units.

The projected flow from this can be based on 1) for a high estimate, the level of service described by the County for new development, at 260 gpd per unit or 2) for a low end estimate, based on the assumption that population, occupancy and usage patterns will match the existing service area.

In the former case, the expected flow is 110,760 gpd (426 x 260), which added to the current 0.195 MGD would yield 0.306 MGD in flow, or bring the existing plant to 90% of permitted treatment capacity.

In the latter case, with 1400 presently served units, the flow per unit is about 139 gpd each; 426 more would be another 0.059 MGD, for a total flow to the plant of 0.254 MGD, and would places the existing plant at 75% of treatment capacity.

3.2 Long Term Flow Projection

At this writing, plans for redevelopment of Grenelefe and the addition of other properties in the area is at a conceptual development stage. Detailed projected unit counts and a reasonable timeline for their progressive addition remains under development by others. In general it is expected that the short term capacity of 0.495 MGD should be sufficient for 5 to 10 years, after which wastewater flow may increase to 1 MGD.

3.3 Owner Specified Design Capacity

The owner has directed that the plant should be modified to meet BMAP and reuse treatment level requirements and be expanded to 0.0.495MGD, with planning and permit level design for expansion of the treatment plant to 1 MGD. .

The selected capacity provides ample additional capacity over what is necessary to serve Smokey Groves. Depending on actual flow that results from that development, the 0.495 MGD plant provides 0.189 to 0.240 MGD available capacity for additional development.

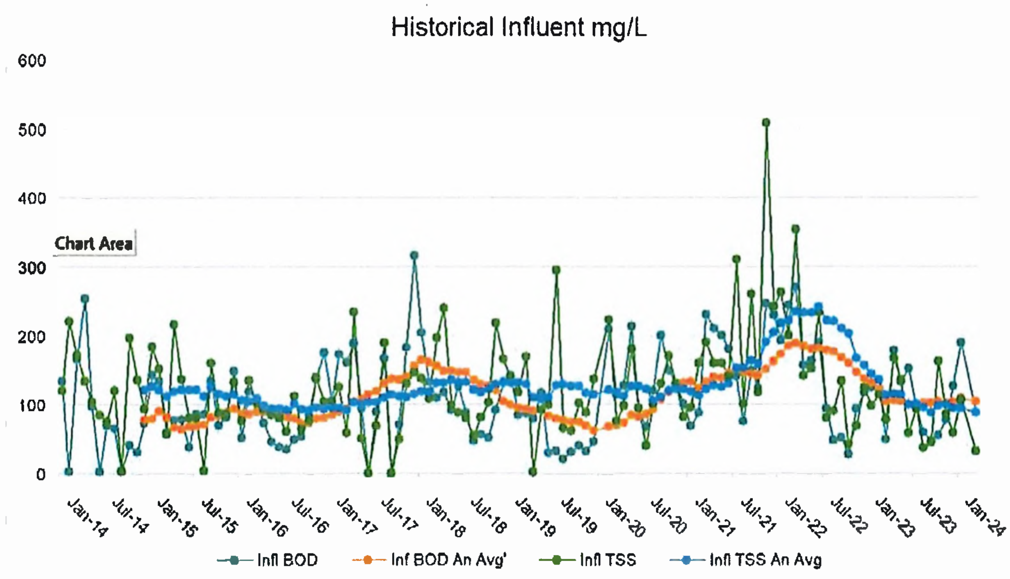
3.4 Peak Hour Flow

Peak hour flows were estimated by consideration of the characteristics of the service area and plant performance. Based on this, the peak hour factor is estimated to be more or less 3 times the average daily flow.

3.5 Physical and Chemical Characteristics

The major parameters used to evaluate influent strength are influent BOD, TSS, TKN. Other significant parameters assessed include COD, pH, and alkalinity.

The chart below indicates graphically the historical, available influent test data.



Based on available test and variable test data, the influent strength for BOD and TSS is considered to be as follows:

Table 2.3
Influent Strength

	BOD	TSS
3 Year Avg	137	148
Std Deviation	66	99
Avg + St Dev	**203**	**248**

(For aeration basin and aeration system sizing, the higher estimate is used. For nitrogen removal analysis, the lower average will be used to assess the need for a carbon supplement).

Other design parameters have been selected as follows:

Infl Soluble BOD	67	mg/L
Infl COD	406	mg/L
Infl Soluble COD	134	mg/L
Total Suspended Solids:	248	mg/L
Total Kjeldahl Nitrogen:	40	mg/L

3.6 Summary, Projected Flow and Organic Loading

The historical annual pattern of flow in terms of the ratios of maximum three month, annual average, and maximum months are expected to continue as the overall quantity of flow increases. BOD, TKN and TSS are expected to be randomly variable on a day to day basis, but to overall maintain the historical averages reviewed.

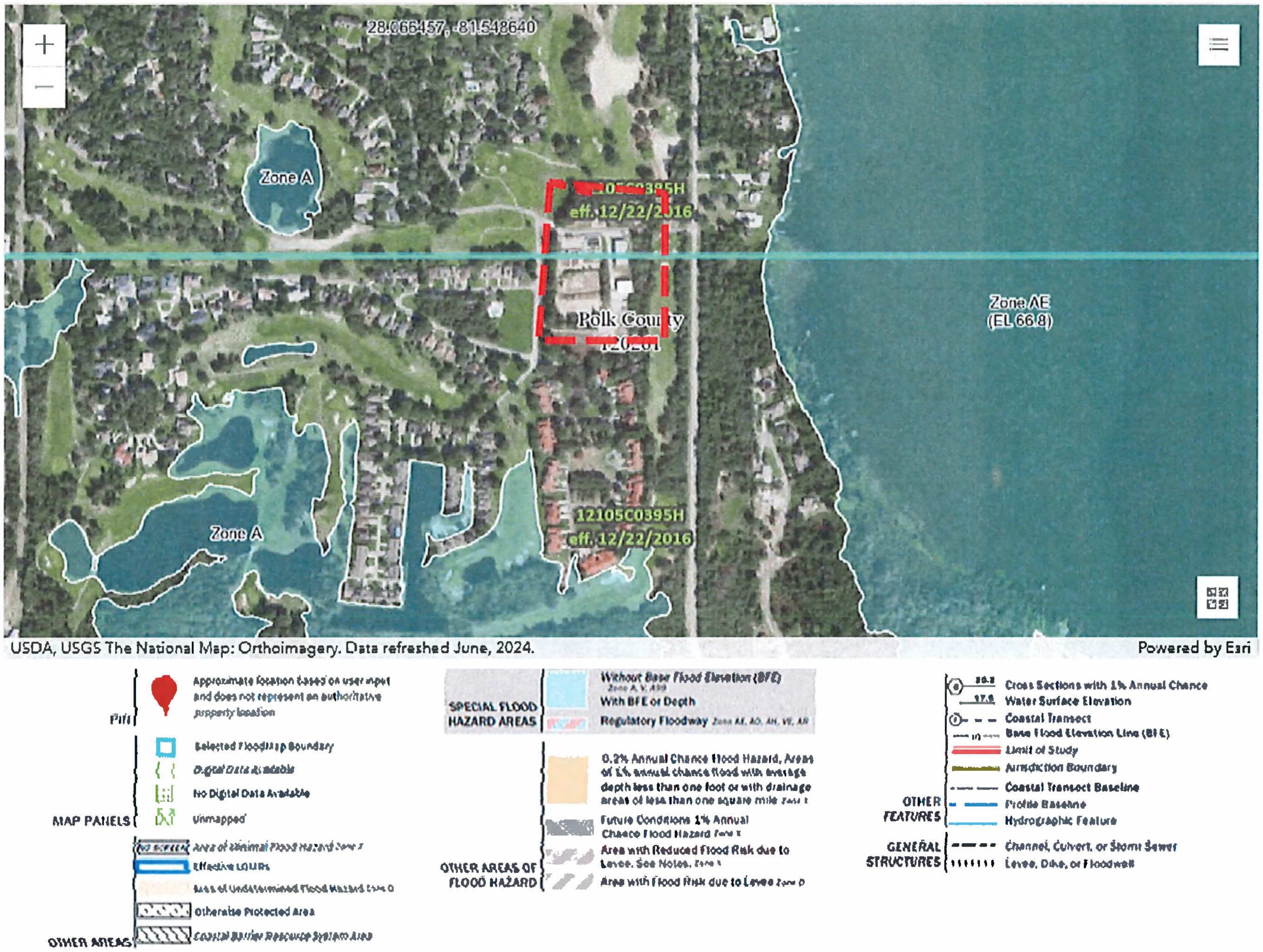
A summary of the projected design flow and loading conditions is as follows:

Organic		
	BOD	203 mg/L
	TKN	40 mg/L
	TSS	248 mg/L
Hydraulic		
	Annual Average	0.495 MGD
	Long Term AADF	1 MGD
	Peak Hour	3 x AADF

3.0 FACILITY ENVIRONMENTAL CONSIDERATIONS

3.1 Stormwater Management and Flood Protection

According to FEMA mapping, the existing treatment plant, proposed plant improvements, and proposed rapid infiltration basin systenm improvements, are outside the limits of a flood zone A or AE. (The northern part of the plant and the north R.I.B are located on community panel 12105C0385H, and the southern part of the plant and southern R.I.N. are found on community panel 12105C039H; excerpt below).



No significant additional runoff is expected to be generated by this facility. Rainfall on the open air process tankage will largely fall into the treatment plant. Only incidental, incremental runoff from the tops of the walls plant structure is created.

Direct rainfall elsewhere on the site falls into the effluent disposal system infiltration basins and percolates below. The infiltration basin berms are designed to prevent the entry of runoff.

3.2 Environmental Effects of Project

3.2.1 Proximity to Residential Areas

This treatment plant is existing, was originally intended to serve the Grenelefe resort community and is located within the community it serves. The proposed improvements take place partly within the existing structure. The structural addition has been located on the side of the plant furthest away from the served community and within Grenelefe's original maintenance office and warehouse area.

3.2.2 Odor

Normally, odor from this project is expected to be minimal. The liquid in the process SBR tankage is aerated/treated, and normally has no objectionable odor associated with it.

The major source of potential odor will be from the flow equalization tankage and the sludge digester(s).

Raw wastewater piping from the pretreatment screening will be designed to minimize direct contact with air entering the tankage

The surge tank is aerated to prevent septic conditions from developing.

Digesters can be sources of odor when the air is left off too long for decanting and upon re aerating releases entrained gas. The digester is intended to be aerated 24/7, with incidental short periods during operator attendance for decanting, which should preclude the likelihood of poor odor. Control of digester odors will be covered in the facility O&M manual.

3.2.3 Noise

The electric driven pumps used at this plant are quiet in operation. The major source of noise will be from the new facility blowers. The existing treatment plant uses a mix of mechanical surface aerators and centrifugal blower unit.

New blowers for the SBR will be located between the existing plant and the new SBR structure on the east side away from the homes in Grenelefe. These blowers will be a special noise reduction model, enclosed in a manufacture supplied weather and sound deadening enclosure.

3.2.4 Public Accessibility

The treatment plant and modified/expand rapid infiltration basin will be fenced : no public access is allowed.

3.2.5 Lighting

Lighting at the treatment plant site will be limited to lighting for service workers and the operator; lighting will be provided at the SBRs and SBR mechanical pumps and blowers with outlets to connect portable, temporary lighting.

3.2.6 Aerosol Drift

In the SBR treatment plant, adequate free board (two feet) is provided to minimize loss of liquid or any aerosols over the side of the plant. Aeration is induced at the bottom of the liquid held, so there is no splashing of liquid at the surface..

The effluent disposal system is a rapid infiltration basin system, not a spray system, and the discharge of aerosols from the disposal is not expected.

4.0 TECHNICAL INFORMATION AND DESIGN CRITERIA

The facility design criteria are based on what effluent discharge standards have to be met. These in turn are based on how the treated effluent is disposed of or reused and also depend on the permitted capacity of the treatment system.

The treatment process is then selected with consideration to the current plant performance, the projected waste strength and flow, and the discharge standards that have to be met.

Information presented in this section discuses the applicable discharge standards, reviews the flow and loading projected, outlines with reference to the process flow diagrams how existing unit processes and tankage will be integrated with new components in order to yield the capacity required, and how this is constructed in phases.

4.1 Effluent Disposal/Reuse Method

The method of effluent reuse will be by disposal to rapid infiltration basins. The geo-hydraulics and ground water monitoring plan are discussed in the accompanying geotechnical report. The physical construction features are discussed in section 6.0 of this report and are shown in the accompanying permit drawings.

4.2 Required Levels of Treatment

As required by this method of effluent disposal or reuse, the wastewater plant will have to achieve the following technology based levels of treatment (TBELs):

Grenelefe Water Utilities
Wastewater Treatment Plant
Permit TBELS

Disposal to rapid rate systems

1. BOD and TSS maximum concentrations -
   20 mg/L annual average
   30 mg/L monthly average
   45 mg/L weekly average
   60 mg/L any one sample
2. pH range - 6.00 to 8.50
3. Fecal Coliform -
   200 #/100 annual average
   800 #/100 maximum
4. Minimum Cl_2 conc. - 0.5 mg/L

Nutrient Reduction Requirements based on Capacity

6. TN for Capacity of 0.495 MGD 6 mg/L annual average
7. TN for Capacity of 1 MGD 3 mg/L annual average
8. TP for Capacity of 0.495 MGD 3 mg/L annual average
9. TP for Capacity of 1 MGD 1 mg/L annual average

4.3 Historical Nutrient Reduction Performance

Consideration is given with respect to how well the existing treatment plant meets the required level of treatment for reduction of nutrients. The existing treatment process is extended aeration.

4.3.1 Reduction of Total Nitrogen

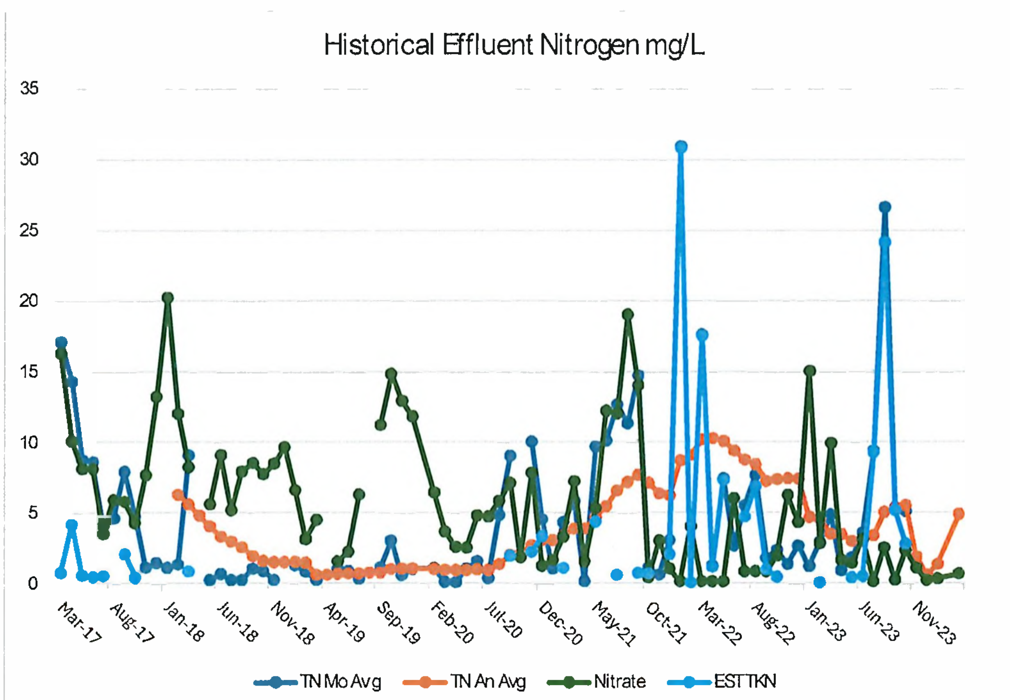
Total Nitrogen has several forms: ammonia, organic nitrogen, nitrate and nitrite. Reduction of nitrogen typically consists of two consecutive processes which address the different forms nitrogen is present in.

Almost all of the incoming raw wastewater to a treatment plant is in the form of ammonia and organic nitrogen. The first process in reducing nitrogen is the conversion of the combined ammonia and organic nitrogen (together called TKN) to nitrate, called nitrification. The second process is the reduction of nitrate (and a very small amount of nitrite) to nitrogen gas, called denitrification.

During normal operation, most wastewater plants may have nitrification and denitrification processes happening as aerators turn on and off. How much nitrogen reduction occurs can vary with the equipment used, volumes available and operating practice.

The Grenlefe treatment plant effluent has been tested for a number of years for total nitrogen and nitrate content;. TKN is not tested, but it is possible to subtract nitrate from Total Nitrogen to get an estimate. With this data, is possible to assess how the existing plant as is performs.

The chart below shows how well the treatment plant as is reduces nitrogen and in what forms remain in the plant effluent:



For successful nitrogen reduction, TKN should be low, generally less than 2 mg/L. In the last few years,TKN will has often been between 7.5 and 15.

In the historical data, where nitrate is very low and TKN is very high, the plant was not oxidizing (nitrifiying) ammonia and organic nitrogen well. This may result from long aerator off times or aerator out of service events.

When successfully reducing Total Nitrogen, Nitrate will be higher than TKN in the effluent and Total Nitrogen will be slightly higher than Nitrate.

With successful nitrogen reduction (denitrification) , the TN will be less than the BMAP requirement.

Looking back at the past three years, results can be summarized as:

Table 4.3
Existing Treatment Plant Performance

Summary March 2021 to March 2024

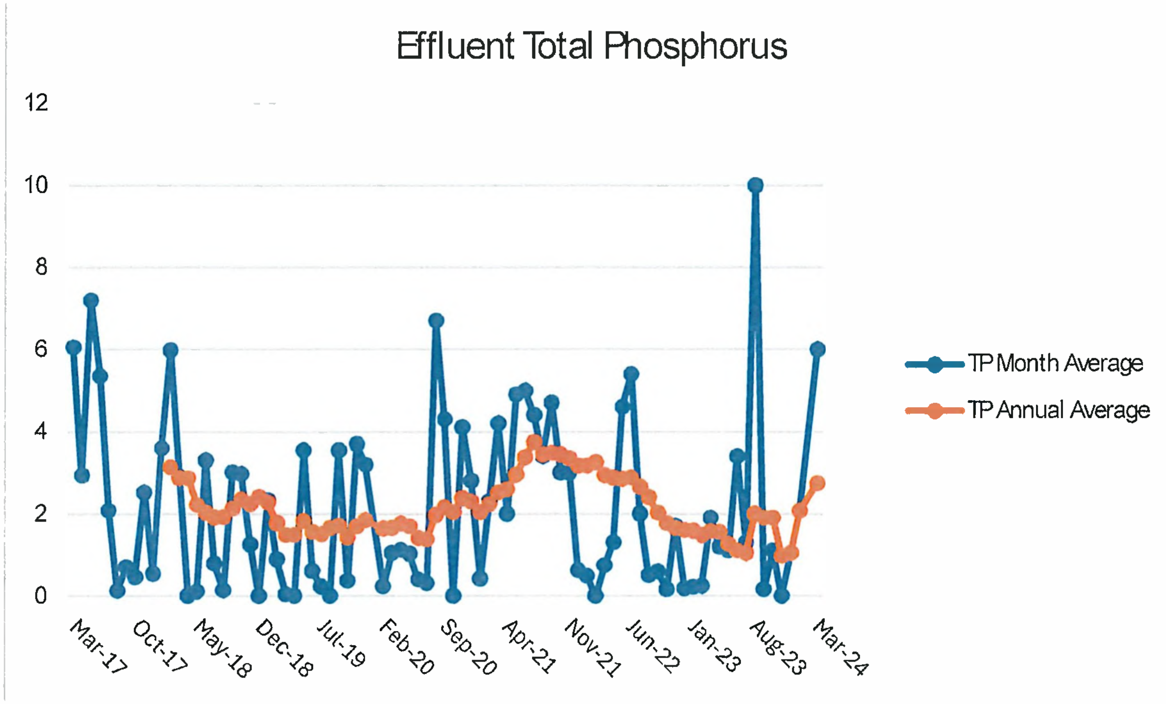
TN Max Month	30.9	mg/L
TN Max An Avg	10.25	mg/L
Max Nitrate Month	15	mg/L
Max Mo TKN	30.1	mg/L
Min Mo TKN	-13.84	mg/L

(The negative TKN resulting from back computation indicates TN may not have been reliably determined in the month tested and is considered anomalous).

In summary the plant at times appears able to get below 10 TN but not consistently; 6 mg/L or lower cannot be achieved with the existing plant as is. Elevated effluent TKN (above 2 mg/L) indicates there are months the facility does not oxidize ammonia and organic nitrogen to the low levels it needs to.

4.3.2 Phosphorus Reduction

The following chart depicts the treatment plants historical peformance with the total phosphorus content of the effluent:



Results can be summarized as follows:

Summary March 2021 to March 2024

TP Max Month	10	mg/L
TP Max An Avg	3.7	mg/L

In many months, the effluent TP annual average is less than 3, but it is not consistent. The treatment plant has no chemical biological process for reducing TP so months where its meeting the 3 mg/L TP standard in the BMAP (for a capacity of 0.495 MGD) likely reflects a low influent TP loading. The treatment plant as is will not meet a 1 mg/L TP limit at 1 MGD.

4.4 Design Capacity and Facility Hydraulic/Organic Loadings

Organic

BOD	203 mg/L
TKN	40 mg/L
TSS	248 mg/L

Hydraulic

Annual Average	0.495 MGD (initial expansion)
Annual Average	1 MGD (final expansion)
Peak Hour	3 x AADF

Organic loadings on the individual unit processes and an evaluation of the overall efficiency of the process in terms of relevant criteria are provided in section 4.8 of this report.

4.4 Process Selection

In developing the proposed SBR treatment process and phasing, several alternatives were evaluated:

- Grenelefe WWTF, Alternate 1, Restore and Modify Existing Plant; This involved restoring the existing phase 1 and phase 2 treatment plants, then combining the phase 1 and 2 treatment plants with the in service phase 3 plant to provide a sequential pre anoxic, aeration, post aeration second anoxic process followed by reaeration. The restorative and modification work was costly, requiring the construction of new clarifiers, and while it could yield a capacity of 0.495 MGD, expansion to 1 MGD was not practical without a building another separate process flow train. Alternative 1 was rejected due to cost and complexity.

- Grenelefe WWTF Alternate 2, Construct New Flow Train; this alternative was to construct a new conventional BNR treatment plant with anoxic tankage, new settling tanks with some reuse of the existing treatment plant tankage for sidestream processes. Alternative was rejected because of the cost of constructing the new tankage, clarifiers and all the mixing, recirculation, and recycle pumping systems required.

- Grenelefe WWTF, Alternate 3, Convert to Sequencing Batch Reactor Process. Use of Sequencing Batch Reactor technology eliminates equipment required in the other alternatives such as new clarifiers, return sludge and recirculation pumps. The batch reactor process uses system controls to induce anoxic times necessary for denitrification to occur within the same tankage used for aeration; it eliminates the need to construct separate dedicated anoxic denitrification tankage. Components of the existing plant could be retained with some modification for unit processes such as flow equalization, sludge digestion and disinfection. Overall the SBR process presented as the simplest and least cost approach. In addition, in Florida there are now a significant number of SBR plants in operation operating BNR modes; this is established developed technology.

Alternative 3 was selected for development.

4.7 Proposed Facility Modifications and Process Flow Diagrams

This section describes the specific modifications proposed and references the process flow diagrams provided in the accompanying permit drawings.

4.7.1 Treatment Facility Phases and Unit Process Capacities

The following describes the existing and proposed facility phases and existing or proposed unit process capacities:

Treatment plant phasing notes:

1&2. **Phase 1 and 2**, original treatment plant constructed in the 1970s. Not in service 2024.

Phase 1 components consist of (2) 52050 gallon mechanically aerated aeration compartments, and a 45970 gallon diffused aeration compartment. Settling was in a 25,100 gallon final settling tank (remains existing, not in service). There is a 12,980 gallon sludge digester compartment and a 5800 gallon chlorine contact tank

Phase 2 is a mirror image of phase 1: components consist of (2) 52050 gallon mechanically aerated aeration compartments, and a 45970 gallon diffused aeration compartment. Settling was in a 25,100 gallon final settling tank (remains existing, not in service). There is a 12,980 gallon sludge digester compartment and a 5800 gallon chlorine contact tank

3. **Phase 3**: existing treatment plant in service 2024

Phase 3 consists of (3) 123,032 gallon mechanically mixed aeration tanks, (1) 12,591 gallon diffused aeration compartment, (2) 31,556 gallon final settling tanks, a 28,842 gallon sludge digester, a 10,054 gallon chlorine contact tank. Effluent is filtered by (7) 64 SF downflow sand filters, which drain into a 25,000 gallon clearwell. The clearwell drains to a 19,947 gallon post filter chlorine contact tank, then to a pump tank which sends the water to the(4) existing rapid infiltration basins. Backwash water flows to a 28,778 gallon mudwell and then is pumped back into the phase 3 diffused aeration compartments

The process flow diagram for phases 1,2 and 3 are shown on sheet C2 of the accompanying permit drawings.

The combined capacity of phase 1, 2 and 3 is 0.680 MGD for treatment, but, since phases 1 and 2 are not in service and since the effluent reuse/disposal capacity is limited to 0.340 MGD, usable plant capacity is 0.340 MGD.

The following describes proposed improvements

4. **Phase 4** Improvements include upgrades the plant headworks, conversion of existing tankage for influent flow equalization and SBR decant effluent flow equalization. Construction of a three basin Sequencing Batch Reactor treatment system, and conversion of select phase 3 aeration tanks to sludge digestion and reconstruction of the rapid infiltration basin system,

Phase 4 is broken down into two phases:

Phase 4A: proposed replacement of the headworks screen, and conversion of the phase 2 diffused aeration tank to 45970 gallons in flow equalization. All phase 3 components remain in use as currently configured. Plant capacity remains 0.340 MGD.

The process flow diagram for phase 4A is shown on sheet C3 of the accompanying permit drawings.

Phase 4B has extensive modifications as follows:

- Conversion of the phase 2 digester tank into a grit removal compartment of 9248 gallons capacity;
- Conversion of the phase 1 diffused aeration chamber into a raw wastewater flow equalization chamber of 45970 gallons;
- Conversion of one of the phase 1 mechanical aeration tanks, the phase 1 digester, and the phase 1 settling tank, and the phase 1 CCC into a decant flow equalization unit process of 95,930 gallons total volume.
- Conversion of one phase 3 mechanical aeration zone, one diffused aeration zone, and consolidation with the existing sludge digester to provide 164,465 gallons in sludge digestion volume.
- Construction of a (3) Sequencing Batch Reactors, each with a volume of 250,000 gallons
- The existing Phase 3 (7) 64 SF each filters, 25,000 gallon clearwell, 19,947 gallon CCC and 28,778 gallon mudwell, remain in service.
- Expansion, modification and consolidation the (4) existing rapid infiltration basins into two basins, designated the North and South rapid infiltration basin.

Plant treatment and reuse disposal capacity will be 0.495 MGD. Sheet C4 in the accompanying drawings depicts the process flow diagram for this phase.

Note: the improvements proposed for phase 4A and the grit chamber conversion of phase 4B are presently being separately permitted under a minor modification for this facility in order to expedite this work getting into construction.

5. **Phase** 5: Increases the treatment capacity to 1 MGD. Unit Process additions and modifications include the following:

- Construction of additional (3) 250,000 gallon each SBR chamber (Total of 6).
- Conversion of (1) Phase 1 aeration chamber to decant equalization, providing a total decant equalization available volume of 147,980 gallons
- Conversion of (2) Phase 2 aeration tanks to raw wastewater flow equalization use, provide a total FEQ unit process capacity of 196,040 gallons
- Conversion of (2) phase 3 aeration chambers to sludge digestion, providing a total
- sludge digestion capacity of 410,529 gallons.
- Demolition of the phase 3 sand filters and mudwell
- Conversion of the existing phase 3 25,000 gallon filter clearwell into a chlorine contact tank
- Construction of (3) denitrification filters of 150 SF each.

Sheet C5 of the accompanying permit drawings depicts the process flow of the phase 5 treatment plant.

4.8 The SBR Process

A Sequencing Batch Reactor (SBR) treatment system is activated sludge system like a conventional continuous flow treatment plant. In both systems, raw wastewater is aerated to induce aerobic conditions, nitrifying influent ammonia and organic nitrogen into nitrate, and grows a biomass which consumes BOD, and which is then separated from the treated water through settling. In both systems, it is possible to induce anoxic conditions for nitrate reduction. The difference though is flow through a conventional treatment plant is continuous, passing through all the anoxic and aeration basins such a plant may have 24 hours a day, flowing continuously into a settling tank where the biomass separates from the treated wastewater, with settled water flowing continuously out of the settling tank.

An SBR plant will receive a fixed volume of wastewater first in one tank, which it will then process as a batch volume through aeration and anoxic steps in the same tank, building up a biomass. Following processing steps, the biomass settles in the same tank, leaving treated effluent above the settled biomass. The effluent is removed by a decant device which draws off the treated water. Meanwhile, wastewater is still entering the plant, but, the wastewater enters the next SBR basin in sequence, while processing the batch(es) of wastewater that arrived earlier in the other basins. As each batch completes processing, raw wastewater will flow again to the basin that has completed its batch. Each basin will have several such batch fill and treat cycles during the day.

For phase 4B, The plant will include three 250,000 gallon independent SBR basins capable of processing 0.495 MGD. For phase 5, the plant have six 250,000 gallon basins, all capable of independent operation and operating in parallel.

The equipment used in each SBR is relatively basic. Influent is admitted to each basin via an automated control valve which directs the liquid to disperse through the basin via a submerged manifold. Air supply to each basin comes from a dedicated blower, which sends air to a submerged jet aeration diffusion manifold. Mixing of the liquid in the basin is supplied by a dedicated recirculation or motive pump which recirculates biomass out from and back into the tank through a mixing manifold. Effluent is removed by a floating decant device. Waste biomass is removed by a valved drain pipe, sending waste biomass to the plant sludge digester.

4.8.1 SBR Treatment Steps

The SBR batch process in each basin typically consists of the following steps:

1) FILL - In the FILL mode, a motor actuated valve on the SBR fill line opens, and screened and degritted wastewater flows from surge pumps pulling raw wastewater from the flow equalization tankage, sending it into the SBR basin. The plant process control system is set up to insure that at least one SBR is in the FILL mode at all times.
The FILL mode can be further broken down into two sub steps:

1a) The ANOXIC FILL step: During ANOXIC FILL, raw wastewater enters the SBR basin and makes contact with the MLSS in the basin. The anoxic fill step acts to initiate the reduction of the nitrate component of total nitrogen present.

1b) The AERATED FILL step : during the AERATED FILL mode, screened sewage continues to enter the SBR while the SBR recirculation or "motive" pump is activated, and air is supplied to the SBR by a blower. Both the mixed liquor from the recirculation pump and the air from the blower are forced into the SBR through nozzle jet aerators. The jet aerators discharge the mixed liquor/air mixture into the basin as a high energy plume which results in an efficient oxygen transfer rate mixing within the SBR The aeration reduces both BOD and oxidizes (nitrifies) influent ammonia and organic nitrogen.

2) REACT - The REACT step is similar to AERATED FILL except that the SBR has completed filling and screened sewage flow has been directed to the next basin in sequence. The REACT step itself has two sub steps:

2a) ANOXIC REACT: during the prior aerated step, incoming ammonia and organic nitrogen are nitrified. The Anoxic React step is an additional nitrate reduction step in which anoxic conditions in the basin are induced. During this step, the supply of air is off but the recirculation or motive pump system is in operation to keep the MLSS mixed.

2b) AERATED REACT : Following anoxic react,, the air supply to the basin is turned back on. The remaining BOD within the basin is metabolized and overall settleability of the MLSS in the plant is improved. It is during REACT that the SBR functions similar to a typical conventional aeration basin.

3) SETTLE - During SETTLE, the recirculation or motive pump and the basin air supply is deactivated. The biomass is permitted to separate from the treated water and settle to the bottom of the basin. The SBR basin functions as a clarifier with a zero flowrate entering or exiting the basin during this step.

4) DECANT - Once the biomass has settled sufficiently, a motor actuated valve in the SBR effluent piping opens, allowing the effluent to flow out of the SBR. The effluent enters the effluent piping through a decanting mechanism which is suspended within the clear fluid in the basin.

5) IDLE - During the IDLE step, excess biomass (sludge) is wasted from the SBRs while waiting to begin the treatment cycle again at the FILL mode. It is anticipated that the duration of the IDLE periods may vary depending on the peak flows into the facility, the raw wastewater strength, and the mass of biomass grown.

4.8.2 Nitrogen Removal

The following discussion provides an overview of the design methodology for reduction of total nitrogen.

Nitrogen removal consists of two processes, the first is the conversion of ammonia and organic nitrogen (together called TKN) to nitrate, and then the reduction of nitrate to nitrogen gas. The first is called nitrification, the second is called denitrification.

Nitrogen control in extended aeration processes can be obtained in accordance with recommendations contained in the Water Pollution Control Federation Manuals of Practice on Nutrient Control (MOP FD-7), Wastewater Treatment Plant Design (MOP 8), as well as the USEPA manual, Nitrogen Control.

Nitrification

In order to denitrify biologically it necessary to first nitrify. Nitrification design is based around determining a process solids retention time that is long enough to ensure complete nitrification and to ensure that adequate oxygen is supplied. In Florida average temps and considering a normal strength wastewater, generally 6-7 days is needed; an appropriate safety factor is applied to this.

Since the biological process design (see design table in the appendix) has 1) a longer SRT then the nitrification design, even with a safety factor of two, and 2) the capacity of the air supply system has been sized to supply sufficient oxygen for oxidizing TKN, complete nitrification would be expected with the tank sizes proposed.

Denitrification

Denitrification occurs in an environment where there is little or no dissolved oxygen present but nitrate is available, and is called an anoxic condition. Biological denitrification is accomplished by creating anoxic zones in the nitrified mixed liquor. In the absence of free dissolved oxygen, the biota of the mixed liquor will turn to the molecular oxygen contained in nitrate. During anoxic conditions, micro organisms in the plant biomass will turn to the oxygen that is chemically bound to nitrate. As they consume the oxygen in nitrate, nitrogen gas is released and bubbles away.

In SBRs, anoxic conditions are induced, by cycling the air supply on and off long enough for anoxic conditions to develop. While the air is off, mixing is maintained by the recirculation or motive pumps for each basin.

In this case, the SBR batch cycle has been developed in a roughly analogous manner as to how a conventional treatment plant, using a Bardenpho like process, would operate: first, with a predenitrificaton anoxic zone at the head of the process (corresponding to the Anoxic Fill step of the SBR); and second, with a post aeration anoxic zone downstream after aeration (corresponding to the Anoxic React step of the SBR).

This operating scenario is expected to reliably reduce the nitrate produced by the complete nitrification of incoming TKN to less than 6 mg/L TN for Phase 4B with its capacity of 0.495 MGD. As further discussed in the next section, the SBR vendor's operating scenario estimates the effluent total nitrogen can be reduced to 3 mg/L. Its an ambitious target for this technology, and owner and operator will have ample opportunity to try to operate the SBR to achieve 3 mg/L during Phase 4B; In the event the result cannot be consistently be achieved, the planned phase 5 will include a polishing set of three denitrification fitlers (discussed further in this report) to assure meeting the 3 mg/L TN standard required of the Phase 5 1 MGD plant.

Modeling and Calculations

Two calculations sets or models were used to validate the SBR's expected performance.

Reference is made to the appendix of this report which provides a spreadsheet style table of design calculations used to size the SBR, determine the lengths of processing time for each Step in a batch, and select operating parameters to meet TN reduction requirements. These calculations are developed by the Engineer of Record (EOR).

The SBR equipment and process design is proprietary; this permit application is based on using the EcoCycle™ SBR system by the Parkson Corporation. In addition to the spreadsheet calculations, the selected vendor of the SBR equipment has provided modelling calculations to likewise provide recommended operating parameters for the duration of each Step in a batch.

To meet a 6 mg/L TN standard, the EOR calculations provide for the following operating characteristics of the steps in the SBR cycle:

Flow	**MGD**	**0.5**	**1**
Influent Flow per hr	CFH	2785	5570
Influent Flow Per Fill Time/cycle	CF	8356	16711
Fill Events per Day, all basins		8	8
Cycles per Day per basin		2.67	1.33
Duration of Each Cycle	hours	9	18
Fill time	hours	3	3
React	hours/cycle	4.41	13.41
Settling Time	hours/cycle	0.75	0.75
Decant Time	hours/cycle	0.75	0.75
Idle/SludgeWaste	hours/cycle	0.09	0.09
Percent time of Fill time Aerated		20%	20%
Percent of Fill Time Mixed		80%	80%
Aeration Time in Fill	hours	0.6	0.6
Anoxic Time In Fill	hours	2.4	2.4
Percent time of React time Aerated		60%	60%
Percent of React Time Mixed		40%	40%
Aerated Time In React	Hours	2.65	8.05
Anoxic Time In React Mixed	Hours	1.76	5.36
Total Time Aerated/cycle	hours	3.25	8.65
Total Mxed Time /Cycle	hours	4.16	7.76

The EOR calculations provide for 3 hours of fill time to each basin, 2.67 batch cycles per basin, with 4 hours of anoxic time per batch per basin, and 9 hours total cycle time, to reduce TN in the effluent to less than 6 mg/L.

The Parkson Corporation develops the operating cycle a little differently to meet a 3 mg/L standard:

CYCLE TIMES			
Batches per day	4.00	per SBR	
Complete Cycle time	6.00	hrs. per basin	
Fill time at ADF	2.00	hrs.	
Anoxic Fill time	1.50	hrs.	75 % of FILL is anoxic.
Aerated Fill	0.50	hrs.	
React time	1.81	hrs.	39 % of cycle is aerated.
Denite time	0.50	hrs.	
Settle Time	1.00	hrs.	3.7 hrs. anoxic per cycle
Decant time	0.60	hrs.	
Idle time	0.09	hrs.	2.3 hrs. aerated per cycle

Their recommendation is for a 6 hour cycle. 4 per day per basin, with 3.7 hours of anoxic time per basin per cycle.

The SBR cycle is entirely adjustable and customizable, which provides flexibility to optimize the process and adjust for changing conditions or performance requirements. The key conclusion is that however the cycle is operated, the selected process volumes, 750,000 gallons of gross tankage per half a million gallons per day in flow, is sufficient to meet the discharge requirements.

Denitrification-Adequacy of Carbon Source

Biological denitrification is accomplished by creating anoxic conditions in the nitrified mixed liquor. In the absence of free dissolved oxygen, the biota of the mixed liquor will turn to the molecular oxygen contained in nitrate. To access the oxygen bound in the nitrate, and be assured that biological denitrification can occur, there needs to be an adequate carbon substrate.

The adequacy of the substrate is typically checked by noting the ratio of COD to TKN. Denitrification is expected to occur most efficiently at ratios of 14 and above.

In the instant case, for the domestic wastewater plant, a normal domestic wastewater is anticipated with a design BOD of 203 mg/L, a corresponding COD of 406 mg/L, and a TKN of 40 mg/L. (406/40 =10.1)

It is deemed that the domestic wastewater influent will not have adequate carbon for denitrification. Based on the foregoing, a carbon supplement is deemed needed.

Methanol has been commonly used in the past, but due to safety and handling issues and the wide success of glycerin as a substitute, a glycerin (sugar water) feed will be provided to the SBRs

Noting that while the influent can reach 203 mg/L BOD, which is an important consideration for sizing the air supply system, it averages closer to 137 mg/L. This in turn indicates a potentially low COD of 274 mg/ and greater need for a carbon supplement..

Total carbon supplement feed is estimated as follows per half million gallons per day in flow:

Supplemental Carbon Requirement

Flow	MGD	0.5
(1) soluble COD required = 8 x No3-N load	mg/L	263
(1) Total COD required, from nitrate load	mg/L	798
(2) Total COD = TKN x 14	mg/L	560
Design COD =	mg/L	798
Design Soluble COD	mg/L	263
Available Total COD	mg/L	274
Soluble COD available	mg/L	90
Deficit =	mg/L	173
Lbs per Day Supplemental COD needed	mg/L	720
mg/L COD in 50% sugar solution	#/day	685000
gal/day required		126

(Twice the gallons per day is indicated potentially needed at 1 MGD)

Process Alkalinity

The process of nitrification consumes alkalinity. As alkalinity is consumed, the mixed liquor becomes progressively more acidic. The influent alkalinity data and the analysis below shows that addition of alkalinity is likely necessary to ensure a stable pH in the mixed liquor. Calculatons and usage estimates are as follows:

Alkalinity

Flow Rate (MGD) =	MGD	0.500	1.000
Influent Alkalinity:	mg/L	200	200
Influent TKN:	mg/L	40	40
Target Effluent NO3-N:	mg/L	2	2
Alkalinity consumed by nitrification:	mg/L	149	149
Residual Alkalinity	mg/L	50.9	50.9
Target Desired Residual Alkalinity	mg/L	100	100
Deficit of Alklinity	mg/L	49.1	49.1
Required Dose NaOH mg/L/mg/L deficit		0.799	0.799
Requred Dose, NaOH	mg/L	39.2	39.2
# NaOH needed/day	#/day	164	327
Estimated Liquid Vol gal/day	gpd	25.01	50.03

This is resupplied with liquid sodium hydroxide (or alternatively, soda ash solution).

4.8.3 BOD Removal

From the table of design calculations in the appendix of this report, the loadings on the proposed SBR should produce an effluent less than 20 mg/L BOD. The long solids retention time coupled with adequate air supply used in this process tends to produce a fast settling sludge, with a clear supernatant, and very low in soluble BOD.

4.8.4 TSS Removal

From the table of design calculations in the appendix of this report, loadings on the SBR should produce an effluent less than 20 mg/L TSS in effluent removed during the Decant Step. The long solids retention time used in this process tends to produce a fast settling sludge, with a clear supernatant, very low in solids content.

The existing treatment plant has (7) gravity sand filters. These were originally installed for the purpose of meeting a 5 mg/L TSS standard when the effluent was reused on the South Golf Course in the 1990s. They are still in use. These will be maintained as an additional treatment step in Phase 4B.

In Phase 5, these will be replaced with (3) dentrification filters to assure the plant is capable of meeting a 3 mg/L TN standard. The filters used will also produce a low TSS.

4.8.5 Phase 5 Denitrifcation Filtration

The SBR process is expected to meet a 6 mg/L TN standard as required for the permitted 0.495 MGD treatment plant. The SBR equipment manufacturer indicates their equipment can be operated to produce an effluent of less than 3 mg/L TN. Between the time the Phase 4B plant is placed into operation and before the Phase 5 plant will be in operation, there will be ample opportunity to optimize the operation of the SBR to produce an effluent with TN as low as possible without the need for additional treatment steps.

In the event the plant does not consistently meet a 3 mg/L target, the proposed permitted design for Phase 5 includes the addition of denitrification filters to ensure the effluent meets a 3 mg/L standard.

Denitrification filters are multi media filters intended for polishing fully nitrified effluent with a moderate level of nitrate remaining in order to get the final effluent down to a low level.

Theoretically capable of removing 15-20 mg/L nitrate, more reliable, consistent operation is expected when they are used to polish a lower nitrate load, 6-10 mg/L.

In the phase 5 plant, decanted effluent flows to decant flow equalization unit process, from which pumps will pull water to alternately dose three parallel denitrification filters.

While similar in many respects to a conventional gravity sand filter for TSS removal, there are important differences. The media is deeper and selected for denitrification. The hydraulic design maintains parts of the media to remain saturated. A glycerin feed is provided to enhance denitrification. In addition to a standard air scour and water backwash cycle, a dentrification filter has a bump cycle (water flow only) to release entrained nitrogen gas. Filtered water flows to a the chlorine contact chambers, which are arranged and valved so water can be retained for filter bump and backwash cycles.

Water enters each filter over a laminar flow baffle to induce smooth, non turbulent flow of water downward. The purpose of this is to avoid entrainment of air in the water.

As the water flows into the media, it will pond, the depth of ponding increasing as more solids are captured, which increases filter headloss. During the start of the filter run (a filter run is the time water is loaded into the filter and when the filter needs to be back washed to eliminate captured solids, reduce head loss and depth of ponded water), water will pond a few inches. Over several hours, this may increase to over a foot or more.

Part of the media is designed to remain permanently saturated. This induces anoxic conditions in the pore spaces of the filter. A denitrification reaction will then start, causing bubbles of nitrogen gas to form in the media.

Water drains from the filters into a manifold at the bottom of the filter, then to the chlorine contact chambers.

A filter bump cycle is necessary to purge the filter of entrained nitrogen gas. The operation is automatic. When a bump cycle is called for, the pump or pumps sending water to the filter being bumped is shut down, and the other filter pumps will send water to the other filters. A backwash pump is started to send water to the filter being bumped. This sends a reverse flow of water up from the bottom of the media, slightly expanding the bed and releasing nitrogen gas. Backwash water exits the top of the filter via an overflow and then drains by gravity to the mudwell tank.

The backwashing of the filters is also controlled automatically and can also be triggered manually. Backwash also shuts down the dose pump feeding the filter being back washed. At the start of the cycle, the filter is air scoured for several minutes. This is then followed by water backwash pumped by the backwash pump, As with the bump cycle, backwash water exits the filter overflow and runs into the mud well tank.

As the mudwell tank fills, smaller transfer pumps return the water to the plant flow equalization tankage.

The denitrification filters used will be of a proprietary, manufactured design, by Leopold or equal.

Sizing calculations are provided in the appendix titled "Denitrification Filter Calculations".

Denitrification in denitrification filters requires there be an adequate level of organic carbon in the influent water for use as a substrate. Since following treatment upstream all the organic carbon has been used, a carbon supplement is deemed needed. As with the supplement used in the SBRs, a glycerin feed will be dosed into the feed going to the filters. The feed will use typical peristaltic chemical to dose solution to the filters.

4.8.6 Phosphorus Reduction

In both Phase 4B (0.495 MGD) and Phase 5 (1 MGD), phosphorus will be reduce by chemical precipitation. Theoretically the batch cycle in an SBR can be configured to induce anaerobic conditions needed for phosphorus reduction; after the phase 4B plant is constructed, it may be useful to program a custom cycle to try that, however, for simplicity and reliability, a chemical alum feed is designed to dose each batch in the SBR for the reduction of phosphorus. Estimated dosage is as follows

Design Flow	**MGD**	**0.5**	**1**
Influent TP	mg/L	6	6
Effluent TP	mg/L	3	1
Influent P - Effluent P	mg/L	3	5
Dosage, 1.3 mg Al per 1 mg P removed	mg/L	3.9	6.5
Consumption, Alum	#/day	16.3	54.2
AlPO4 Produced:	mg/L	11.8	19.7
Al(OH)3 Produced:	mg/L	3.9	6.4
Total Produced:	mg/L	15.7	26.1
Estimated Liquid Vol gal/day	gpd	2.49	8.29

4.8.7 SBR Control System

The SBR control system will be furnished as a manufactured unit with the SBR equipment by the SBR equipment manufacturer. The proprietary name of the selected control system is the Parkson DynaPhase™ Controls package.

The SBR controls will all be housed in a single cabinet adjacent to the SBR plant structure. Sheet M13 of the accompanying permit drawings provides the general arrangement of controls within the cabinet. It should be noted there will be one control cabinet for the (3) SBR chambers associated with Phase 4B and one control cabinet for the (3) SBR chambers that are added to the plant in Phase 5. In this manner, two phases operate in parallel with one another.

The operator can select each SBR to be in either Manual or Automatic. When an SBR is in automatic, the control system will call each automatic valve to open or close as required and call each pump and blower to run or not run based on the current treatment step. When an SBR is in manual mode, the control system will not call any valve to open or any pump or blower to run.

Each mechanical pump, blower and motorized valve has a corresponding Hand Off Automatic control selector in the control panel.

For the majority of its operating time, the SBR is expected to be in automatic mode.

In automatic mode, the operator can select certain parameters to control the duration of time or depth of water that is associated with the batch reactor Step.

The following discussion is taken in large measure from Parkson's controls system overview documentation.

Selection of parameters is through the plant HMI panel in the control cabinet. Parameters in some cases relate to level of water in the SBR (sensed through a level transducer and some are time based. These parameters are as follows:

Maximum Fill
The operator selects the maximum number of minutes the control system will allow for fill (**Anoxic Static Fill, Anoxic Mixed Fill**, and **Aerated Fill**) by adjusting the Maximum Fill Time set point. There are separate Maximum Fill Time set points for two-tank, and three-tank operation.

Anoxic Static Fill Percent
The operator has the ability to separate the anoxic fill into Anoxic Static Fill and Anoxic Mixed Fill. In single tank mode, Continuous Feed % determines the percent of the calculated anoxic fill time that will be static (no mixing). The remaining anoxic fill time will be Anoxic Mixed Fill. In sequencing mode, Tank % determines the percent of the calculated anoxic fill time that will be static (no mixing). The remaining anoxic fill time will be Anoxic Mixed Fill. Each tank has an individual set point, allowing the operator the flexibility to select different anoxic static fill times for each tank.

Aeration Setpoint/Operation
Once the anoxic time is complete, the tank will enter aeration, which is split into Aerated Fill and React. An operator can shift aeration time into Aerated Fill by adjusting the Maximum Anoxic Fill set point. The system calculates the required aeration time based on the current percent of design flow and the aeration set points entered by the operator. Aerated Fill is the time remaining after completion of Anoxic Fill and will last until the Maximum Fill time expires, calculated air time expires or the level reaches Top Water Level (TWL). Once one of these three conditions has been met, the SBR enters React and attempts to remain in React until the required aeration time or minimum react time is complete. The second anoxic Step, is set by the operator through the HMI for timed operation.

Settle Set Point
The Settle set point allows the operator to adjust the duration of the settle step. The time that the operator has entered into the Settle set point begins at the beginning of Settle Prep. The actual settle prep duration falls within the settle step. For example, if the operator enters a five minute settle prep and a 45 minute settle, mixing will occur for the first five minutes of settle and will be off for the remaining 40 minutes (45 – 5).

Decant Set Points
Decant is not, by design, a timed treatment step. When an SBR tank enters decant, the control system will monitor the water level in the tank. When the water level reaches the bottom water

level (BWL) set point, decant is terminated

Idle or Wasting Duration

The operator can enter set points for the number of minutes (Waste Sludge Time) and the volume (Waste Sludge Volume) to waste sludge from each SBR. Individual set points are provided for each tank, allowing the operator the flexibility to select different waste sludge times and volumes. Both the time and volume set point will always be utilized.

See also section 4.15, prevention of upsets, which discusses the control system's response to a component failuure.

4.9 SBR Aeration System

Air supply to the SBR basins is provided by air compressors. Aeration required is calculated in the appendix of this report, both by EOR (357 SCFM per basin) and the Parkson Corporation (359 SCFM per basin). Air supply specified is 359 SCFM per basin.

There are four blowers connected to a common manifold. Three blowers are provided, one for each basin, with one redundant backup. Air is admitted to each basin by a motorized valve operated by the SBR control system. The air distribution system in the SBR basins is a proprietary jet aeration system by the Parkson Corporation.

4.10 SBR Recirculation Flow

The biomass in each SBR has to be mixed and kept in suspension, particularly during anoxic denitrification and aeration Steps. A motive pump for each basin is used to withdrawn biomass from the basin and recirculate it back into the basin through a jet distribution system.

The Jet aeration and mixing system is a proprietary system; Parkson's calculations for this system are provided in the appendix, and call for each motive pump to have a capacity of 1465 gpm at 17 feet TDH.

4.11 Chemicals Used

The following chemicals will be used by this facility:

- glycerin, as a carbon feed supplement to the treatment process (see 4.8.2)
- Sodium Hydroxide, to balance pH and restore alkalinity (see 4.8.2)
- alum, used to precipitate phosphorus (see 4.8.6)
- Chlorine solution as a disinfectant (see 4.12)

Calculations for the dosages are included/referenced in the calculations provided in the appendix

All chemicals used will be in the solution form, commercially mixed and delivered to the site by chemical supplier. Solution will be stored in polyethylene containers, with drain and level markings, and set on appropriately sized secondary containment pads. Solution is administered by peristaltic feed pumps.

Sodium hydroxide (or soda ash) will be dosed into the raw wastewater pumped to the SBRs when the surge pumps are turned on.

The alum and glycerin feeds will be initiated by the SBR control system

Chlorine will be dosed into the water pumped to the chlorine contact tank and initiated with the pump engagement.

4.12 Pretreatment, Influent and Decant Equalization, Chlorination, Sludge Digestion

This section describes the design basis of the side stream and supporting unit processes.

4.12.1 Pretreament-Screening

At this writing a minor modification permit application is under review to modify the headworks of the treatment plant, provide grit removal, and flow equalization.

As described in the letter report supporting the minor modification, the short term loading on the existing Phase 3 headworks coarse bar rack is 300 gpm more or less from existing development lift stations, plus will be 600 gpm more from short term proposed development. The existing bar rack cannot handle this. The bar screen will be replaced with a new hydrostatic screen with a flow thru capacity of 1500 gpm in Phase 4A.

For Phase 5, 1 MGD, peak rates of inflow are forecast to be as high as 2000 gpm: a second parallel screen will be added at that time to provide a total of 3000 gpm in screening capacity.

4.12.2. Pretreatment Grit Removal

The conversion of the grit chamber is covered in the current minor modification of facility permit application under review. The following describes the work to be performed.

The existing grit chamber is a 10 foot diameter circular wet well, which does not function as a grit chamber so much as it functions as a trash trap. In Phase 4A, the well would be pumped out, internal components removed, and then reused to support the platform for a hydrostatic screen.

While grit removal is desirable in the existing plant, it has heightened importance in Phase 4B and Phase 5.

A rectangular digester chamber in the 1970s plant (from Phase 2) would be repurposed for grit removal with new aeration to promote grit settling and removal with suitable eductors or direct pumping by sludge haulers. The interior of the tank would be partially grout filled and formed to create a grit collection hopper.

The accompanying drawings sheet M4 depict how the conversion is to be carried out.

Calculations sizing the grit chamber are provided in the appendix of this report.

From the grit chamber, wastewater would flow to the flow equalization tankage.

4.12.3 Influent Flow Equalization.

In Phase 4A, where the existing phase 3 treatment plant is still in use, flow equalization is needed to ensure that that peak hour flows from new development do not excessively load the existing final settling tanks.

For Phase 4A, flow equalization is needed to attenuate the load on the existing phase 3 clarifiers and treatment plant. One phase 2 diffused aeration tank of 45,970 gallons is used for this purpose. Two flooded suction dry mount surge pumps are installed to pump to a flow splitter box , which regulates the flow to the Phase 3 plant in service.

For Phase 4B and Phase 5, flow equalization is needed to enhance the reliability of the Sequencing Batch Reactor Process.

For Phase 4B, the design consists of repurposing the diffused aeration chamber of phase 1 and adding it to the converted flow equalization tank in phase 4A. This provides 91,940 gallons in volume to attenuate peak flows of just more than three times the average daily design flow (0.495 MGD) to less than 1.5 times the average daily flow. The spitter box is removed, a third pump is added the two installed in Phase 4A. Pumping rate to the plant is adjustable with the type of belt driven pump used, and is selected so that the rate pumped to the SBR is not more than 1.5 times the design average flow.

For Phase 5, two out of service aeration tanks in Phase 2 are added to the two equalization tanks already converted in phase 4A and Phase 4B. Each tank is 52,050 gallons and the combined total of equalization volume available is 196,040 gallons.

All flow equalization basins will be aerated to control odors.

Calculations for sizing the chambers, the pumps and aeration needed are provided in the appendix of this report.

4.12.4 Decant Flow Equalization

Decant Flow Equalization is provided in phase 4B and phase 5. Unlike a conventional, continuous flow plant, the effluent discharged from the settling cycle is not continuous but released in batches in large volume over a short period of time. This can cause short circuiting of chlorine contact time, overload existing filters used in Phase 4B and the denitrification filters proposed for Phase 5. Peak rates of decant flow are 1389 gpm in Phase 4B and 2778 gpm in phase 5.

In Phase 4B Existing Phase 1 final settling, digestion and one aeration compartment will be utilized for decant equalization, providing 90,130 gallons in volume. Phase 5, an additional 52050 gallon Phase 1 aeration tank is converted for decant equalization.

Referencing the calculations provided in the appendix, this is more than enough volume to equalize the flow coming out of the SBRs for both phases.

Two 500 gpm flooded suction pumps are used to pull water from decant equalization and transmit to downstream unit processes in Phase 4B. In phase 5 a third pump is added.

4.12.5 Chlorine Doses, Residuals and Contact Times

For phase 4A and 4B, the existing (7) parallel sand filters, clearwell, and chlorine contact tank are maintained in operation. Total volume for phase 4B is 19500 gallons in this un modified part of the plant.

For Phase 5, the existing filters are demolished, the existing mudwell is demolished, and the existing clearwell is converted to a chlorine contact tank, of 25,000 gallons and operates in parallel with the 19500 gallon CCC.

For handling safety and other reasons, chlorine will be used as a disinfectant in its liquid form, rather than as a gas.

Detailed dosage, residual and contact time calculations are provided in the appendix of this report.

The dosage is computed as 8 mg/L, the desired residual is 0.5 mg/L. Minimum chlorine contact time at peak flow exceeds 15 minutes, and exceeds 30 minutes at average flow in both Phase 4B and Phase 5.

4.13 Biosolids Storage, Treatment, and Disposal Plan

Sludge wasted from the SBR process is sent to an aerobic digester. In both Phase 4B and Phase 5, the digester is converted tankage repurposed from the presently in service Phase 3 plant.

For Phase 4B, an existing aeration tank of 123032 gallons, an existing diffused aeration tank of 12951 gallons and the existing 28482 gallons digester (164,465 gallons total) will be used to process waste sludge from the SBR.

For Phase 5, the other two mechanically aerated aeraton chambers of 123032 gallons each will be converted for sludge digestion.

The conversion is fairly elementary: The larger aeration tank will continue to be mixed with its mechanical surface aerator, and the other tanks will use the existing diffused aeration system already installed in them. Tank outlets to former settling tanks will be closed off. Submersible portable electric pumps will be used to remove supernatant and pump to the flow equalization tankage.

The primary purpose of the aerobic digester is sludge holding prior to removal, provide sludge stabilization, and for additional decanting to thicken the sludge and reduce the volume that has to be hauled.

In Phase 4B, the digesters have a total capacity of 164,465 gallons. Referring to the calculations in the appendix of this report, at 0.495 MGD the theoretical waste sludge flow is 9373 gallons per day. Thickening to 1% solids creates a theoretical supernatant flow of 5836 gallons per day. The supernatant from this unit process is returned to the head of the treatment plant and the flow equalization tank.

Considering the volume recovery associated with supernating, the sludge digestion tankage should hold 46 days of waste sludge flow.

For Phase 5, with the additional volume but higher flow rate, the sludge holding capacity is 61 days.

Oxygen requirements for the digester have been determined based on 2 lbs/O2 per lb. of VSS destroyed or 30 scfm per 1000 cf, whichever is greater.

Calculations are provided in the appendix of this report.

Supernatant will be returned using adjustable air eductor and sent to surge. A gravity overflow back to flow equalization will be provided.

Sludge from this facility is and will be removed by A-1 Quality and processed in their biosolids treatment facility.

4.14 Operational and Control Strategies

An O&M manual is to be provided which will cover all aspects of plant operation, including normal operation, preventative maintenance, problem diagnosis and recovery.

The primary control strategies from the site operators point of view are:

- Setting the flow equalization pump rates and controls to maximize flow attenuation on downstream unit proceses

- Field testing mixed liquor for dissolved oxygen, settleability, and the effluent for pH, chlorine residuals and nitrogen species as discussed in 4.8.2.

- Adjusting the plant aeration react times as testing and manufacturer O&M direction indicates needed, wasting sludge to the digester to maintain an appropriate sludge volume in the plant, and adjusting all chemical dosage and operation to assure meeting treatment and disinfection standards.

- Logging daily activities as required by permit in a logbook.

- Carrying out necessary repairs and routine preventative maintenance to essential mechanical and control equipment

- The SBR cycles and operating sequence can be validated and optimized using suitable field test kits to measure

  1) plant dissolved oxygen,
  2) effluent ammonia content
  3) effluent nitrate content, and
  4) running daily settling tests.

  The DO tests confirm the plant oxygen level when the blowers are running, confirm the DO level when anoxic conditions are induced. About 2 mg/L DO is expected during the on cycle, and less than 0.2 mg/L after 15 minutes is expected during the off cycle.

  The ammonia test kit is to check to make sure that complete nitrification occurs. Results of less than 1 mg/L ammonia shoud be expected. If significantly high ammonia test results are encountered, aerator "on" time must be increased

  The nitrate test kit will check the adequacy of the denitrification cycles. Typical results will normally be in the 4-6 mg/L range, and the operator should strive for 3 mg/L as deemed practical by the SBR equipment supplier.

After initial plant startup to develop a biomass, field testing should be daily to establish trends and adjustments made to the batch cycle.

4.15 Prevention of Upsets

Only domestic wastewater flows to this treatment plant, so upsets from industrial sources are not expected.

Most important in the prevention of upsets is the ability to prevent hydraulic overloads and maintaining the plant biomass volume within an acceptable range, and to assure its settle-ability by proper aeration control.

This plant is to be equipped with a surge or flow equalization tank to prevent hydraulic overload. Additional information on the features of this tank are covered in section 4.12.3

The contracted operator will have appropriate test equipment for measuring sludge volume, plant DO and effluent field testing as described in the foregoing sections.

In addition, the SBR control system has operator selectable automatic protocols that are followed during the detection of faults. The following is taken from information furnished by the Parkson Corporation:

The operator can choose for an automatic component failure response to be either disabled or enabled for each SBR. If disabled, the control system will generate an alarm and continue to cycle the SBR when an alarm occurs. If enabled, the control system will decide if the alarm is critical or non critical and take the appropriate response.

If a critical alarm is detected, the control system will sound an alarm and indicate which piece(s) of equipment has failed. The operator has five minutes to correct the problem or to disable failure response for that tank. If the operator has not cleared the alarm or disabled failure response within five minutes, that tank will be taken out of service (Failed Off) until the operator clears the alarm and places the tank back into service. The only exception to this is if the tank with the failure is the only tank in operation. The control system will not automatically take all tanks out of service.

To put a tank back into service that has failed off, the following sequence must be performed:
1. The alarm must be cleared
2. The tank selector must be turned to Manual
3. The tank selector must be turned to Auto

Typical critical alarms are influent valve failure, air valve failures, effluent valve failures, and blower failures.

If a non-critical alarm is detected the control system will sound an alarm and indicate which piece(s) of equipment has failed. The tank will continue to cycle and the alarm will be cleared once the alarm has been acknowledged and the failure no longer exists.

4.16 General Construction Features

New tankage is to be constructed from poured in place concrete for durability, low maintenance and longevity.

Liquid piping will be generally be PVC schedule 40, except where steel or ductile iron is specified or required.

Aeration will be supplied via diffused aeration from compressors, using the SBR manufacturer's proprietary jet aeration system. Air transfer piping is sized to keep velocities low so that head loss is limited to 1 psi.

Aeration transfer piping will be fabricated steel or galvanized schedule 40 steel for service life in the sun and coated for protection from corrosion.

Multiple positive displacement blowers are provided for redundancy, each equipped with a motor and pulley system to set proper blower speed.

Control systems will be in weather proof control panels, NEMA 4 rated. Motor controls include HOA switches for manual or automatic operation. Controls include protection from transient voltage surge and lightning strike suppression.

Tank grating and walks between or over tanks and access stairs of aluminum construction are provided where equipment access may be required.

Specified pumps will all be solids handling pumps, designed for use with wastewater solids and raw wastewater.

Additional remarks are provided below in section 4.18

4.17 Flow Metering and Measuring

Flows to this facility are presently measured and will continue to be measured by an effluent flow meter measuring the rate and cumulative volume of flow per day pumped from the plant pump tank to the rapid infiltration basin for all Phases.

4.18 Reliability Classification

Flow Equalization

Flow equalization is used to control the hydraulic loads on downstream unit processes.

The basic concept is to be able to accept incoming instant rates of flow in excess of 300% of design capacity, then store and equalize the flow so that only 150% is discharged to downstream unit processes, and ensure those units processes can handle 150% of design flow

The surge tanks and pump systems are often inline with the flow. Two pumps are installed in Phase 4A which pump to a flow regulator box. Normally one pump at a time is running, the other is a redundant backup. For phase 4B and phase 5, a third pump is added. Up to two pumps are needed, with the third a redundant backup. All will be setup to alternate duty points.

In phase 4A, a splitter box is used to send a measured amount of flow forward into the Phase 3 treatment works and a certain amount is returned to the surge tank. This method provides a more continuous, controllable way of feeding the treatment plant. This is not required in Phase 4B and phase 5.

The surge tank is aerated to help control odors.

Only one surge tank is required to meet Class III reliable criteria. The dual surge pumps, each 100% redundant, meet the class III reliability requirement.

Pretreatment

The purpose of pretreatment facilities such as screens is to prevent the entry of objects into the treatment process that would adversely effect plant operation such as by causing pumps to clog, etc. For this purpose, a single hydrostatic screen is proposed for Phase 4A ad phase 4B, and two screens each capable of handling at least 50% of the flow is provided in Phase 5.

Multiple racks are not required for Class III reliable systems, but a bypass is provided. The proposed screening system is Class III reliable.

Aeration/Bioprocess Tankage

New aeration or bioprocess tankage consists of (3) SBR basns in Phase 4B and (6) in Phase 5. .

Aeration/Process Tankage having multiple independent tankage, meets or exceeds Class III reliability

Air supply for the aeration process, is diffused aeration with supply for air compressors. Four are provided in Phase 4B, with one redundant, each valved to the main header, are provided for reliability. For Phase 5, there will be (8) compressors, two of which will be redundant backps

Air compressors, considering size, horsepower, and future requirements are of the positive displacement type. Intake and discharge silencers are to be required Compressors are belt driven. . Air volume is controllable by varying the drive pulley size. Operating at low noise level with sound insulating house will be required

Aeration Supply Compressors meet or exceed Class III reliability

Filters

Effluent filtration prior to disinfection is provide, although not required for effluent disposal to a rapid rate system. The existing plant has (7) sand filter units which will not be modified in Phase 4A and Phase 4B. Any unit can be removed from service and still maintain filtration.

For Phase 5, these will be demolished and (3) new denitrification filters will be provided. Any one can be removed from service and the remaining units capable of handling at least 50% of the total flow.

Decanted flow into the decant equalization unit process which has (2) pumps phase 4B and (3) pumps in Phase 5 to send the water to the filters. Any one can be removed from service and 100% flow maintained. Filters are backwashed by one of two alternating pumps, either of which can be removed from service and maintain backwash capacity 100%.

The Phase 5 filters provided meet or exceed Class III reliable criteria.

Chlorine Contact Tanks

Settled water is to be disinfected prior to discharge to the reuse/disposal system.

For Phase 4A and 4B, there is no modification proposed of the existing system which has a single chlorine contact in series with a clearwell tank.

For Phase 5, the clearwell tank is converted to a parallel chlorine contact tank.

The chlorine contact chambers for the Phase 5 treatment plant therefore consists of two chambers, each sized to at a minimum, provide adequate contact time at peak flow for, at a minimum, 50% of the flow.

The chlorine contact chambers provided meet or exceed Class III reliable criteria.

Standby Power

This facility electrical design will include provision of a permanent generator to to provide power to the entire plant in the even of power outage.

5.0 OUTFALLS

This facility has no existing or proposed surface water outfalls.

6.0 EFFLUENT DISPOSAL OR REUSE SYSTEM

Effluent from the existing treatment plant is disposed or reused by (4) existing rapid infiltration basins with net permitted capacity of 0.340 MGD. These basins are numbered 1 to 4; their general location and arrangement is shown on sheet C1 of the accompanying permit drawings.

It is proposed to eliminate existing basin no 4, expand basin 3, and consolidate and expand basins 1 and 2 into one basin, so the facility has a net reuse / disposal capacity of 0.495 MGD.

Sheet C7 provides an overview of the proposed rapid infiltration basin expansion and configuration. The expanded and consolidated infiltration basins are designated North R.I.B., and South R.I.B.

6.1 Project Area Features and Land Use

Land Use

The area proposed for the treatment plant is the current treatment plant site: construction of new tankage occurs in an area immediately adjacent and was used for general maintenance purposes, with a couple of unused offices and warehouse buildings which will be removed.

The South Rapid Infiltration Basin is in the same area immediately south of the plant and presently encompasses the existing number one and two infiltration basins.

The North Rapid Infiltration Basin encompasses the number 3 infiltration basin and will also use existing cleared area that was formerly used as a golf course (currently not in use and at this writing not expected to be a golf couse in the vicinity of the North RIB.

Setbacks

Reference is also made to the construction drawings- see sheet C7, Reuse Plan, which shows the setbacks to various features with a combined furnished survey and aerial view. Setbacks are provided to residential property lines and occupied buildings of at least 100 feet and to ROWs of more than 50'. Setbacks of 500' to known private domestic wells, as identified by the geotechnical consulting engineer (reference geotechnical report).

Flood Plain and Site Drainage

As discussed in section 3.1 of this report, the existing treatment plant, proposed plant improvements, and proposed rapid infiltration basin systenm improvements, are outside the limits of a flood zone A or AE. (The northern part of the plant and the north R.I.B are located on community panel 12105C0385H, and the southern part of the plant and southern R.I.N. are found on community panel 12105C039H.

All construction occurs largely within an existing grassy are. In the southern basin, there is necessary removal of a building and some asphalt. Clearing of trees or vegetation around existing basins 1,2, and 3 will be minimized to help keep the area screened. The effluent infiltration basins are designed so that any rainfall that falls within the basins remains entirely within the basin and the basin berms are graded in a way to prevent the entrance of stormwater.

Topography

Terrain at the project site is more or less level with a slight overall slope from West to East. From available topographic mapping it appears to vary between elevation 80 and 74, with much of the site outside the limits of existing infiltration basins around 75.

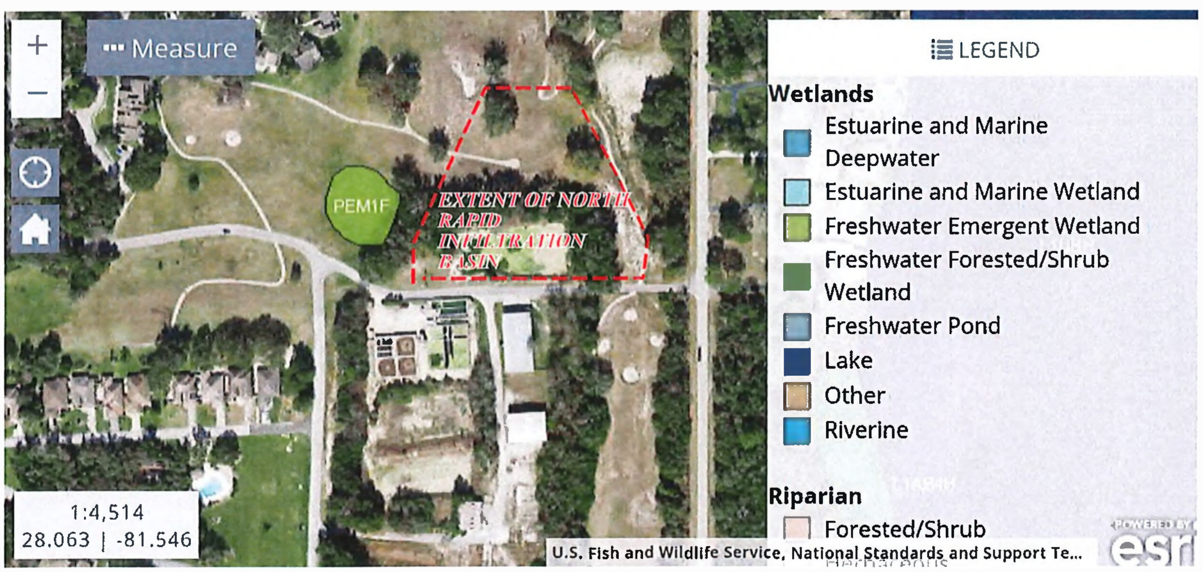
Vegetative Community

The proposed effluent reuse disposal modification or expansion is largely planted with grass at this time; in the north RIB area this somewhat overgrown former golf course turf. As noted above, clearing of trees is limited to what surrounds the existing basin preserving as much as possible.

Wetlands

There are no wetlands within the proposed construction area that would be subject to grading, cut or fill earthwork operation.

According to the National Wetland Inventory mapping, there is circular area on the former golf course west of the North Rapid infiltration basin site designated as an emergent wetland. No impacts to this area are proposed by this application.



6.2 Local Water Wells

Refer to the accompanying geotechnical report which identifies drinking water wells in the vicinity of the proposed effluent disposal system.

Homes within Grenelefe are served by the public water system operated at Grenelefe. The private utility has two supply wells. Referring to the mapping in the geotechnical report, the southernmost one is located approximately 2400 feet to the west/northwest of the Northern R.I.B. The northernmost supply well is located about 3900 feet to the North/Northwest.

East of the project area and east of Lake Marion Rd there are a couple of homes on private wells. The well locations were located by Andreyev Engineering using the Florida Department of Health database, and were field checked by Tract Engineering. Well locations are depicted in the geotechnical report (figure 9), and also shown on sheet C7 (containing Tract Engineerin's field locaates). The set back circles of 500' centered on each well are depicted; the proposed rapid infiltration basin system is at least 500 feet from known private wells.

6.3 Site Soils

See the accompanying hydrogeologic report for data and information about site soils.

6.4 Site Hydrogeology, System Loading and Proposed Capacity

See the accompanying hydrogeologic report for data and information about site hydrogeology, subsurface characteristics and hydraulic modelling.

The expanded and reconfigured rapid infiltration basins are intended to have a design capacity consistent with the objective of the Phase 4B expansion, 0.495 MGD.

The four existing effluent disposal basins ponds are rated for 340,000 gpd capacity on 100,188 sf of area, a loading rate of 3.39 gpd/sf.

The reconfigured system will have the Northern RIB at 2.593 acres of designed bottom area, and the Southern RIB at 2.37 acres of bottom area. Loading rate at 0.495 MGD on 216,188 SF is 2.29 gpd/sf (lower rate than as presently permitted).

6.5 Ground Water Monitoring Plan

See the accompanying hydrogeologic report for data and information about the proposed groundwater monitoring plan. The engineering permit drawings also show the location and construction features of the proposed monitor wells.

6.6 Construction Features

Effluent Rapid Infiltration Basins

Two ponds are proposed, to facilitate loading and resting on a 7 day load, 7 day rest cycle. Loading and resting will be accomplished with manually excercized valve operators.

Total pond depth is 6', based on the geotechnical engineering report bottom elevation of 75, and a top elevation of 81 to preclude entry of surface stormwater. The depth provides at least 1' in normal working depth, and in excess of 3' of freeboard above that as required by rule.

There is proposed an interbasin overflow pipe which interconnects the two basins, at elevation 78. Normally this would not be used unless one basin ponded to a depth of more than 3 feet.

Each basin also has an emergency overflow device located one foot below the top of each basin at elevation 80. Actually comprising two overflow pipes, one from each cell, any emergency overflow will be directed to the lower terrain off to the east.

All pond berms will be graded with 3:1 side slopes and sodded. Width across the top level berm is 8 feet.

Each basin has been designed with an effluent distribution system to discharge water at various points in each basin to spread the water out. (See sheet C7A and C7C of the accompanying permit drawings).

The rapid infiltration basins site will be fenced with warning signs posted to restrict public access.

Transfer Pumping

Efflient is currently pumped to the existing rapid infiltration basin; as water leaves the chlorine contact tank, it it drains to an effluent pump station. No changes are proposed for this system, except that existing effluent main piping will be connected to the proposed distribution piping within each expanded and reconfigured basin.

6.7 Conceptual Phase 5 Effluent Reuse/Disposal

This permit application is intended to permit treatment plant expansion through Phase 5 (1 MGD) capacity but proposed effluent reuse expansion is to be limited to Phase 4B (0.495 MGD).

Referencing the accompanying geotechnical report, "In addition to the evaluation of the existing RIBs in the general vicinity of the plant, two additional areas were assessed for potential additional RIB sites. The locations of these potential RIB sites were identified on portions of the defunct golf courses, which were in the western portions of the Grenelefe development substantially separate from the existing plant area."

These two areas were investigated to assess their soil and groundwater conditions. As the reconfiguration of the existing infiltration basin system was :

1) sufficient to handle foreseen needs through 0.495 MGD,
2) the need to provide 1 MGD in disposal capacity is conceptual only based on the developer's long term forecast of potential development, and;
3) use of these areas would require construction of a new effluent transmission system, it

It was determined to not develop a complete designed system of these areas at this time, and to do so at such time as clear definitive development plans and timetable warranted permitting same.

APPENDIX

SBR Design Calculations

Parameter	*Unit*	*Result Phase 4B*	*Result Phase 5*
I. Influent Parameters			
Influent Fllow, MGD	MGD	0.5	1
Influent Flow CF	CF	66845	133690
Bio/Chem Oxygen Demand:	mg/L	203	203
Infl Soluble BOD	mg/L	66.99	66.99
Infl COD	mg/L	406	406
Infl Soluble COD	mg/L	134	133.98
Total Suspended Solids:	mg/L	248	248
Total Kjeldahl Nitrogen:	mg/L	40	40
Total Phosphorus	mg/L	6	6
II. Effluent Parameters			
Effluent TKN	mg/L	1	1
Effluent Nitrate	mg/L	2	2
Efflkunt TN		3	3
Effluent BOD	mg/L	20	20
Effluent Soluble BOD	mg/L	2	2
Effluent TSS	mg/L	20	20
Effluent TP	mg/L	3	1
III Basin Geometry			
Flow	MGD	0.5	1
No of basins		3	6
freeboard	ft	1.5	1.5
Length	ft	42.5	42.5
Width	ft	42.5	42.5
Min Water Depth	ft	13.1	13.1
Max Water Depth	ft	18.5	18.5
Min Wet Vol Ea Basin	cf	23662	23662
Max Wet Vol Ea Basin	cf	33416	33416
Min Wet Vol Ea Basin	MGAL	0.177	0.177
Max Wet Vol Ea Basin	MGAL	0.250	0.250
Total Min Vol All Basins	MGAL	0.531	1.062
Total Max Vol All Basins	MGAL	0.75	1.50
IV SBR Cycle			
Flow	MGD	0.5	1
Influent Flow per hr	CFH	2785	5570
Influent Flow Per Fill Time/cycle	CF	8356	16711
Fill Events per Day, all basins		8	8
Cycles per Day per basin		2.67	1.33
Duration of Each Cycle	hours	9	18

Parameter	*Unit*	*Result Phase 4B*	*Result Phase 5*
Fill time	hours	3	3
React	hours/cycle	4.41	13.41
Settling Time	hours/cycle	0.75	0.75
Decant Time	hours/cycle	0.75	0.75
Idle/SludgeWaste	hours/cycle	0.09	0.09
Percent time of Fill time Aerated		20%	20%
Percent of Fill Time Mixed		80%	80%
Aeration Time in Fill	hours	0.6	0.6
Anoxic Time In Fill	hours	2.4	2.4
Percent time of React time Aerated		60%	60%
Percent of React Time Mixed		40%	40%
Aerated Time In React	Hours	2.65	8.05
Anoxic Time In React Mixed	Hours	1.76	5.36
Total Time Aerated/cycle	hours	3.25	8.65
Total Mxed Time /Cycle	hours	4.16	7.76
V. Decanter			
Decant Cyclers per Day		8.00	8.00
Volume Per Decant Cycle	gal	62500	125000
Decant Time per Cycle	hours	0.75	0.75
Decant Flow Rate	gpm	1389	2778
Total cycle duration	hours/day	3.0	3.0
Time Between Decant Flow Rate	hours	2.3	2.3
Min Requried Outflow Rate	gpm	463	926
Max Flow Rate to CCCs	gpm	521	1042
Selected Pump Rate to CCCs	gpm	500	1000
VI BioProcess Design:			
Process Mode		SBR	SBR
Temp		20	20
MLSS mg/L		3310	3310
SRT days		40	40
Yeild Coefficient		0.62	0.62
Total Required SBR volume		0.75	1.50
Food, BOD #/day	#/day	1034.16	2068.32
Mass, # MLSS	#	20699	41398
F:M		0.050	0.050
BOD Loading, #/1000 cf		10.32	10.32
Total V/Q, hrs.	hours	36.0	36.0
Min Required Time In Aeration			
Eqn			
V/(Q)=Delta BOD * yield * Min. SRT/MLSS		0.9	0.9
Total Aerated Time/cycle		3.2	8.6

Parameter	*Unit*		*Result Phase 4B*	*Result Phase 5*
VII Sludge Wasting				
WAS, lb/day	Lb/day		517	1035
Settling and thickening Factor			2	2
Settled Sludge Concentration	mg/L		6620	6620
Vol/wasted per day	gpd		9373	18746
VIII Nitrification Design				
Data:				
Min. Monthly Temperature =	degrees C		20	20
Min. Month M.L. pH			6.8	6.8
b decay	d-1		0.1	0.1
mu -a =			0.48	0.48
from eqn:				
mu-a = (a*exp(-b/(273+T)))/(1+(c/10^(-pH))+(10^(-pH)/d))				
a= 4.70*10^14				
b= 9.98*10^3				
c= 2.05*10^-9				
d= 1.66*10^-7				
(Antoniou et alia)				
TargetSnH				
KnH =	ppm		1	1
Target So= 1.0 ppm	ppm		0.4	0.4
KO,a =	ppm		1	1
		ppm	0.4	0.4
eqn.:				
1/SRT min = mu-a * (Snh/(Knh+Snh))*(So/(Ko,a+So)-b decay				
1/SRT=			0.147	0.147
SRT=	days		6.78	6.78
MLVSS/MLSS =			0.75	0.75
N Content of Sludge =	%		7	7
Eqn:				
N produced in SLudge = Yield*Delta BOD*MLVSS/MLSS*N Content in Sludge				
N in Waste Activated Sludge	mg/L		6.52	6.52
Eqn				
= N in Sludge - MLVSS/MLSS*(N Content of Sludge)*TSS	mg/L		6.46	6.46
Aerobic Digester SRT	mg/L		30	30
Process SRT	days		40	40
Total BioProcess SRT =	days		70	70
Yield @ Total SRT =			0.562	0.562
N Solubilized in Digester:				
eqn (Y -Y@total SRT)* (WAS N normal)			0.36	0.36

Parameter	*Unit*	*Result Phase 4B*	*Result Phase 5*
TKN Oxidized = Infl TKN - Eff. TKN - N in WAS + N in Digester	mg/L	32.90	32.90
Effluent TKN	mg/L	1	1

IX Denitrication Design

Nitrateload on Anoxic Process Eqn

N03-N Load = TKN Oxidized - effl. NO3+ So as NO3 +Infl. NO3

TKN oxidized	mg/L	32.90	32.90
Effl Nitrate	mg/L	2	2
So as NO3 = 0.3478 x So	mg/L	0.3478	0.3478
Infl Nitrate	mg/L	0	0
NO3N Load on Anoxic Cycle :	mg/L	31.24	31.24
Adequacy of Substrate			
Infl Soluble COD/ NO3-N =	(8 Min.)	4.3	4.3
infl tot. COD/infl. TKN =	(14 Min.)	10.15	10.15
Supplemental Carbon Requirement			
(1) soluble COD required = 8 x No3-N load		263	263
(1) Total COD required, from nitrate load	mg/L	798	798
(2) Total COD = TKN x 14	mg/L	560	560
Design COD =	mg/L	798	798
Design Soluble COD	mg/L	263	263
Avsailable COD in Raw wastewater	mg/L	406	406
Soluble COD available	mg/L	134	134
Deficit =	mg/L	129	129
Lbs per Day Supplemental COD needed	mg/L	539	1077
mg/L COD in 50% sugar solution	#/day	685000	685000
gal/day required		94	189
Dentrication Rate Constants			
Rsdn g NO3-N/(g MLVSS d)=	Phase 1 fast rate with adequate substrate	0.07300	0.07300
Redn g NO3-N/(g MLVSS d) (not used) =	Phase 2 slow rate *with < adequate substrate*	0.01536	0.01536
Anoxic Time, Fill cycle,		2.4	2.4
Anoxic Time, React Cycle	Hours/Cycle	1.8	5.4
Total Anoxic Time, Hour Cycle	Hours/Cycle	4.2	7.8
Eqn:			
(Volume /Q) = NO3-N Reduced/(Rsdn * MLVSS)	hours	4.14	4.14

Remarks:

Total Available Anoxic Time > Reqd Anoxic Time

Parameter	*Unit*	*Result Phase 4B*	*Result Phase 5*
Effuent Nitrate	mg/L	2	2
Alkalinity			
Flow Rate (MGD) =	MGD	0.500	1.000
Influent Alkalinity:	mg/L	200	200
Influent TKN:	mg/L	40	40
Target Effluent NO3-N:	mg/L	2	2
Alkalinity consumed by nitrification:	mg/L	149	149
Residual Alkalinity	mg/L	50.9	50.9
Target Desired Resdual Alkalinity	mg/L	100	100
Deficit of Alklainity	mg/L	49.1	49.1
Required Dose NaOH mg/L/mg/L deficit		0.799	0.799
Requred Dose, NaOH	mg/L	39.2	39.2
# NaOH needed/day	#/day	164	327
Estimated Liquid Vol gal/day	gpd	25.01	50.03
X. Phosphorus Reduction			
Design Flow	MGD	0.5	1
Influent TP	mg/L	6	6
Effluent TP	mg/L	3	1
Influent P - Effluent P	mg/L	3	5
Dosage, 1.3 mg Al per 1 mg P removed	mg/L	3.9	6.5
Consumption, Alum	#/day	16.3	54.2
AlPO4 Produced:	mg/L	11.8	19.7
Al(OH)3 Produced:	mg/L	3.9	6.4
Total Produced:	mg/L	15.7	26.1
Estimated Liquid Vol gal/day	gpd	2.49	8.29
XI SBR Aeration			
Influent BOD	mg/l	203	203
Influent TKN	mg/L	40	40
Effluent BOD	mg/L (soluble)	20	20
Effluent TKN	mg/L	1	1
Effluent Nitrate	mg/L	2.00	2.00
Q, MGD	mgd	0.500	1.000
eqn:			
*O2 lb/day = Q * 8.34 * [F*(So-S) + 4.6* delta TKN]*			
F=	(ref MOP8)	1.43	1.43
O2 #/d for BOD	lb/day	1095	2189
O2 #/hr for BOD	lb/hr	45.61	91.21
O2 #/d for TKN	lb/day	748	1496
O2 $/hr for TKN	lb/hr	31.17	62.34
Nitrate Reduction credit			
TKN oxidized (TKN In - TKN eff)	mg/L	39.0	39.0

Parameter	*Unit*	*Result Phase 4B*	*Result Phase 5*
O2 Released = 8.34 *Q* (2.86 *(TKN oxidized-Eff NO3N)	#/day	441.3	882.5
O2 Released, #/hr	#.hr	18.4	36.8
Total O2 =O2 BOD red+O2 TKN oxidation-O2 denite credit	lb/hr	58.4	116.8
N1 = O2 lb/day=	lb/hr	58.4	116.8
N1/N2 = (beta x Csw-CL)/Csxalphax theta ^(\T-20)			
alpha		0.85	0.85
beta		0.95	0.95
Csw		9	9
Co=	target DO	1.5	1.5
Cs		9.17	9.17
theta =	(Std)	1.024	1.024
T =	deg C	20	20
N1/N2 =		0.65	0.65
N2=	lb/hr	89	179
Diffuser Efficiency		0.08	0.08
SCFM Needed	SCFM	1070	2140
Process O2, lb/day		2820	5639
Process O2/kg/day		1280	2560
Diffuser Efficiency, %		8	8
Air Rqd., SCFM	SCFM	1070	2140
Air Reqired/Liters/sec		505	1010
Air supply, CF/# BOD		1820	1820
No of Basins		3	6
Air Flow per Basin	SCFM	357	357
Water Depth	ft	17.5	17.5
Air losses	psi	0.5	0.5
Compressor Discharge Pressure	psi	8.1	8.1

Sidestream Flow Equalization Other Processes Calculations

		Existing Plant Phase 4A	*Ph 4B 0.5 MG*	*Ph 5 1 MGD*
Aerated Grit Chamber				
Design Flow	MGD	0.34	0.5	1
Overeall length	ft	31.67	31.67	31.67
Cross Section Area	sf	37.39	37.39	37.39
Volume	gal	8857	8857	8857
Influent Flow Rate	gpm	1000	1143	2143
HRT	minutes	8.86	7.75	4.13
Horizontal Velocty	fps	0.06	0.07	0.13
Air reqd	cfm/ft	5.5	5.5	5.5
Air required	SCFM	174	174	174
Diffuser Capacity	SCFM	5-50	5-50	5-50
No of drops		5	5	5
No of Diffuser		10	10	10
Flow per diffusor		17	17	17
Air Header Dia	inches	4	4	4
Velocity	fpm	2661	2661	2661
Flow Equalization Tank				
Design Flow	MGD	0.34	0.5	1
Volume of Tank	Gal	91940	91940	196040
Vs/Q		0.270	0.184	0.196
10 States Peak Factor:				
equiv pop, thousands		3	5	10
Calc'd peak factor		3.4	3.2	3.0
Design Peak Factor		3.4	3.2	3
Design - OutFlow Peak		1.5	1.5	1.5
Peak Inflow to Plant	gpm	803	1111	2083
Theoretical Minimum Vs	Gal	65732	86665	153330
Fwd Flow to Plant	gpm	354	521	1042
Pumping Rate Rqd	gpm	521	521	1042
No of Pumps in Use		1	1	2
Total Pumps		2	3	3
Splitter Box Forward Flow	gpm	354	n/a	n/a
Return Flow	gpm	167	n/a	n/a
Air required	SCFM/1000 gal	1.25-2	1.25-2	1.25-2
Low Range	SCFM	115	115	245
Upper Range	SCFM	184	184	392
Selected Flow	SCFM/1000 gal	2.00	2.00	2.00
SCFM/1000 CF		15	15	15
Aeration Per Tank:				
Tank 1	Vol, Gal	45970	45970	45970
Air Required	SCFM	92	92	92

		Existing Plant Phase 4A	*Ph 4B 0.5 MG*	*Ph 5 1 MGD*
Tank 2	Vol, Gal	45970	45970	45970
Air Required	SCFM	92	92	92
Tank 3	Vol, Gal	0	0	52050
Air Required	SCFM	0.0	0.0	83.3
Tank 4	Vol, Gal	0	0	52050
Air Required	SCFM	0.0	0.0	83.3
Tank 1, no of drops		7	5	5
No of Diffuser		14	10	10
Flow per diffusor	SCFM	7	9	9
Tank 1 and 2 header Size	Inches			
Air Header Dia	inches	4	4	4
Velocity	fpm	1054	1054	1054
Post Settling Decant Equalizaton				
Decant Cyclers per Day		N/A8		8
Volume Per Decant Cycle	gal		62500	125000
Decant Time per Cycle	hours		0.75	0.75
Decant Flow Rate	gpm		1389	2778
Total cycle duration	hours/day		3	3
Time Between Decant Flow Rate	hours		2.3	2.3
Min Requried Outflow Rate	gpm		463	926
Max Flow Rate to CCCs	gpm		521	1042
Selected Pump Rate to CCCs	gpm		500	1000
Start EQ Volume	gal		0	0
Vol in during Fill	gal		62500	125000
Vol Out during Fill	gal		22500	45000
Max Volume In EQ End Fill	gal		40000	80000
Time Remaining Till Next Fill	hours		2	2
Time Rqd To Pump Rem Vol Out	hours		1.33	1.33
Decant Volume Avalable	gallons		90130	132180
Chlorine Contact				
Basin Volume	gal		19947	44947
HRT at Post Settling FEQ	minutes		40	90
Flow Rate if all Pumps running	gpm		1000	1500
Peak HRT	minutes		19.9	30.0
Cl2 Residual, mg/L			0.5	0.5
Cl2 Dose, mg/L			8	8
Consumption, lb/day			33.4	66.7
Hypochlorination System				
Est. Sodium Hypochlorite strength, %			12.5	12.5
Dose required, mg/L			8	8
Available Chlorine, lb/gal			1.04	1.04
dose, #/gal			0.00006675	0.00006675

	Existing Plant Phase 4A	*Ph 4B 0.5 MG*	*Ph 5 1 MGD*
Avg dose, #/day		33	67
Avg dose, gal/day		32	64
Residual * Detention		10	240
Aerobic Sludge Digestion:			
WAS Flow, gpd		9373	18746
Was Flow M^3/day		35.5	71.0
Total Solids,#/day		517.48	1034.96
Total Solids kg/day		234.94	469.87
WAS, mg/L		6620	6620
% Volatile		75	75
WASv, mg/L		4965	4965
Total VSS,#/d		388	776
VSS, #/Digester cf/day		0.02	0.01
Thick Solids,%		1.3	1.3
Digester Vol, gal		164465	410529
Digester Vol, M^3		622.50	1553.85
Initial Est.SRT, days		31	40
Temp, Degrees C		20	20
VSS Destroyed, %		34.52	38.92
Avg. Solids, mg/L		9100	9100
Supernatant Solids,mg/L		300	300
WAS Fraction Not Destroyed		0.74	0.71
WAS Fraction in Digester		0.38	0.36
Supernatant, gpd		5836	11987
Vol ASD /(Qwas in - Q super out), d		46	61
TSS in Digester, #		12482	31157
Total SS Removed, #/d		398	763
Supernatant TSS,#/d		14.6	30.0
Sludge Discharge,#/d		384	733
Sludge Rem/year, DTR		70.0	133.7
Sludge Discharge,gpd		3537	6759
Slidge Diischarged, M^3/d		13.4	25.6
Digester SRT, days		31.4	40.8
Sludge Stabiliz. Class		<B	B
Digester HRT, days		17.5	21.9
O2 Rqd, VSS, #/d		268	604
O2 Rqd kg/day		122	274
Air, SCFM		222	501
Diffuser Effic.,%		5	5
Air Rqd. Mixing, SCFM		660	1647
Design SCFM		660	1647
Design Air, l/s		311	777

Denitrification Filter Design

(Phase 5 only)

1.0 General Dosing and Sizing

1	= Total Flow, MGD
1.5	= Peak Inflow Rate
3	=number of filters
2	= number used
333333	=flow rate, single filter, gallon day
694	=design average flow rate into dose tank, gpm
1042	=design max inflow rate to dose tank, gpm
520.8	=selected pump rate, gpm, each filter
150	=area each filter,SF
3.47	=load rate single filter, pump running gpm/sf

1.1Filter Surface Loading

Solids, SLR

1	= Total Flow, MGD
20	=normal effluent TSS
300	=Total Filter Surface Area Loaded
0.556	= SLR, #/sf/day

1.2 Hydraulic Loading, QLR

3.472	=QLR, gpm/sf

1.3 Estimated Head Loss

Reynolds Number of Sand

Eqn Nr =grain diameter x filter flow velocity/ kinematic velocity

2	=d, grain diameter, mm
0.002	= d, grain diameter meters
3.47	flow rate, gpm sf
138.89	flow rate, Liters/ M^2-min
0.138888889	flow rate m^3/min
0.00231	velocity, m/s
0.000001003	kinematic velocity at 20 d C
4.62	= Nr

Drag Co Efficient

eqn Cd = 24/Nr+3(Nr^0.5)+0.34

6.94	= Cd

eqn Hl = 1.067/(Shape Factor) x Cd x (1/(porosity^4) x Filter Depth / (grain diameter) x fitration rate^2/accel due gravity

1	shape factor
6.94	=Cd
0.4	=porosity
6	filter depth, ft
1.875	filter depth, meters
0.002	=grain diameter, meters
0.00231	= filtration velocity, m/s
9.82	= accel due to gravity, m^2/s

0.148 = head loss, meter
0.473 =head loss, feet

2.0 Backwash Frequency and Filter Bumping

2.1 Formula : SSL x 517 /(TSS in - TSS out) x Q = Backwash Fequency

0.556 =SLR, solids loading, lbs/SF
517 = conversion factor
20 = TSS in
1 =TSS out (operational target)
1 = Q, MGD

15 = Backwash Interval hours

2.2 Backwash Cycle

Steps

1 start air scour, run for 150 seconds
2 start backwash pump, continue to run air until water reaches overflow; stop bower, duration, 120 seconds
3 continue to run backwash pump for 600 seconds
12 = total pump run time, minutes

Air Required

3.6 =rate. SCFM/SF of filter area
137 =SCFM per filter

Flow Required

6.0 = gpm/sf of filter area
900 gpm

2.2 Filter Bump Frequency

10 = NO3-in
1 =NO3-out, operational target
3.472 =QLR, gpm/sf
0.375 = [no3-in - no3out] x 8.34 x QLR / (10^6) x 1440 = Lbs NO3 per sf per day
0.1 = max denite specific filter capacity before bumping, lbs NO3-N/SF
3.75 bumps per day

900 =flow rate, gpm, per filter

Bump Cycle

5 drain down, minutes
2 pump duration, minutes
1 drain down, minutes

2.3 Head Loss in Media during backwash

eqn hl = (depth of bed) x [1 - avg porosity fraction] x [S.G. of media - S.G. of water]

6 =depth of bed
0.4 =avg media porosity
2.7 =specific gravity of the media

6.12	feet

3.0 Carbon Source Feed Source For Denitrification

3.1 Formula : Cm = 2.47 x (NO3-in) + 1.53 x (Nitrite in) + 0.87 Do

6	= NO3-N
0	= Nitrite
1	= dissoved oxygen in
15.69	=CM mg/L Carbon source as methanol

Check: if CM < 4 x NO3-in

2.62	= ratio, CM:NO3-n
result is	less than 4

3.2 For Glycerin Dosing

1188000	mg/L COD in methanol
685000	mg/L COD in 50% sugar solution
1.73	ratio Methanol COD/ Sugar Soln COD
27.2	Dosing, Carbon Source as Glycerin, mg/L
226.9	#/day required
50	percent commercial soln strength
54.4	gal/day required

4.0 Media Design

Anthracite	
2	ft, 'Depth Anthracite
3.65	size particles, mm
3/16 to 3/32	inches, effective size range
1.6	uniformity coefficient
2.7	hardness not less than, MOH scale
1.5	specific gravity
5%	acid solubility, shall be less than, tested per AWWA B100
Filter Sand	
4	ft, filter sand
2-3	mm. size particles
1.4	uniformity co-efficient
0.8	sphericity
6-7	hardness, MOH scale
2.6	specific gravity
5%	acid solubility, shall be less than, tested per AWWA B100

Gravel

18 inches graded gravel, depth

size	depth	
1/2x1/4	4	top
1/4x1/8	4	
1/2x1/4	4	

3/4x1/2	2	
1 1/2x3/4	4	bottom

EcoCycle SBR™ **Sequencing Batch Reactor (SBR) Design Outline**

Parkson
Treating Water Right

Grenelefe Resort - FL
Three Tank SBR
Rev. 2 - 3mg/l TN

Designer: AT
Date: 14-Aug

Flow (ADF)	0.50	MGD average	1,893 m^3/d
Flow (PDF)	1.50	MGD	5,678 m^3/d

INFLUENT CHARACTERISTICS

	mg/l	lbs/d	kg/d
BOD	203	847	384
* COD	406	1,693	768
TSS	248	1,034	469
TKN	40	167	76
NH4-N	27	111	50
* TN	40	167	76
P	5.8	24	11
* TDS	500	2,085	946

* Inert TSS fraction 40 %

EFFLUENT REQUIREMENTS

	mg/l	lbs/d	kg/d
BOD	10	42	18.9
COD	NR	NR	NR
TSS	10	42	18.9
TKN	NR	NR	NR
NH3-N Sum	1.0	4	1.9
NH3-N Win	1.0	4	1.9
TN	3.0	13	5.7
** P	NR	NR	NR

** Alum or ferric chloride addition req'd

SITE CONDITIONS

Winter WW Temperature (min.)	15 °C	59 °F
Summer WW Temperature (max)	27 °C	81 °F
Average WW Temperature	21 °C	70 °F
Elevation	150 ft	46 m
Average barometric pressure	14.61 psia*	101 kPa
Winter Air Temperature	0 °C	32 °F
Summer Air Temperature	38 °C	100 °F

PROCESS DESIGN PARAMETERS

Design MLSS	3,310 mg/l @ TWL		
Design MLSS	3,972 mg/l @ BWL		
Hydr. Retention Time provided	1.50 days	36.0 hours	
Aerobic Sludge Age (SRTox)	9.9 days		
System SRT	25.8 days		
Biosolids growth rate	0.22	gVSS/gCODr/d	
	0.45	gVSS/gBODr/d	
F:M (adjusted for aeration %)	0.21	gCOD/gMLSS/d	
	0.11	gBOD/gMLSS/d	
System F:M	0.04	gBOD/gMLSS/d	
Avg biosolids yield	372	lbs./day*	169 kg/d
Avg net sludge yield (bio+inerts)	743	lbs/d based on CODr*	337 kg/d
	802	lbs/d based on BODr*	364 kg/d
Mass aerobic MLSS req'd	7,969	lbs	3,614 kgs
Mass aerobic volume req'd	0.29	MG	1,093 m^3
Aerated portion of day	38.5	%	
Required total SBR volume	0.75	MG	2,838 m^3

BASIN DIMENSIONS

Number of SBR basins	3	
Rectangular Dimensions:		
Length/Width Ratio	1.0 : 1	
Length	42.5 ft.	12.95 m
Width	42.5 ft.	12.95 m
Round Dimensions		
Diameter	48 ft.	14.62 m
Top Water Level	18.5 ft.	5.64 m
Bottom Water Level	15.4 ft.	4.70 m
TWL at Design Average Flow	18.5 ft.	5.64 m
Total Volume in SBR's	0.75 MG	2,838 m^3
Total Retention Time in SBR	36.0 hrs.	

AERATION SYSTEM SIZING

First Estimate :

lbs. O2/lb. BOD removed	1.25	kg O2/kg BOD removed		
lbs. O2/lb. TKN oxidized	4.6	kg O2/kg TKN oxidized		
lbs. O2/lb. NO_3x denitrified	-2.86			
Denitrification Credit	50 %			
Actual Oxygen Req'd, AOR	1,460	lbs. O2/day	662	kg/d

Second Estimate :

AOR = CODi - CODw - CODes + 4.6*TKNox - 2.86*NO3Ndn

where :	CODi	influent	=	1,693 lbs./day	768 kg/d
	CODw	wasted	=	446 lbs./day	202 kg/d
	CODes	eff soluble	=	271 lbs./day	123 kg/d
	TKNox**	oxidized	=	135 lbs./day	61 kg/d
	NO_3Ndn	denitrified	=	61 lbs./day	28 kg/d
	Mass balance AOR			1,424 lbs./day	646 kg/d

Use highest estimate ***DESIGN AOR = 1,460 lbs/day*** ***662 kg/d***

Conversion Formula from ASCE Manual of Practice : *SOR =* $\frac{AOR * Cs}{a * (\beta Csd - DO) * Ø\text{^}(T-20)}$

Cs = DO saturation at Stnd Conditions
= 9.092*(1+0.4*D/34)
= 11.07 mg/l

Csd = DO saturation at design conditions
Cst = DO saturation@liquid Temp & 1 sea level
where : = Cst*(Fe+0.4*D/34)
= 7.95 mg/l

ElevFactor Fe = 0.99 Therefore, Csd = 9.62 mg/l

Alpha, a	0.85 *	SWD, D	18.5 ft
D.O., mg/l	2.0 mg/l	Beta, ß	0.95 *
WW Temp T	27 °C	Theta, Ø	1.024

Standard Oxygen Required, SOR	=	**2,257**	**lbs. O2/day**	**1,025 kg/d**
SOR Peaking Factor	=	**1**		
DESIGN SOR	=	**2,257**	**lbs. O2/day**	**1,025 kg/d**

CYCLE TIMES

Batches per day	4.00	per SBR	
Complete Cycle time	6.00	hrs. per basin	
Fill time at ADF	2.00	hrs.	
Anoxic Fill time	1.50	hrs.	75 % of FILL is anoxic.
Aerated Fill	0.50	hrs.	
React time	1.81	hrs.	39 % of cycle is aerated.
Denite time	0.50	hrs.	
Settle Time	1.00	hrs.	3.7 hrs. anoxic per cycle
Decant time	0.60	hrs.	
Idle time	0.09	hrs.	2.3 hrs. aerated per cycle

JET AERATION SYSTEM SIZING

Aerator elevation	2.5	ft.		0.76 m
Nozzle Angle	25	°		
Avg aerator submergence	15.9	ft.		4.85 m
Total aeration time	2.31	hrs./cycle		
	9.2	hrs./basin/day		
SOR	81	lbs./hr/basin		37 kg/hr
Normal gassing rate at ADF	44.9	SCFM / jet		1.27 m^3/min/jet
Max gassing rate	88.0	SCFM / jet		2.49 m^3/min/jet
Oxygen transfer efficiency (ADF)	21.9	%		
Design air flow	359	SCFM		10 m^3/m
Jets required per basin	8.0	Model 44 A Jets		
Add'l jets for mixing	0			
Total jets per basin	8.0			
Jet headers per basin	1	Type : **D**		D = Dual, S = Single
Jets per header	8	Model 44 A Jets		

BLOWER SIZING DETAILS

Operating blowers	=	1	per aerating basin		
Type of Blowers :	=	1	*1=PD, 2=Centrifugal, 3=Turbo*		
Total Number of Blowers	=	4	including a spare		
Air flow per blower	=	359	SCFM	610 m^3/hr	
Inlet losses	=	0.3	psig *	2.07 kPa	0.02 bar
Net inlet pressure	=	14.31	psia (absolute)	98.67 kPa	0.98 bar
Discharge piping losses	=	0.7	psig *	4.83 kPa	0.05 bar
Losses at aerator	=	0.1	psig	0.69 kPa	0.01 bar
Total discharge pressure	=	7.99	psig average	55.09 kPa	0.55 bar
		8.03	psig maximum	55.34 kPa	0.55 bar
		6.69	psig minimum	46.14 kPa	0.46 bar
Site air flow required	=	391	ICFM average	11.08 m^3/min	
Assumed blower efficiency	=	60	% *		
BHp per blower	=	18.6	BHp/Blower	13.9 BkW	
				14.7 kW @ 94% ME	
Blower BHp/aerating basin	=	18.6	BHp/Basin	13.9 BkW	
				14.7 kW @ 94% ME	

JET MOTIVE PUMPS			
Number of pumps	1	per basin	
Type of Pumps :	1	*1=Dry pit, 2=Submersible, 3=Axial flow*	
Total number of pumps	3		
Design pressure at nozzle	17	ft.	5.2 m
Flow per nozzle	183	GPM	11.5 l/s
Flow per pump	1,465	GPM	92.4 l/s
System headloss	4	ft.*	1.2 m
Total pump head	21	ft.	6.4 m
Assumed pump efficiency	75	% *	
BHp per pump	10.4	BHp/Pump	7.7 BkW 8.2 kW @ 94% ME
Total pump BHp/basin	10.4	BHp/Basin	7.7 BkW 8.2 kW @ 94% ME

DECANTERS		
Cycles per day	12	
Avg TWL to BWL volume	41,667 Gallons	158 cubic meters
Max TWL to BWL volume	41,667 Gallons	158 cubic meters
Decant time	0.60 hrs.	36 minutes
Average decant flow	1,157 GPM	73 liters per second
Number of decanters per basin	1	
Average flow per decanter	1,157 GPM	73 liters per second

SLUDGE WASTING		
Dry solids (BOD estimate)	802 lbs/day	364 kg/d
Solids concentration in WAS	0.85 %	
Total volume wasted per day	11,312 gallons per day	43 m3 / day
Wasting frequency	4 per tank per day	
Volume wasted each period	943 gallons	4 m3
Length of each wasting period	9.4 minutes	
WAS pump rate	100 gpm	6 liters per second
WAS pump discharge head	15 ft	4.6 meters
WAS pump efficiency	40 %	
WAS pump BHp	0.9 BHp	0.7 kW

POWER SUMMARY

Equipment	BHp/basin	Hours/day operating		kW hr/day	kW hr/annual
SBR blowers	18.6	27.72		384	140,174
SBR jet pumps	10.4	33.72		260	95,078
Cost of power per kWhr	0.05		Total	645	235,252
****Annual power cost**	**$11,763**				

** does not include corrections for motor efficiency, VFD losses, V-belt losses, or power factor

***Denotes parameters assumed by Parkson. These parameters to be confirmed by Owner or Owner's representative**

Denitrification Kinetics Calculation

Design Calculations to Determine Required Time to Denitrify Wastewater

Design Parameters			
MLSS	3,310	mg/l	Mixed Liquor Suspended Solids
	0.65		volatile fraction of mixed liquor solids
MLVSS	2,152	mg/l	Mixed Liquor Volatile Suspended Solids
D.O.	0.1	mg/l	Dissolved Oxygen Concentration in the Anoxic Zone
T	15	° C	Basin Liquid Temperature
$(NO_3)_o$	32	mg/l	Design Influent Nitrate Nitrogen concentration
$(NO_3)_e$	3	mg/l	Design Effluent Nitrate Nitrogen concentration

1 Determine Rate of Denitrification Corrected for Temperature

$$R_{DN(T)} = R_{DN(20)} * K^{(T-20)} * (1\text{-D.O.})$$

where:

$R_{DN(T)}$ = Rate of denitrification at the design temperature

$R_{DN(20)}$ = 0.1 = Rate of denitrification at 20° C

K = 1.09 = 1.03 to 1.1 (1.09 commonly used)

$R_{DN(T)}$ = 0.058 g NO_3-N/g VSS-day

2 Determine the Time Required for Denitrification

$$t = [(NO_3)_o - (NO_3)_e]/[R_{DN}*X_V]$$

where:

$(NO_3)_o$ = Influent Nitrate Nitrogen, (mg/l)

$(NO_3)_e$ = Effluent Nitrate Nitrogen, (mg/l)

X_V = MLVSS concentration, (mg/l)

t = Anoxic Time, (days)

t = 0.233 days

= 5.6 hours required

= 24.0 hours provided