# RFP-05-02
# Design-Build Services
# for
# Three Oaks WWTP Expansion
# for
# Lee County

## PRELIMINARY ENGINEERING REPORT

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TABLE OF CONTENTS

SECTION 1 INTRODUCTION ..... 1
*1.1 Background* ..... *1*
*1.2 Request For Proposal* ..... *1*
SECTION 2 BASIS OF DESIGN ..... 3
*2.1 Existing Facility* ..... *3*
*2.2 Raw Wastewater Quality and Flows* ..... *4*
*2.3 Effluent Quality and Disposal* ..... *4*
*2.4 Reliability Design Standards* ..... *5*
*2.5 Existing Plant Analysis* ..... *5*
SECTION 3 EXPANSION DESIGN CRITERIA ..... 8
*3.1 Expansion Flow Criteria* ..... *8*
*3.2 Expansion Loading Criteria* ..... *8*
*3.3 Effluent Water Quality Standards* ..... *9*
*3.4 Effluent Disinfection Requirements* ..... *9*
*3.5 Biosolids Requirements* ..... *9*
*3.6 Provisions For Reliability* ..... *11*
SECTION 4 WWTF EXPANSION/UPGRADE ..... 12
*4.1 Introduction* ..... *12*
*4.2 Preliminary Treatment* ..... *12*
*4.3 Biological Treatment Process* ..... *14*
*4.4 Secondary Clarifiers* ..... *15*
*4.5 Filters* ..... *19*
*4.6 Disinfection* ..... *21*
*4.7 Transfer Pumps* ..... *21*
*4.8 Effluent Pumps* ..... *22*
*4.9 Chemical Feed Systems* ..... *23*
*4.10 Sludge Holding & Dewatering* ..... *24*
*4.11 Plant Drain Pump Station* ..... *26*
*4.12 Site Work and Stormwater Management* ..... *27*
*4.13 Electrical Upgrades* ..... *28*
*4.14 Environmental Effects* ..... *31*
APPENDIX A ..... 33

# LIST OF TABLES

*Table 2-1* *Summary of Design Flows* .......... 4
*Table 2-2* *Summary of Effluent Quality* .......... 4
*Table 3-1* *Three Oaks WWTP – Design Flows* .......... 8
*Table 3-2* *Three Oaks WWTP Influent Design Criteria* .......... 8
*Table 3-3* *Summary of Effluent Quality* .......... 9
*Table 3-4* *Pollutant Concentration Limits For Land Application of Wastewater Residuals* .......... 10
*Table 4-1* *Summary of Design Criteria-Preliminary Treatment* .......... 13
*Table 4-2* *Summary of Design Criteria-Odor Control* .......... 14
*Table 4-3* *Summary of Design Criteria-Biological Treatment Process* .......... 15
*Table 4-4* *Summary of Design Criteria-Secondary Clarifiers* .......... 17
*Table 4-5* *Summary of Design Criteria-Filters* .......... 20
*Table 4-6* *Summary of Design Criteria-Transfer Pump Station* .......... 21
*Table 4-7* *Summary of Design Criteria-Effluent Pump Station* .......... 22
*Table 4-8* *Summary of Design Criteria Sodium Hypochlorite Feed System* .......... 23
*Table 4-9* *Summary of Design Criteria-Sludge Dewatering* .......... 25
*Table 4-10* *Summary of Design Criteria-Plant Drain Pump Station* .......... 27
*Table 4-11* *Duty Load Summary* .......... 29

# SECTION 1 INTRODUCTION

## 1.1 Background

The Three Oaks Wastewater Treatment Plant (WWTP) is owned and operated by Lee County Utilities (LCU) under Permit Number FL0145190 issued by the Florida Department of Environmental Protection (FDEP). The WWTP receives raw wastewater from the southern portions of Lee County. The wastewater is dominantly residential and commercial in nature. The Three Oaks WWTP service area and land use maps are provided in Appendix A. Incoming wastewater is combined with brine water from the Pinewoods Reverse Osmosis (RO) Water Treatment Plant prior to entering the treatment facility. The existing facility is currently permitted for a capacity of 2.404 million gallons per day (mgd) average annual daily flow (AADF). However, the facility has an existing treatment capacity of 3.0 mgd AADF. The WWTP design capacity will be expanded from 3.0 mgd to 6.0 mgd with the ability to expand to 9.0 mgd in the future. Expansion beyond 6.0 mgd AADF is beyond the scope of this report. However, general information will be provided for the planned future expansion to 9.0 mgd AADF in sufficient detail to demonstrate that the 6.0 mgd AADF facility incorporates features to facilitate this expansion.

This report will include the following:

- Existing plant capacity analysis including an existing plant analysis and historical flow analysis,
- Design criteria for expansion of the plant to 6.0 mgd,
- Description of proposed treatment facilities for expansion to 9.0 mgd.

## 1.2 Plant Improvements

The improvements include expanding and upgrading the facility to a design capacity of 6.0 mgd, with the ability to expand to 9.0 mgd in the future. The improvements include:

- ✓ Replace the existing headworks with a larger capacity structure (considering future capacity of 9.0 mgd). Include automated screening, grit removal and odor control facilities.
- ✓ Modifications to the existing oxidation ditches to increase treatment capacity.
- ✓ Increase clarifier capacity (to approximately 6.0 mgd with largest clarifier out of service) and increase RAS/WAS pumping capacity.

- ✓ New filter feed pump station.
- ✓ New deep bed filters.
- ✓ New chlorine contact basin.
- ✓ New bulk sodium hypochlorite storage and feed system.
- ✓ New effluent transfer pump station.
- ✓ New reuse/deep injection well pumping station.
- ✓ New deep injection well surface facilities.
- ✓ New sludge dewatering facilities.
- ✓ Yard piping modifications and expansion as needed.
- ✓ Electrical and instrumentation / controls improvements.
- ✓ Additional emergency power with diesel generator and fuel storage tank.
- ✓ Site civil improvements, including grading, stormwater drainage, and paving.

# SECTION 2 BASIS OF DESIGN

## 2.1 Existing Facility

The Three Oaks WWTP has an existing treatment capacity of 3.0 mgd and is proposed to be expanded utilizing the Design-Build project delivery method. The existing plant uses an oxidation ditch treatment process and includes the following major components:

- Headworks with screening and grit removal
- Surge tanks (3)
- Screw pump transfer station
- Oxidation ditches (One with a capacity of 1.5 mgd and two with a capacity of 0.75mgd, each)
- Clarifiers (One 64 ft. diameter and three 40 ft. diameter)
- Rectangular aerobic digesters (3) and lime stabilization tanks (2)
- Traveling bridge filters (Three with 0.5 mgd capacity, each)
- Hydro-Tech disc filters (Three with 1.0 mgd capacity, each)
- Effluent diversion chamber (to divert reject water to the reject storage tank)
- Chlorine contact chambers (3)
- Reuse pumping station
- Reject storage tanks (2)
- Reuse storage tanks (2)

Effluent disposal is currently accomplished through the use of discharge to a reclaimed water system. Wastewater residuals are stabilized and disposed of off-site. The existing site plan is provided in Appendix A.

## 2.2 Raw Wastewater Quality and Flows

Expansion of the Three Oaks WWTP is based on the following design flow rates summarized below:

**Table 2-1 Summary of Design Flows**

| Parameter | Initial Expansion | Future |
|---|---|---|
| Average Annual Flow, mgd | 6.0 | 9.0 |
| Maximum Month Flow (MMF), mgd | 7.5 | 11.25 |
| Maximum Daily Flow (MDF), mgd | 9.0 | 13.0 |
| Peak Hourly Flow (PHF), mgd | 18.0 | 22.5 |

## 2.3 Effluent Quality and Disposal

Currently, effluent is discharged to the County's reclaimed water system, suitable for public access reuse. Additionally, the County is currently installing a deep well at the WWTP to provide backup disposal capacity.

The following table summarizes the permitted effluent quality requirements.

**Table 2-2 Summary of Effluent Quality**

| Parameter | Deep Well Disposal | Reclaimed Water System |
|---|---|---|
| $CBOD_5$, mg/L | 20 | 20 |
| TSS, mg/L | 20 | 5 |
| Fecal Coliform, #/100mL | -- | 0 (with allowable excursions) |
| Chlorine Residual, mg/L | -- | 1 |

The existing reject storage and reclaimed water storage tanks will continue to be used as part of the plant expansion. Construction of the new injection well will allow more effective management of wet weather conditions and reject events.

## 2.4 Reliability Design Standards

In accordance with FDEP's reuse standards (62-610 FAC), facilities that are permitted for public access reuse must have Class 1 reliability. The proposed facilities will be designed to be in compliance with the Class 1 reliability standards as defined in EPA's manual of practice MCD-05: Design Criteria for Mechanical, Electric, and Fluid System and Component Reliability.

## 2.5 Existing Plant Analysis

The existing plant consists of an extended aeration, activated sludge plant designed to provide advanced secondary treatment, basic disinfection and aerobic digestion. The existing plant is operating at approximately 2.0 mgd annual average daily flow with a permitted capacity of 2.404 mgd AADF and a treatment capacity of 3.0 mgd AADF.

The plant is nominally designed for 3.0 mgd AADF in the extended aeration mode. Unit processes that potentially limit its treatment capacity include:

- Screening & Grit Removal
- Process Aeration
- Secondary Clarification
- Filtration
- Disinfection
- Effluent Pumping
- Aerobic Digestion

The existing Headworks comprise screening and grit removal. The screening unit is a ¼-in Vulcan packaged automatic step screen. A ½-in manual screen is provided in the event that the mechanical screen is taken offline for maintenance or repairs. The screenings fall by gravity into a stainless steel handcart, manually transferred to a dump receptacle, and disposed of at a solid waste facility. The package system is designed for a peak flow of 9.0 mgd. Grit is removed from influent wastewater using a single grit separator with a peak capacity of 7.0 mgd. Grit is a term that refers to the inorganic solids in wastewater such as sand. Generally, the objective of grit removal is to reduce the wear on pumping equipment due to the abrasiveness of grit. If grit is not treated, it tends to accumulate in process tanks because it is difficult to keep the material in suspension.

There are three existing oxidation ditches at the WWTP, two equally sized tanks with a volume of 0.75 million gallons (mg) each and a third tank with a volume of 1.5 mg. These tanks were originally sized to provide 24-hours of hydraulic detention time at 3.0 mgd AADF. Each of the 0.75 mg oxidation ditches is equipped with three (3) 33-Hp Lakeside brush-type aerators. The 1.5 mg oxidation ditch is equipped with three (3) 50-Hp Lakeside brush-type surface aerators. The brush-type aerators provide oxygen and mixing for 5-day carbonaceous biological oxygen demand ($CBOD_5$) removal and oxidation of ammonia-nitrogen to nitrate-nitrogen. Therefore, the plant capacity is limited by the current aeration capacity. The oxidation ditch volume will be adequate for flows up to 9.0 mgd AADF.

There are four (4) secondary clarifiers at the Three Oaks WWTP, three (3) 40-ft diameter clarifiers and one (1) 64-ft diameter clarifier. The primary purpose of the secondary clarifiers is to allow time for solids to settle to the bottom and scum to rise to the top. Settled sludge is removed from the bottom of the clarifiers and is pumped to the aerobic digester. Floating scum is collected by a full radius surface skimmer and scum trough, which is collected and also pumped to the digester. Clarified wastewater overflows for further treatment. The existing secondary clarifiers currently have a combined surface area of 6,987 sf. At 9.0 mgd PHF, the average peak hydraulic surface-loading rate for the clarifiers is 1,288 gpd/sf. Peak solids loading rates are expected to be 50 lb/d/sf. The design capacity of the existing clarifiers is 3.0 mgd AADF and 9.0 mgd PHF.

The Three Oaks WWTP utilizes two types of filter technologies. The first is an Aqua-Aerobics traveling bridge type filter. These filters currently treat reject effluent that is stored in on-site storage tanks. There are a total of three (3) traveling bridge type filters at the facility each with a 0.75 mgd capacity. The second type of filters is a Hydro-Tech disk filter. There are three disk filters each with a 2.0 mgd PHF capacity. For the current plant PHF flow factor of 2.0, each disk filter has a 1.0 mgd AADF capacity. The objective of filtration is to reduce the suspended solids concentration in the secondary effluent from 20 mg/L to a maximum concentration of 5 mg/L prior to disinfection. The disk filters provide a combined 2.0 mgd AADF of filtration capacity and are currently treating all of the facility's effluent.

There are three (3) chlorine contact chambers at the WWTP. Two of the contact chambers have a volume of 14,000 gallons each and the third has a volume of 29,700 gallons. FDEP regulation 62-600.440 prescribes values for CT (C, residual disinfectant concentration (mg/L), times T, contact time (minutes)) for specific treatment and disinfection conditions to achieve desired levels of pathogen inactivation under various conditions. At the current design flow of 3.0 mgd AADF and current peak flow factor of 2 times the AADF with equalization achieved in the surge tanks, the existing chlorine contact tanks will provide 27.7 minutes of contact time at AADF and 13.9 minutes at PHF. FDEP regulations require a minimum 15 minutes of contact time at PHF while maintaining a chlorine residual of 1.0 mg/L. Thus, the existing chlorine contact capacity is only about 2.8 mgd AADF and 5.5 mgd PHF.

Another major plant constraint is effluent pumping. The existing effluent pump station consists of two (2) chambers, each with a volume of 10,770 gallons. The first chamber has three (3) 50-Hp Peerless vertical turbine pumps with a capacity of 800 gpm each. These pumps deliver effluent to the reclaimed water distribution system. With all three pumps online the facility can provide a maximum 3.45 mgd of reclaimed water to its service area. The other chamber has two (2) 30-Hp Peerless vertical turbine pumps, each with a capacity of 425-gpm, which transfer effluent to the on-site storage tanks.

The Three Oaks WWTP has three (3) 0.217 MG square aerobic digesters. The sludge is aerated and mixed using diffused air provided by four (4) 900 scfm Hoffman blowers. Based on a max-month influent CBOD load of 10,200 lbs/day and a sludge yield of 0.80 lbs MLSS / lb BOD applied, the existing digester provides approximately 7.6 days of hydraulic detention time at 1% solids.

Several key components of the existing plant will be retained for use as part of the expanded facility. These include:

- Oxidation ditches including aerators.
- Secondary clarifiers
- Reject and reclaimed water storage tanks
- Digesters
- Existing electrical system including generators

# SECTION 3 EXPANSION DESIGN CRITERIA

## 3.1 Expansion Flow Criteria

The expansion is based on a flow of 6.0 mgd, with an ultimate plant capacity of 9.0 mgd. Table 3-1 provides a summary of the criteria used to develop design flows for the WWTP expansion.

**Table 3-1 Three Oaks WWTP – Design Flows**

| | Design Flow | |
|---|---|---|
| | **Initial Expansion** | **Future** |
| Average Annual Flow, gpd | 6,000,000 | 9,000,000 |
| Maximum Month Flow, gpd | 7,500,000 | 11,250,000 |
| Peak Hourly Flow, gpm | 12,500 | 15,625 |

| | | |
|---|---|---|
| Peaking Factors: | | |
| MMF/ADF | 1.25 | 1.25 |
| MDF/ADF | 1.44 | |
| PHF/ADF | 3.0 | 2.5 |

## 3.2 Expansion Loading Criteria

Using the criteria presented in Table 3-1, the wastewater design loads shown in Table 3-2 were developed using historical data for use in design calculations for plant sizing.

**Table 3-2 Three Oaks WWTP Influent Design Criteria**

| Parameter | Design Loading, lb/d @ MMF | Design Loading, mg/L | |
|---|---|---|---|
| | | **@ MMF** | **@ AADF** |
| TSS | 12,800 | 205 | 205 |
| CBOD5 | 10,200 | 163 | 163 |
| TN | 1,250 | 20 | 20 |
| Min. Water Temp. Deg. C | | 20 | 20 |

## 3.3 Effluent Water Quality Standards

The existing WWTP discharges into the County's reclaimed water system, but a deep well is being constructed for backup disposal.

To account for these anticipated methods of effluent disposal, it has been assumed that the plant will be required to provide effluent meeting the limits shown in Table 3-3.

**Table 3-3 Summary of Effluent Quality**

| Parameter | Concentration mg/L |
|---|---|
| $CBOD_5$ | 20 |
| TSS | 5 |
| TN | 10 |
| Chlorine Residual | 1 |
| Fecal Coliform, #/100mL | 0 (with allowable excursions) |

## 3.4 Effluent Disinfection Requirements

The Three Oaks WWTP will utilize sodium hypochlorite for disinfection. The disinfection system will be designed to meet high-level disinfection requirements in accordance with Chapter 62-600 of the Florida Administrative Code (F.A.C.). A "CT" of at least 25 will be required for high-level disinfection at PHF. The minimum contact time at peak hourly flow will be 15 minutes. Based on current permit limits, a minimum chlorine residual of 1.0 mg/L must be maintained following disinfection.

## 3.5 Biosolids Requirements

Both the Florida Department of Environmental Protection (FDEP) and the United States Environmental Protection Agency (USEPA) regulate domestic wastewater residuals. The FDEP has not been delegated authority by the USEPA over domestic wastewater residuals. Therefore, the regulatory issues discussed will apply to both sets of regulations. The applicable FDEP regulations are contained in Chapter 62-640, F.A.C., while the USEPA regulations are contained in Title 40 of the Code of Federal Regulations (CFR) Part 503. The regulations establish monitoring, record keeping, reporting requirements, and site use restrictions for the land application of residuals. In addition, all land-applied residuals must meet the following concentration and stabilization requirements:

- Pollutant Concentration Limits
- Pathogen Reduction
- Vector Attraction Reduction

### 3.5.1 Pollutant Concentration Limits

Regulations establish ceiling concentration limits for 9 different pollutants. Residuals may be land applied only if the concentrations of all these parameters do not exceed the established ceiling concentrations. In addition to the ceiling concentration limits, cumulative pollutant loading rates are also regulated for 8 of the 9 pollutants. However, if the average monthly pollutant concentrations meet "Exceptional Quality" or "High Quality" pollutant concentration limits established for these same 8 heavy metals as defined in Chapter 62-640.850(3)(a), F.A.C., and 40 CFR Part 503.13, then the cumulative pollutant loading rates do not apply.

The following table summarizes the various pollutant concentration limits discussed above.

**Table 3-4 Pollutant Concentration Limits For Land Application of Wastewater Residuals**

| Pollutant | Ceiling Concentration Limits (mg/kg) (1) | Cumulative Pollutant Loading Rates (lbs/ac) | "High Quality" Pollutant Concentration Limits (mg/kg) (2) |
|---|---|---|---|
| | | | |
| Arsenic | 75 | 36.6 | 41 |
| Cadmium | 85 | 34.8 | 39 |
| Copper | 4,300 | 1,340 | 1,500 |
| Lead | 840 | 268 | 300 |
| Mercury | 57 | 15.2 | 17 |
| Molybdenum | 75 | -- | -- |
| Nickel | 420 | 375 | 420 |
| Selenium | 100 | 89.3 | 100 |
| Zinc | 7,500 | 2,500 | 2,800 |
| | | | |

Notes: (1) Dry weight basis
(2) Monthly averages

### 3.5.2 Pathogen Reduction

Domestic wastewater residuals must also meet pathogen reduction requirements in order to be land applied. The regulations define two types of residuals that may be land applied with respect to pathogens: Class A or Class B. Residuals that do not meet the requirements of either Class A or Class B may not be land applied. Chapter 62-640.600 F.A.C. and 40 CFR Part 503.32 define a number of alternatives that may be used to meet either Class A or Class B requirements.

### 3.5.3 Vector Attraction Reduction

Chapter 62-640, F.A.C., defines Vector Attraction as "the characteristics of residuals that attracts rodents, flies, mosquitoes, or other organisms capable of transporting infectious agents". All land-applied residuals must meet one of ten defined vector attraction reduction alternatives described in Chapter 62-640.600 F.A.C. and 40 CFR Part 503.33.

## 3.6 Provisions For Reliability

In accordance with FDEP's reuse standards (62-610 FAC), facilities that are permitted for public access reuse must have Class 1 reliability. Therefore, the proposed facilities shall be designed to be in compliance with the Class 1 reliability standards as defined in EPA's manual of practice MCD-05: Design Criteria for Mechanical, Electric, and Fluid System and Component Reliability.

# SECTION 4 WWTF EXPANSION/UPGRADE

## 4.1 Introduction

This section summarizes the expansion and upgrade to the Three Oaks WWTP proposed by the Wharton-Smith/Boyle Design-Build Team. A site plan, hydraulic profile and process flow schematic/mass balance are included in Appendix A.

## 4.2 Preliminary Treatment

The current headworks will be replaced with a new cast-in-place concrete preliminary treatment structure. The structure will be designed and constructed to process 9.0 mgd AADF and 22.5 mgd PHF. Initially, sufficient screening capacity will be installed to meet a design flow of 6.0 mgd AADF and 18.0 mgd PHF. Grit removal equipment will be installed to meet future design flows of 9.0 mgd AADF and 22.5 mgd PHF. All influent wastewater will flow by force main through an in-line magnetic flow meter to the preliminary treatment structure. Samples for regulatory compliance will be collected at the headworks. Raw wastewater will be sampled at the head of the preliminary treatment structure upstream of the screening units, upstream of the in-plant wastewater returns and prior to any recycle flows. A refrigerated composite sampler will be provided at the screening structure.

Preliminary treatment will consist of mechanical screening and grit removal. The structure will include a packaged screening system including one (1) automatic bar screen, with a capacity of 18.0 mgd PHF, and two (2) manual bar screens for bypassing. An identical automatic screen will need to be added in the future (replacing one of the manual bar screens) for the 9.0 mgd plant. The mechanically cleaned screen will be used to filter out trash and other large objects found in the wastewater influent. The screen will clean itself automatically and deposit the screenings into chute/conveyor that will transport the screenings to a washer/compactor prior to dropping the screenings into a dumpster located at grade next to the preliminary treatment structure. If the automatic screen malfunctions or when the water level reaches the over flow level, the wastewater will be routed through emergency bypass channels containing manual bar screens prior to grit removal.

Grit will be removed from the influent wastewater using one (1) grit separator (Headcell). The grit treatment tank will be constructed and equipped for future design flows of 9.0 mgd AADF and 22.5 mgd PHF. Grit will be removed from the bottom of the Headcell using grit pumps and delivered to a grit classification unit. This unit separates grit from the Headcell's underflow. Grit underflow from the classifier will flow by gravity to a dewatering unit. The clarified water will drain to a wet well and pumped back to the preliminary treatment structure upstream of the screens and prior to the grit separator. Dewatered grit will be transported to a dumpster and trucked to a landfill.

At the effluent end of the headworks structure will be a splitter box with proportional/adjustable weir gates that will control the flow split to the biological treatment trains. Raw wastewater will be conveyed by gravity via 30 and 36-inch pipes to each of the three aerations basins.

Table 4-1 summarizes the design criteria for the mechanical screens and grit removal unit.

**Table 4-1 Summary of Design Criteria-Preliminary Treatment**

| Parameter | Initial Facilities for 6.0 mgd AADF | Future Facilities for 9.0 mgd AADF |
|---|---|---|
| **Mechanical Screens** | | |
| No. of new units | -- | 1 |
| Total no. of units | 1 | 2 |
| Capacity @ PHF (each), mgd | 18 | 18 |
| Total capacity @ PHF, mgd | 18 | 36 |
| Size opening, mm | 6 | 6 |
| Hp, each | 2 | 2 |
| Manufacturer | Vulcan | -- |
| **Manual Screens** | | |
| No. of new units | 2 | 1 |
| Total no. of units | 2 | 1 |
| Capacity @ PHF (each), mgd | 9 | 22.5 |
| **Screenings Compactor** | | |
| No. new units | 1 | -- |
| Total no. of units | 1 | 1 |
| Hp, each | 5 | -- |
| **Grit Removal** | | |
| No. of headcells | 1 | --- |
| No. of new trays | 11 | -- |
| Total no. of trays | 11 | 11 |
| Total Capacity @ PHF, mgd | 22.5 | 22.5 |
| Manufacturer | Eutek | -- |
| **Classification Unit (Slurry Cup)** | | |
| No. of new units | 1 | -- |
| Capacity (each), gpm | 300 | -- |
| Manufacturer | Eutek | -- |
| **Grit Dewatering Units (Grit Snail)** | | |
| No. of new units | 1 | -- |
| Capacity (each), cu yd/hr | 2.0 | -- |
| Hp, each | 1/3 | -- |
| Manufacturer | Eutek | -- |
| **Grit Pumps** | | |
| No. of new units | 2 | -- |
| Capacity (each), gpm | 300 | -- |
| Hp, each | 20 | -- |
| Manufacturer | Wemco | -- |

Open channels and tanks will be covered on the preliminary treatment structure to contain odor. The odor control system selected for this project is a biofiltration system. Odorous air will be withdrawn from the structure at a rate of 600 scfm.

Table 4-2 summarizes the design criteria for the odor control system.

**Table 4-2 Summary of Design Criteria-Odor Control**

| Parameter | Criteria |
|---|---|
| No. of units | 1 |
| Capacity, scfm | 600 |
| Average $H_2S$ Concentration, ppm | 75 |
| Peak $H_2S$ Concentration, ppm | 150 |
| Manufacturer | USFilter |

## 4.3 Biological Treatment Process

The existing oxidation ditches and brush aerators will be used for the initial expansion of the Three Oaks WWTP. The existing brush aerators appear to be sufficient for the max month loadings that are predicted for the facility. However, one new brush aerator will be installed in the 1.5 MG oxidation ditch to improve process reliability and provide additional air.

Nitrogen control will be achieved in the ditch by promoting simultaneous nitrification and denitrification. This will be accomplished by cycling the brush aerators to create anoxic zones within the ditch. The plant SCADA system will automatically control the aerators based on a software timer and/or dissolved oxygen that will be monitored at two locations within each ditch.

Effluent weir modifications will be needed for each ditch so that the increased design flow will not adversely affect the submergence of the aerators. A new effluent box will be constructed on the north end of the large ditch. The mixed liquor from the large ditch will flow by gravity to the new clarifiers.

A new effluent weir splitter box will be constructed for the smaller ditches on the south end of their structures. The mixed liquor from the smaller ditches will flow by gravity to the existing clarifiers. The existing yard piping between the oxidation ditches and the existing clarifiers will be replaced with larger pipe to accommodate the design flow of 9.0 mgd PHF (refer to the proposed site plan).

Table 4-3 summarizes the design criteria for the biological treatment process.

**Table 4-3 Summary of Design Criteria-Biological Treatment Process**

| Parameter | Initial Facilities for 6.0 mgd AADF |
|---|---|
| **0.75 MG Oxidation Ditches** | |
| No. of Tanks | 2 |
| No. Aerators per Tank @ Hp | 3@30 |
| Manufacturer | Lakeside |
| **1.5 MG Oxidation Ditch** | |
| No. of Tanks | 1 |
| No. Aerators per Tank @ Hp | 3@50 |
| Manufacturer | Lakeside |
| No. New Aerators per Tank @ Hp | 1@50 |
| New Aerator Manufacturer | Lakeside |
| Hydraulic Detention Time, hr | 12 |
| MLSS, mg/L | 3,500 |
| Solids Retention Time, days | 12.2 |
| Actual Oxygen Required, lbs/day | 17,000 |
| Standard Oxygen Required, lbs/day | 23,000 |
| Waste Activated Sludge, lbs/day | 5,725 |

## 4.4 Secondary Clarifiers

Clarifier splitter boxes will split the aeration basins effluent flows between the secondary clarifiers. The flow will be designed to direct approximately 3.0 mgd AADF to the existing clarifiers and 3.0 mgd AADF to the proposed clarifiers. This will be accomplished using adjustable weir gates that are sized proportionally based on flow. At AADF and with all units in service, the average secondary effluent concentrations for both $CBOD_5$ and TSS are 20 mg/L.

Mixed liquor from the two (2) 0.75 MG oxidation ditches will flow by gravity to an adjacent splitter box. Weir gates in the splitter box will divide the flow between the existing secondary clarifiers. The existing facilities include four (4) secondary clarification units: three (3) 40-ft diameter clarifiers and one (1) 64-ft diameter clarifier. Flows to the existing clarifiers will be divided proportionally based on the solids loading rate to each clarifier. The splitter box weirs are sized to divide the mixed liquor flow from the 0.75 mg oxidation ditches approximately 55:45 between the 40-ft diameter clarifiers and the

64-ft clarifier respectively. These clarifiers provide a secondary clarification capacity of 3.0 mgd AADF and 9.0 mgd PHF with all units online. The yard piping associated with the existing clarifiers will need to be replaced to accommodate the unequalized PHF.

Approximately 1.6 mgd AADF / 4.8 mgd PHF will be diverted to a new splitter box located between the existing 40-ft clarifiers. Mixed liquor is further divided between the 40-ft diameter clarifiers at this second splitter box. The existing clarifier splitter box located at the 40-ft diameter clarifiers will be replaced with the new splitter box to accommodate the design flow rates. Each train will have proportionally sized effluent weirs. This will allow flow to be diverted to the new clarifiers if one of the clarifiers is taken out of service for maintenance or repairs. The remaining 1.4 mgd AADF / 4.2 mgd PHF will be directed to the existing 64-ft diameter clarifier. Flow from the 64-ft diameter clarifier will be diverted to the new clarifiers in the event that it needs to be taken out of service for maintenance or repairs.

For the expansion to 6.0 mgd AADF, additional clarifiers will be constructed to provide an additional 3.0 mgd AADF and 9.0 mgd PHF of capacity. Effluent from the 1.5 MG oxidation ditch will be directed to a new splitter box and new secondary clarifiers. Two (2) 85-foot diameter, 14-foot sidewater depth, secondary clarifiers will be provided to offer sufficient capacity to treat 75% of the PHF through the activated sludge train with one unit out of service in accordance with Class I Reliability requirements. In the event that one of the existing clarifiers needs to be taken offline, provisions will be made to divert mixed liquor to the new clarifiers. The pipe proposed for this purpose will ultimately be used to convey mixed liquor from the two small oxidation ditches to new clarifiers when the plant is expanded to 9.0 mgd AADF.

Currently at the 40-ft diameter clarifiers, sludge flow is withdrawn from each clarifier by gravity into the pump station suction piping. Telescoping valves controls sludge flow. These pumps discharge sludge constantly to the two smaller oxidation ditches but have the ability to discharge into the screw pump station via yard piping valves. At the existing 64-ft diameter clarifier the sludge is returned to the existing screw pump station by gravity using a telescoping valve for controlling flow. Construction is in process for the return sludge from the 64-ft clarifier to be pumped directly to the 1.5 mg oxidation ditch. The waste sludge from this clarifier is diverted by a 4-inch diameter pipe that is connected to the suction piping of the RAS/WAS pumps at the 40-ft diameter clarifiers. Wasting of sludge is done through the pump station at the 40-ft diameter clarifiers. No separate scum pumps are provided for any of the existing clarifiers. The screw pumps and the digester level are used to calculate the RAS and WAS flow rates.

During the expansion the existing RAS/WAS pumps located at the 40-foot clarifiers will be used for all of the existing clarifiers. The RAS discharge piping from the 64-ft diameter clarifier to the screw pump station will be plugged and a new 12" pipe will be provided to tie into the suction piping of the existing RAS/WAS pump station. The existing 4" WAS piping to the RAS/WAS pump suction will be removed. The existing RAS/WAS pumps are adequate to provide a minimum return rate of 100 percent of the AADF in addition to the WAS flow from the existing clarifiers. Scum will also be removed using the existing RAS/WAS pumps. The existing telescoping valves will be used to control sludge flow from each clarifier. A new RAS flow meter will be provided to determine RAS flow back to the new preliminary structure. A new motorized valve will be provided to divert sludge flow into the WAS pipe.

A level indicator in the digester will be used to measure the WAS flow rate by measuring the rate of increase in the tank volume to allow for flow adjustment in the new motorized valve.

A new return activated sludge (RAS) and waste activated sludge (WAS) pump station will be constructed adjacent to the new clarifiers. The RAS pumps will be designed to provide a minimum return rate of 100 percent of the AADF. The WAS pumps are sized to allow sludge wasting to occur over a four hour period each day. Wasting sludge could be scheduled as a single batch event or it could be scheduled once every hour for 15 minutes. The RAS and WAS flow rates will be continuously monitored and all new pumps will be driven by variable frequency drives (VFDs).

Each clarifier scum box at the new clarifiers will be connected directly to double disk style pumps. This type of pump will not be damaged if it runs dry. This approach has been used successfully at several plants around the state and it eliminates the maintenance problems associated with scum wet wells.

During future expansion of the WWTP an additional set of 85-foot diameter secondary clarifiers will be constructed. The new clarifier splitter box will be designed and constructed for the addition of two (2) future secondary clarifiers.

Table 4-4 summarizes design criteria for secondary clarification.

**Table 4-4 Summary of Design Criteria-Secondary Clarifiers**

| Parameter | Initial Facilities for 6.0 mgd AADF | | Future Facilities for 9.0 mgd AADF |
|---|---|---|---|
| **Existing Clarifiers** | | | |
| No. of Units | 3 | 1 | Not in Use |
| No. @ Diameter, ft | 40 | 64 | -- |
| Hp, each | ¾ | ¾ | -- |
| Total Surface Area, sf | 3,770 | 3,216 | -- |
| Peak Surface Loading Rate, gpd/sf | 1,300 | 1,270 | -- |
| Peak Solids Loading Rate, lbs/day/sf | 50 | 50 | -- |
| **Existing RAS/WAS Pumps** | | | Not in Use |
| No. of Pumps | 3 | -- | -- |
| Design capacity (each), gpm | 1,210 | -- | -- |
| Total Dynamic Head, feet | 25 | -- | |
| Hp, each | 15 | -- | -- |
| Manufacturer | ITT | -- | -- |
| **New Clarifiers** | | | |
| No. of new units | 2 | | 2 |

**Table 4-4 Summary of Design Criteria-Secondary Clarifiers**

| Parameter | Initial Facilities for 6.0 mgd AADF | Future Facilities for 9.0 mgd AADF |
|---|---|---|
| Total No. of units | 2 | 4 |
| Diameter, ft | 85 | 85 |
| Inboard Launder Diameter, ft | 79.7 (79'-8") | 79.7 (79'-8") |
| Hp, each | ¾ | ¾ |
| Surface Area (each), sf | 4,985 | 4,985 |
| Total Surface Area, sf | 9,970 | 19,940 |
| Peak Surface Loading Rate, gpd/sf | 900 | 1,130 |
| Peak Solids Loading Rate, lbs/day/sf | 35 | 46 |
| Manufacturer | Eimco | -- |
| **RAS Pumps** | | |
| No. of new pumps | 3 | 3 |
| Total no. of pumps | 3 | 6 |
| Capacity Each, gpm | 1,570 | 1,570 |
| Total Dynamic Head, feet | 20 | 20 |
| Hp, each | 20 | 20 |
| Manufacturer | Vaughan | -- |
| **WAS Pumps** | | |
| No. of new pumps | 2 | 2 |
| Total no. of pumps | 2 | 4 |
| Capacity Each, gpm | 260 | 260 |
| Total Dynamic Head, feet | 12 | 12 |
| Hp, each | 5 | 5 |
| Manufacturer | Vaughan | -- |
| **Scum Pumps** | | |
| No. of new pumps | 2 | 2 |
| Total no. of pumps | 2 | 4 |
| Capacity Each, gpm | 80 | 80 |
| Total Dynamic Head, feet | 12 | 12 |
| Hp, each | 3 | 3 |
| Manufacturer | Diadisk | -- |

## 4.5 Filters

A new yard pipe will be provided to convey effluent from the existing clarifiers to a junction point with the effluent pipes from the new clarifiers (see yard piping plan). Secondary effluent will flow by gravity to the filter feed pump station. The filter feed pump station will convey secondary effluent to the new deep bed filters. The filter feed pumps will be vertical turbine pumps with VFDs. The pumps will provide sufficient capacity so that with one unit out of service the remaining pumps can handle peak flows. Three filter feed pumps will be provided, each pump with a capacity of 6,775 gpm. The firm pump capacity will be 13,550 gpm (19.5 mgd). Space will be provided for the addition of a future pump during future expansion to 9.0 mgd AADF. A slide gate will be provided between the filter feed pump station and transfer pump station to discharge to the reject storage tanks for substandard water. Provision will be made to return reject water from the ground storage tanks to the filter feed pump station for further treatment.

New deep bed filters will be provided. Each filter has a surface area of 520 sf and consists of 6 feet of sand and 15-inches of gravel. Each filter will provide 1.5 mgd AADF and 4.5 mgd PHF of capacity. At the current design flow of 6.0 mgd AADF and anticipated influent peak hydraulic flows of 3 times the AADF a total of four (4) filters will be required. Filter influent injection and mixing for sodium hypochlorite and polymer solution will be provided prior to entering the tanks.

The flow will enter the filter structure through an influent channel. Each filter tank will have an influent butterfly valve to isolate it for maintenance purposes. Secondary effluent will flow through an influent trough and over a weir to the filter bed. The filtered effluent will collect beneath the filter media and flow by gravity to the filter clearwell. Filtered effluent will exit the clearwell over a weir, collect in the effluent pipe and will flow by gravity to the chlorine contact tanks.

The filters will be provided with an air and water backwash system. Backwash pumps and blowers (one active and one standby each) and associated valves will be provided for each filter to clean the media. The backwash system is initiated based on the tank water level. Once the water level rises to a preset level, the blowers will initiate and will provide sufficient airflow to scour the filter. Next the backwash pumps will initiate and a concurrent air and water washing will occur. The backwash pumps will pump filtered effluent from the clearwell upward through the filter media. This is followed by a water only backwash cycle to remove any trapped air. The backwash overflows into the influent trough and is discharged in to a backwash collection channel. The backwash will be discharged into a mudwell located adjacent to the filters. Backwash water from the mudwell will flow by gravity to the plant drain system.

A sample pump will be located in the filter clearwell. A filtered water sample will be pumped to a sampling area for continuous turbidity analysis. Turbidity will be indicated and recorded by the SCADA system. The sampling area will be located at the effluent transfer pump station. If the turbidity is high (this is a surrogate parameter to the suspended solids analysis), the SCADA system will close the automatic valves to the reuse storage tanks and open the valves to the reject storage tanks.

Table 4-5 summarizes design criteria for the filters.

**Table 4-5 Summary of Design Criteria-Filters**

| Parameter | Initial Facilities for 6.0 mgd AADF | Future Facilities for 9.0 mgd AADF |
|---|---|---|
| **Filter Feed Pump Station** | | |
| No. of new pumps | 3 | 1 |
| Total no. of pumps | 3 | 4 |
| Capacity per Pump, gpm | 6,775 | 6,775 |
| Total Firm Pump Capacity, gpm | 13,550 | 20,325 |
| Total Dynamic Head, feet | 24 | 24 |
| Hp, each | 75 | 75 |
| Manufacturer | Peerless or Goulds | -- |
| **Deep Bed Filters** | | |
| No. of New Filters | 4 | 2 |
| Total No. of Filters | 4 | 6 |
| AADF per Unit, mgd | 1.5 | 1.5 |
| PHF per Unit, mgd | 4.5 | 3.75 |
| Filter Area (ea.), sf | 520 | 520 |
| Peak Hydraulic Loading Rate, gpm/sf | 6.0 | 5.0 |
| Manufacturer | Leopold | -- |
| **Filter Backwash Pumps** | | |
| No. of New Units | 2 | -- |
| Total No. of Units | 2 | 2 |
| Capacity per Pump, gpm | 3,200 | -- |
| Total Firm Pump Capacity, gpm | 3,200 | -- |
| Total Dynamic Head, feet | 28 | -- |
| Hp, each | 40 | -- |
| **Filter Backwash Blowers** | | |
| No. of New Units | 2 | -- |
| Total No. of Units | 2 | 2 |
| Capacity per blower, scfm | 2,600 | -- |
| Total firm blower capacity, scfm | 2,600 | -- |
| Differential pressure, psig | 10.6 | -- |
| Hp, each | 100 | -- |

## 4.6 Disinfection

The disinfection process will be centralized in an area adjacent to the proposed filter. Filtered effluent will flow by gravity into a junction box located at the head of the chlorine contact tank. It will then be split between two (2) rapid mix basins. The rapid mix chambers provide rapid mixing of sodium hypochlorite before contact.

The chlorine contact tank will be constructed of poured-in-place reinforced concrete. The chlorine contact tank will be designed with two (2) contact channels. Each channel will be 213, 325 gallons. The tanks will be baffled to prevent short-circuiting of the flow. The baffled walls will also be constructed from reinforced concrete. Each chlorine contact channel is designed to provide the required "CT" value for 50% of PHF with one contact channel out of service in accordance with Class I Reliability requirements. At the design flows, 6.0 mgd AADF and 18.0 mgd PHF, the contact time in the chlorine contact tank is 50 and 17 minutes, respectively.

The flow through each contact channel will discharge over an exit weir. The flow over the weirs will be measured/totalized and used to pace the sodium hypochlorite pumps for each rapid mix tank. Space has been reserved on the site for an additional contact tank for the expansion to 9.0 mgd AADF, 22.5 mgd PHF.

## 4.7 Transfer Pumps

The disinfected reclaimed water from the proposed chlorine contact tanks will flow by gravity into the wet well of the transfer pump station. The transfer pump station will be a cast-in-place concrete structure designed for the 6.0 mgd AADF design condition. This station will include three (3) constant speed vertical turbine pumps with a spare slot for one future pump. The pumps will be controlled automatically by the water level in the wet well. One pump is designed as a standby unit to meet Class I Reliability.

Design criteria for the effluent pump station are summarized in the table below:

**Table 4-6 Summary of Design Criteria-Transfer Pump Station**

| Parameter | Initial Facilities for 6.0 mgd AADF | Future Facilities Added for 9.0 mgd AADF |
|---|---|---|
| No. of new pumps | 3 | 1 |
| Total no. of pumps | 3 | 4 |
| Capacity per Pump, gpm | 6,250 | 6,250 |
| Total Firm Pump Capacity, gpm | 12,500 | 18,750 |
| Total Dynamic Head, feet | 50 | 50 |
| Hp Each | 125 | 125 |
| Manufacturer | Peerless or Goulds | -- |

In the event of high turbidity, indicating that the reclaimed water may not be suitable for public access reuse, the monitoring system will alert the operator and the SCADA system will initiate closure of the proposed motorized valve in the pipe to the reclaimed water storage tanks and initiate opening of the proposed motorized valve in the pipe to the reject water storage tanks.

A sample pump will be provided at the transfer pump station for pH measurement. A refrigerated composite sampler will be provided for compliance monitoring of the effluent.

## 4.8 Effluent Pumps

The effluent pump station is designed to withdraw water from the reject storage tanks and/or the reclaimed water storage tanks. Reject water may be pumped to the injection well through the effluent pump station. The same pumps will also be used to pump to the County's reuse system. Automatic valves will be used to select the withdrawal point for the pump station. If the station has been used to pump reject water, good quality reclaimed water will continue to be pumped to the reject tank until the pumps and pipes have been adequately flushed. Four variable speed pumps will be provided initially. Space is available for enlarging the pump station as part of subsequent expansions.

Table 4-7 summarizes design criteria for the effluent pump station.

**Table 4-7 Summary of Design Criteria-Effluent Pump Station**

| Parameter | Initial Facilities for 6.0 mgd AADF | Future Facilities Added for 9.0 mgd AADF |
|---|---|---|
| Number of Effluent Pumps | 4 | Space Available for Expansion |
| Design Capacity, gpm (Injection Wells) | 1,800 | -- |
| Total Dynamic Head, feet (Injection Well) | 139 | -- |
| Design Capacity, gpm (Reclaimed Water System) | 1,500 | -- |
| Total Dynamic Head, feet (Reclaimed Water System) | 70 | -- |
| Total Firm Pump Capacity, gpm | 5,220 | -- |
| Hp, Each | 75 | -- |
| Manufacturer | Peerless or Goulds | -- |

## 4.9 Chemical Feed Systems

The proposed chlorine contact tanks and filter units will require an additional sodium hypochlorite feed and storage system. These facilities will consist of bulk storage tanks, fill station, feed pumps, controls, electrical service and panels, discharge piping and valves, injectors and diffusers. The bulk sodium hypochlorite will be delivered to the plant site at typical solution strength of 10-12%.

Design criteria for the hypochlorite feed system are summarized in Table 4-8.

**Table 4-8 Summary of Design Criteria Sodium Hypochlorite Feed System**

| Parameter | Initial Facilities for 6.0 mgd AADF | Future Facilities Added for 9.0 mgd AADF |
|---|---|---|
| **Storage Tanks** | | |
| No. of new storage tanks | 2 | 1 |
| Total no. of storage tanks | 2 | 3 |
| Type | Double Containment | |
| Bulk Tank Volume, gallons | 5,000 | 5,000 |
| Manufacturer | Snyder | -- |
| **Feed Pumps** | | |
| No. of new pumps | 3 | 1 |
| Total no. of pumps | 3 | 4 |
| Type | Diaphragm | |
| Pump Capacity Each, gph | 1.25-76 | 1.25-76 |
| Manufacturer | Prominent | -- |

The sodium hypochlorite system will be housed in a new building featuring open walled construction for natural ventilation. Spill containment volume will be provided through use of double walled tanks. The hypochlorite storage tank will provide a minimum of 10 days of storage not to exceed 30 days. The new building will also house the diaphragm metering pumps. These pumps will be used to transfer the sodium hypochlorite to the application points including a location prior to the new filters and to the new chlorine contact tanks. The pre-filter application point will be provided to assist with algae control.

A total of three (3) metering pumps will be provided. Two (2) of the pumps will be piped to pump directly to the rapid mix chambers prior to the chlorine contact tank. One (1) pump will pump continuously with the other being the standby pump. Each pump will be driven with VFDs. A third pump will be piped to the pre-filter area for filter maintenance. The operator will manually adjust the pump flow rate for the pre-filter application point.

## 4.10 Sludge Holding & Dewatering

There are three (3) existing digesters (0.217 mg) at the Three Oaks WWTP. The Three Oaks WWTP is currently required by FDEP to treat sludge to Class B standards. The volume of the existing digesters is a limiting factor for aerobic sludge digestion. The existing digesters will provide 7.6 days of storage at the maximum month flow (9.6 days at 6.0 mgd AADF). To extend the residence time in the digester the sludge could be thickened to approximately 3% percent solids. This would increase the residence time to a minimum of 24 days. This should be adequate time to achieve the volatile solids reduction necessary for Class B sludge. Compliance with Class B may be demonstrated by conducting a bench-scale test for additional aerobic digestion and bacteriological testing.

BioWin (Envirosim Associates Ltd, Flamborough, Ontario) models were run in order to simulate the aerobic digestion that is likely to occur in the existing aerobic digesters. The model predicted approximately 26.0% volatile solids reduction resulting in a digested solids concentration of 2.47% in the digester. For this case the solids retention time of the digester is 24.2 days. In order to simulate the Additional Digestion Test an additional batch reactor was added to the model in line with the aerobic digester. A portion of the effluent of the digester was diverted to this reactor where it was aerated for 30 days (30 day SRT). Effluent was also diverted to dilute the solids to less than 2% (1.89% in this model run). The model predicted additional VSS reduction of approximately 6.4%. This result is an approximation of what is likely to occur in the lab scale Additional Digestion Test. Since the VSS reduction is significantly less than 15%, the Additional Digestion Test should be able to demonstrate compliance with the vector attraction reduction requirements.

In addition the BioWin model was used to determine if the existing digester blowers have sufficient capacity to aerate the thickened solids. The existing digesters are aerated using coarse bubble diffusers. The model predicted a required air flowrate of approximately 2,570 SCFM at maximum month flow. Based on a mixing requirement of 30 SCFM per 1000 cf, 2,607 SCFM is required to mix all three digesters. There are four (4) existing blowers each with a capacity of 900 SCFM. With one blower acting as a spare there is 2,700 SCFM of firm blower capacity. Therefore, there is enough existing blower capacity to aerate and mix the thickened solids during digestion for the proposed design.

A belt filter press will be provided. The sludge dewatering press will be capable of operating in a thickening mode and dewatering mode. The capacity of the belt filter press in the thickening and dewatering modes will be 400 gpm and 110 gpm respectively. Sludge from the digester will be pumped to the belt press for thickening. The thickened sludge will be pumped to the digesters. The digested sludge will be pumped back to the belt filter press for dewatering. Thickening will only be provided when the amount of waste sludge is such that thickening is required to obtain adequate volume in the digesters.

Sludge thickening and dewatering will occur in the press building consisting of a pre-engineered metal roof with open sides. The building will be designed to accommodate up to two belt presses and a trailer loading area. The control panel for the press will be positioned on an operator's platform. From the platform, the operator will be able to visually inspect the condition of the sludge on the belt and make adjustments to the sludge feed pumps and to the polymer feed rate.

The belt filter press will be fed from the same set of variable speed progressing cavity pumps during the thickening and dewatering mode located at a new belt filter press feed pump station. A separate thickened sludge return pump will be provided to return the thickened sludge to the digesters. Two belt filter press feed pumps will be provided; one (1) active and the other will be a standby. Thickened sludge will drop into an inlet chute to the thickened sludge return pump. The pump station will consist of two (2) variable speed progressing cavity pumps, one (1) active and one (1) standby.

Dewatered sludge cake will drop into the inlet chute of a variable speed auger pump. The sludge will be pumped to the trailer loading area. At least three sludge drops will be provided for the convenience of distributing the sludge along the full trailer length. The operator will select the drop to use by operating a knife gate valve at floor level. A compressed air cleaning system will be provided to evacuate the trailer feed pipes.

Table 4-9 summarizes the design criteria for the Sludge Dewatering Process.

**Table 4-9 Summary of Design Criteria-Sludge Dewatering**

| Parameter | Initial Facilities for 6.0 mgd AADF | Future Facilities Added for 9.0 mgd AADF |
|---|---|---|
| **Belt Filter Press Feed Pumps** | | |
| No. of new units | 2 | 1 |
| Total no. of units | 2 | 3 |
| Capacity per Pump, gpm | 110-400 | 110-400 |
| Hp, each | 40 | 40 |
| Manufacturer | Seepex | -- |
| **Thickened/Return Sludge Pumps** | | |
| No. of new units | 1 | 1 |
| Total no. of units | 1 | 2 |
| Capacity per Pump, gpm | 90 | 90 |
| Hp, each | 5 | 5 |
| Manufacturer | Seepex | Seepex |
| **Belt Filter Press** | | |
| No. of new units | 1 | 1 |
| Total no. of units | 1 | 2 |
| Belt Width, meter | 2 | 2 |
| Average Sludge Yield, lbs/day | 5,725 | 8,590 |
| Press Throughput, lbs/hr | 1,400 | 1,400 |

### Table 4-9 Summary of Design Criteria-Sludge Dewatering

| | | |
|---|---|---|
| Percent Solids | 18% | 18% |
| Operations, days/hrs per day | 8/7 | 8/5 |
| Hp, each | 6 | 6 |
| Manufacturer | Ashbrook | -- |
| **Cake Sludge Pump** | | |
| No. of new units | 1 | 1 |
| Total no. of units | 1 | 2 |
| Maximum Capacity, gpm | 15 | 15 |
| Hp, each | 15 | 15 |
| Manufacturer | Seepex | -- |

## 4.11 Plant Drain Pump Station

Two (2) submersible plant drain pump stations will be provided. The first drain pump station will be designed to accept washwater from the screenings compactor and grit equipment, and other drain flows from the preliminary treatment structure. The pump station will consist of two (2) constant speed submersible pumps. Each pump will have a capacity of 450 gpm. The firm pump capacity of the plant drain pump station will be 450 gpm. A 6-inch diameter force main will exit the pump station and discharge to the preliminary treatment structure upstream of the influent flow meter.

The second drain pump station will be designed to accept backwash water from the proposed filter units and from the proposed dewatering process. In addition, tank drains from new process tanks, except the preliminary treatment structure, will be directed by gravity to the pump station. The pump station will consist of three (3) constant speed submersible pumps. Each pump will have a capacity of 450 gpm. The firm pump capacity of the plant drain pump station will be 900 gpm. A 12-inch diameter force main will exit the pump station and discharge to the preliminary treatment structure upstream of the influent flow meter. Larger pumps will need to be installed for the expansion to 9.0 mgd, AADF.

Table 4-10 summarizes the design criteria for the Plant Drain Pump Stations.

**Table 4-10 Summary of Design Criteria-Plant Drain Pump Station**

| Parameter | Initial Facilities for 6.0 mgd AADF | Future Facilities for 9.0 mgd AADF |
|---|---|---|
| Plant Drain Pump Station No. 1 | | |
| Location | @ Preliminary Treatment Structure | -- |
| Number of Plant Drain Pumps | 2 | -- |
| Capacity per Pump, gpm | 450 | -- |
| Total Firm Pump Capacity, gpm | 450 | -- |
| Total Dynamic Head, feet | 53 | -- |
| Hp, each | 10 | -- |
| Manufacturer | Flygt | -- |
| Plant Drain Pump Station No. 2 | | |
| Location | Near filters | |
| Number of Plant Drain Pumps | 3 | Replace with larger pumps |
| Capacity per Pump, gpm | 450 | -- |
| Total Firm Pump Capacity, gpm | 900 | -- |
| Total Dynamic Head, feet | 43 | -- |
| Hp, each | 10 | -- |
| Manufacturer | Flygt or ABS | -- |

## 4.12 Site Work and Stormwater Management

Paving will be provided as required for vehicular access to plant structures (refer to site plan). The majority of the new facilities will be open capture and will not create runoff. Sufficient retention pond volume will be needed to treat one inch of runoff from the site. An expansion to the existing retention pond will be provided. To provide additional treatment, swale drainage will be provided because of the treatment benefit associated with it. Some culvert conveyance will be required to route pond overflows to the point of discharge. Site lighting will be provided to comply with required local codes and standards.

## 4.13 Electrical Upgrades

### Introduction

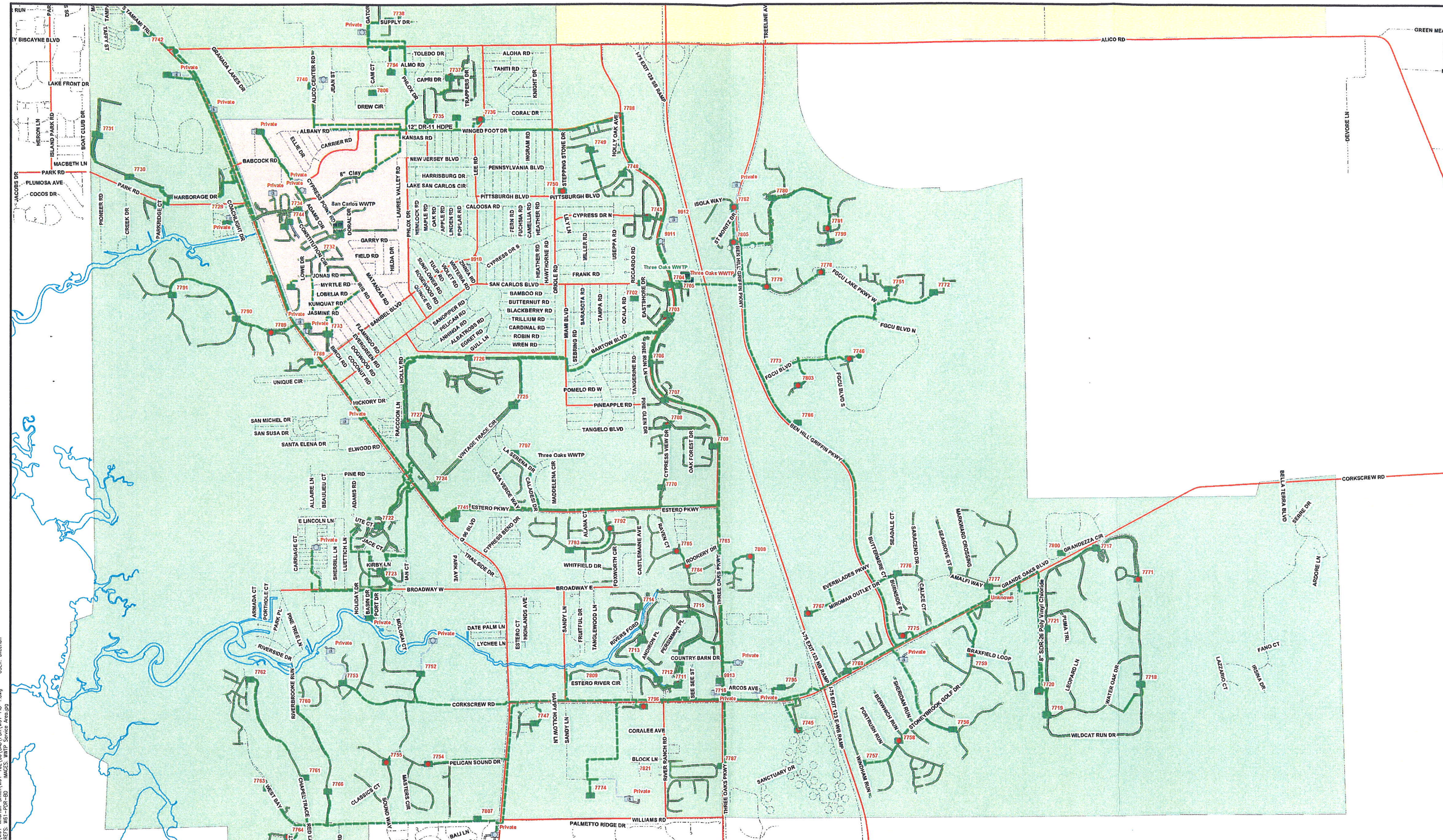
The Three Oaks Waste Water System receives utility electrical service from Florida Power and Light Co (FPL). There are presently three electrical services on the site that provide power to the existing wastewater plant facilities. Florida Power and Light's equipment consists of a radial underground primary feeder from an area substation to two padmount transformers. The service padmount transformers step the 23KV primary voltage down to a 480-volt, three phase grounded wye service. One of the FPL transformers is located near the operations building in the south central portion of the site and the other is in the north eastern corner of the site adjacent to the digesters. We have identified the services per the naming convention as shown on the as-built drawings from the last expansion of the plant. The south central padmount and FPL meter serves Motor Control Center MCC5. MCC5 provides power to the operations building and the "old" part of the plant in the southern area. LCU has recently upgraded this service with a new generator. The northeast service transformer and single FPL meter provides power to two services in the north half of the plant. The services are identified from the previous upgrade documents as MCC1/2 and MCC4. Each service has a separate standby power system and automatic power transfer system.

Lee County Utility facilities start at the transformer secondary with an underground duct bank that feeds the main service disconnects. The existing service mains are rated at 480V-3phase, 600 amps. The main service feeds the normal side of 600amp automatic transfer switches. The standby power side of the automatic transfer switch is fed from diesel driven standby power generators. The existing generators are rated at 480 volts, 3 phase, 400KW for service MCC1/2 and service MCC4 and 350Kw for service MCC5. The transfer switch feeds the associated Motor control centers. The total load of the plant can be transferred from normal FPL utility to the standby power source through the three automatic transfer switches.

### Existing Facility Capacity Evaluation

The existing south central service (MCC5) will not be affected by the proposed project. We will concentrate our comments to the northeastern services. The northeastern FPL service transformer has a kilowatt peak demand (Kwd) recorded by Florida Power and Light Co. (FPL) for the previous 24 months of 318Kwd. The FPL transformer is rated for 300Kva and thus the transformer is slightly overloaded during peak load demand times. The FP&L recorded peak occurred during April of 2005. The Kva demand of the plant can be estimated at 374Kva at 0.85 power factor. The capacity of each of the two existing power services is rated for approximately 400kw, (500 Kva). The loading of the two services is unbalanced. Although the peak load as recorded by FPL is within the rating of each service, MCC1/2 serves 68% (523hp) of the operating load (operating load = connected load minus standby equipment) and MCC4 serves only 32% (250hp) of the existing load. The 523 hp operating load on MCC1/2 has the potential of overloading the electrical gear.

The proposed expansion project will eliminate the overload potential of MCC1/2 by removing 160hp of operating equipment. The proposed expansion also anticipates adding additional load to MCC4 of a net 105hp. Both MCC1/2 and MCC4 services will be balanced at approximately 370hp each after

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| **BOYLE** ENGINEERING CORPORATION<br>320 East South Street Orlando, FL 32801<br>FL. ENGINEERING BUS. NO. 2005<br>407-425-1100 | THREE OAKS WWTP EXPANSION<br>PRELIMINARY ENGINEERING REPORT<br><br>SERVICE AREA | BEC PROJECT NO.<br><br>FE-W61-102-00 | FIGURE<br><br>1 |

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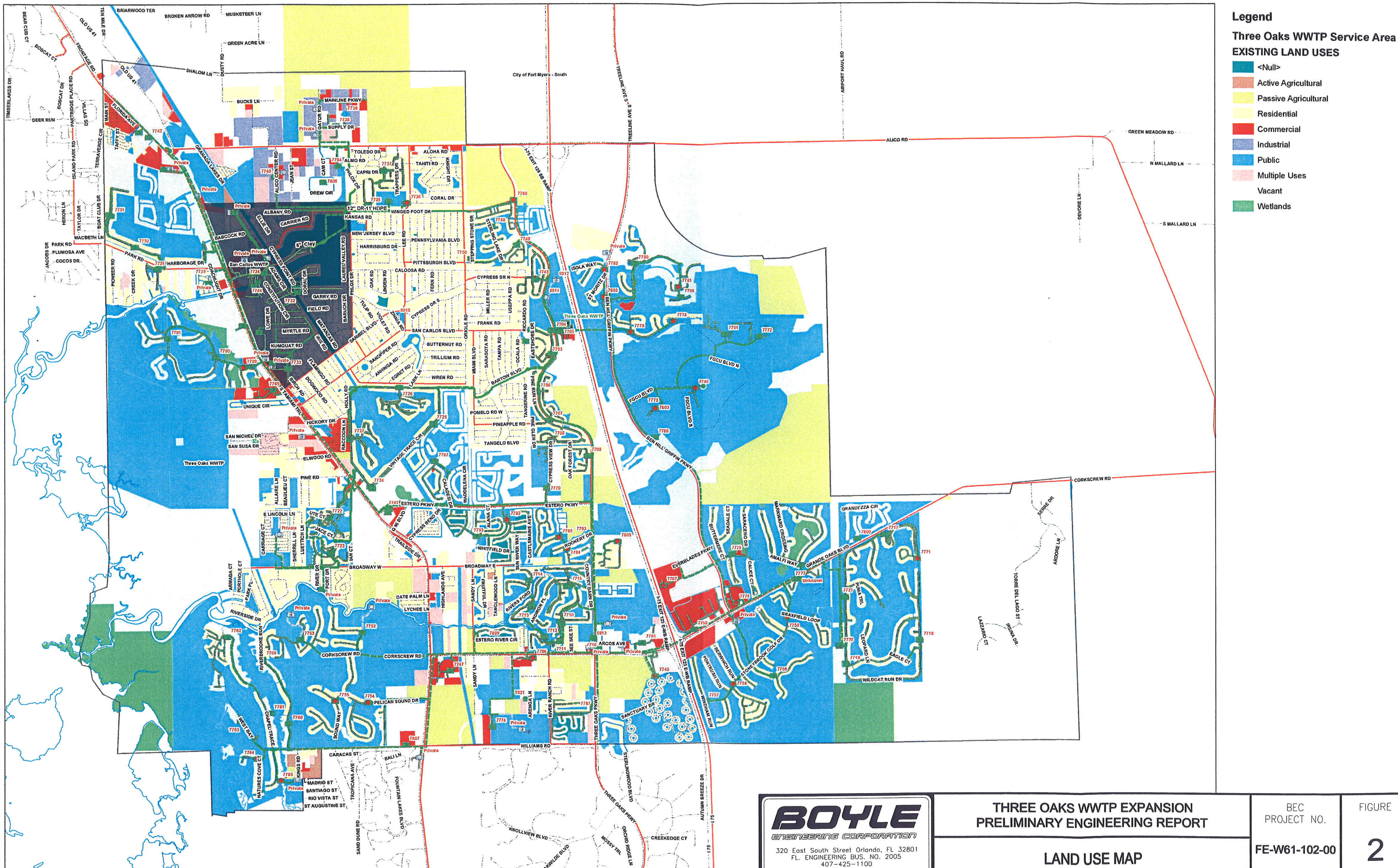
DATE: Oct 23, 2005 9:03am XREFS: W61-PDR-BD IMAGES: WWTP Service Area.jpg

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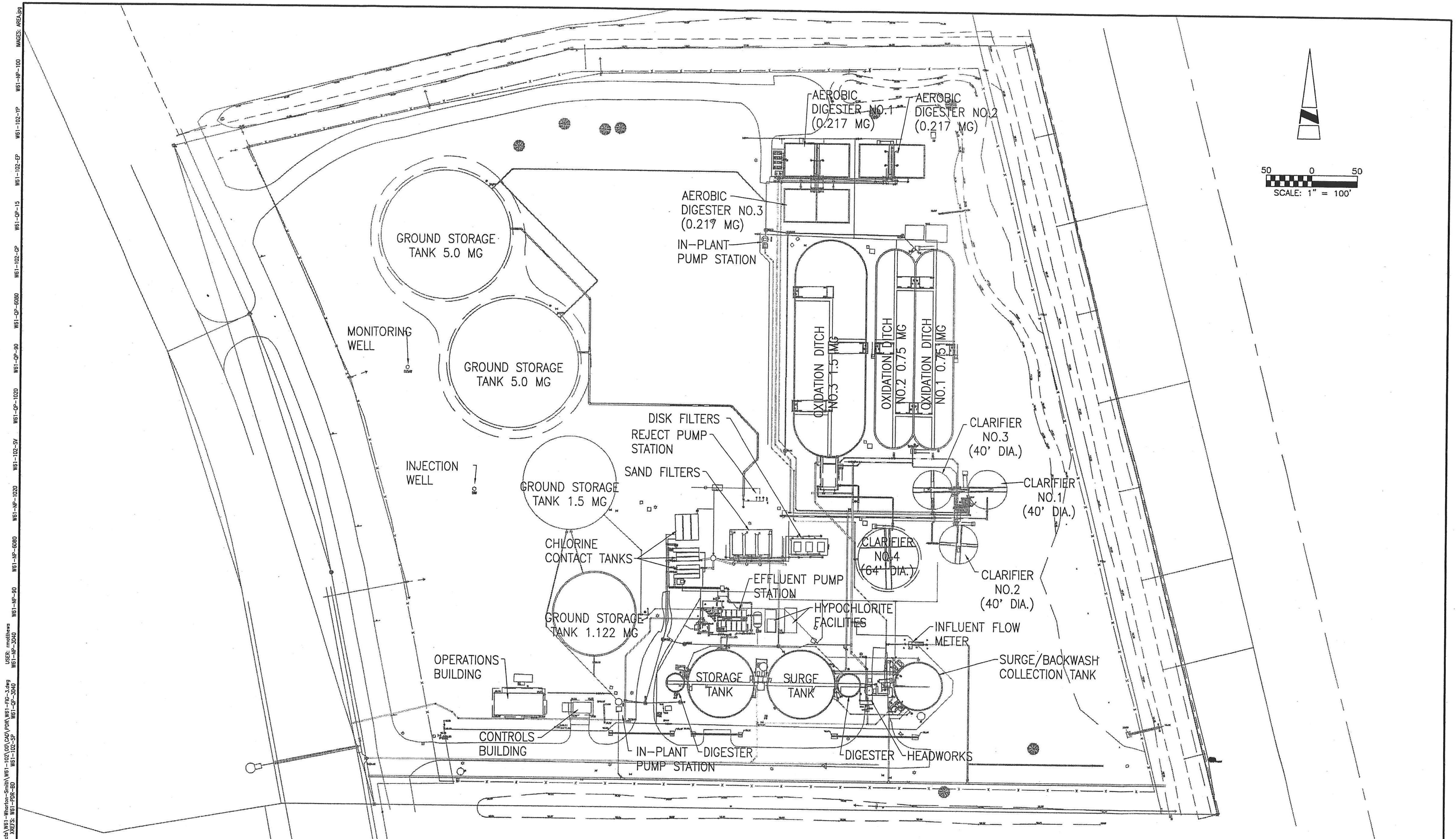
FL. ENGINEERING BUS. NO. 2005
407-425-1100

THREE OAKS WWTP EXPANSION
PRELIMINARY ENGINEERING REPORT

LAND USE MAP

BEC PROJECT NO. FE-W61-102-00

FIGURE 2

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THREE OAKS WWTP EXPANSION
PRELIMINARY ENGINEERING REPORT

EXISTING SITE PLAN

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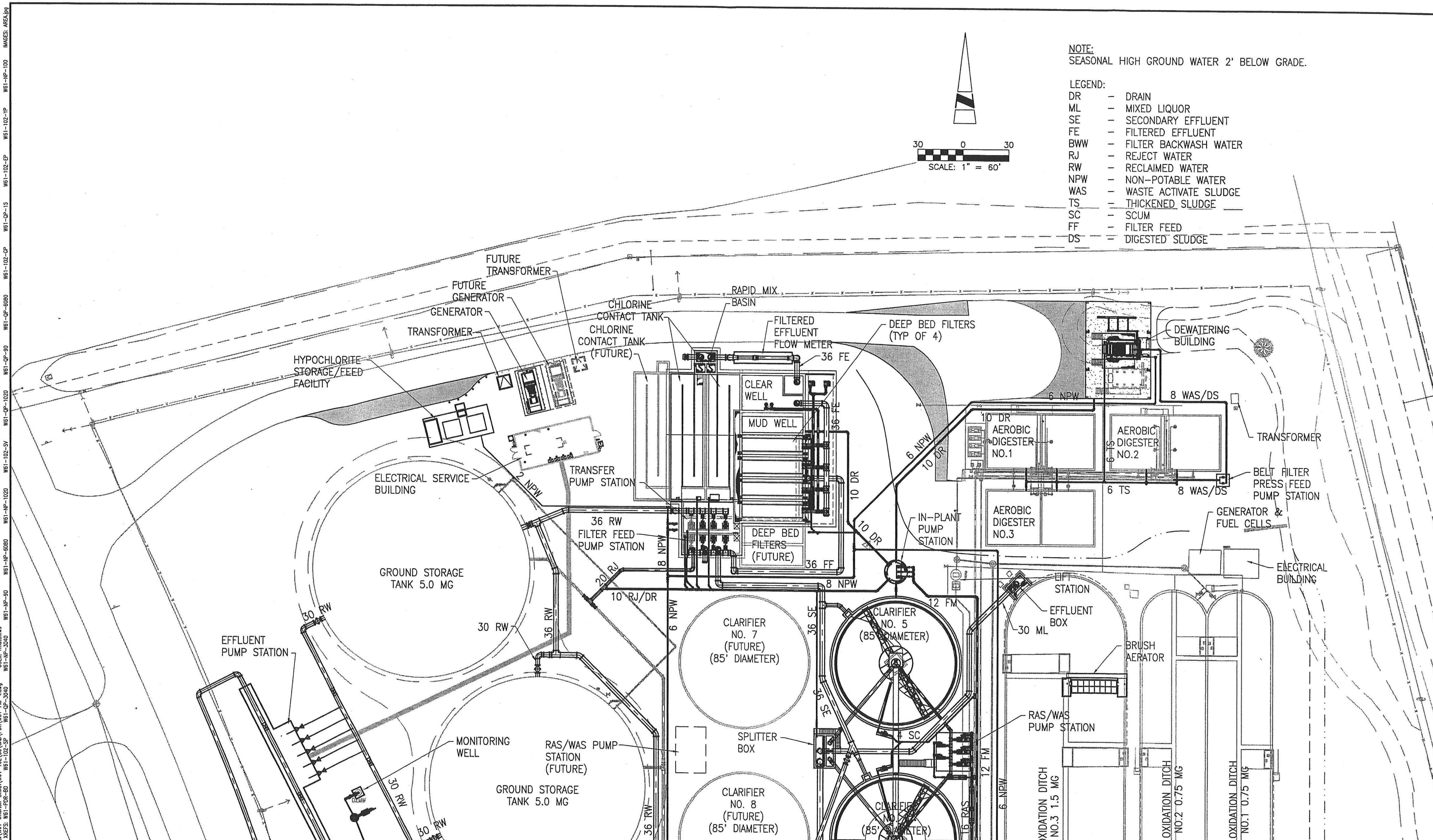
FIGURE 3

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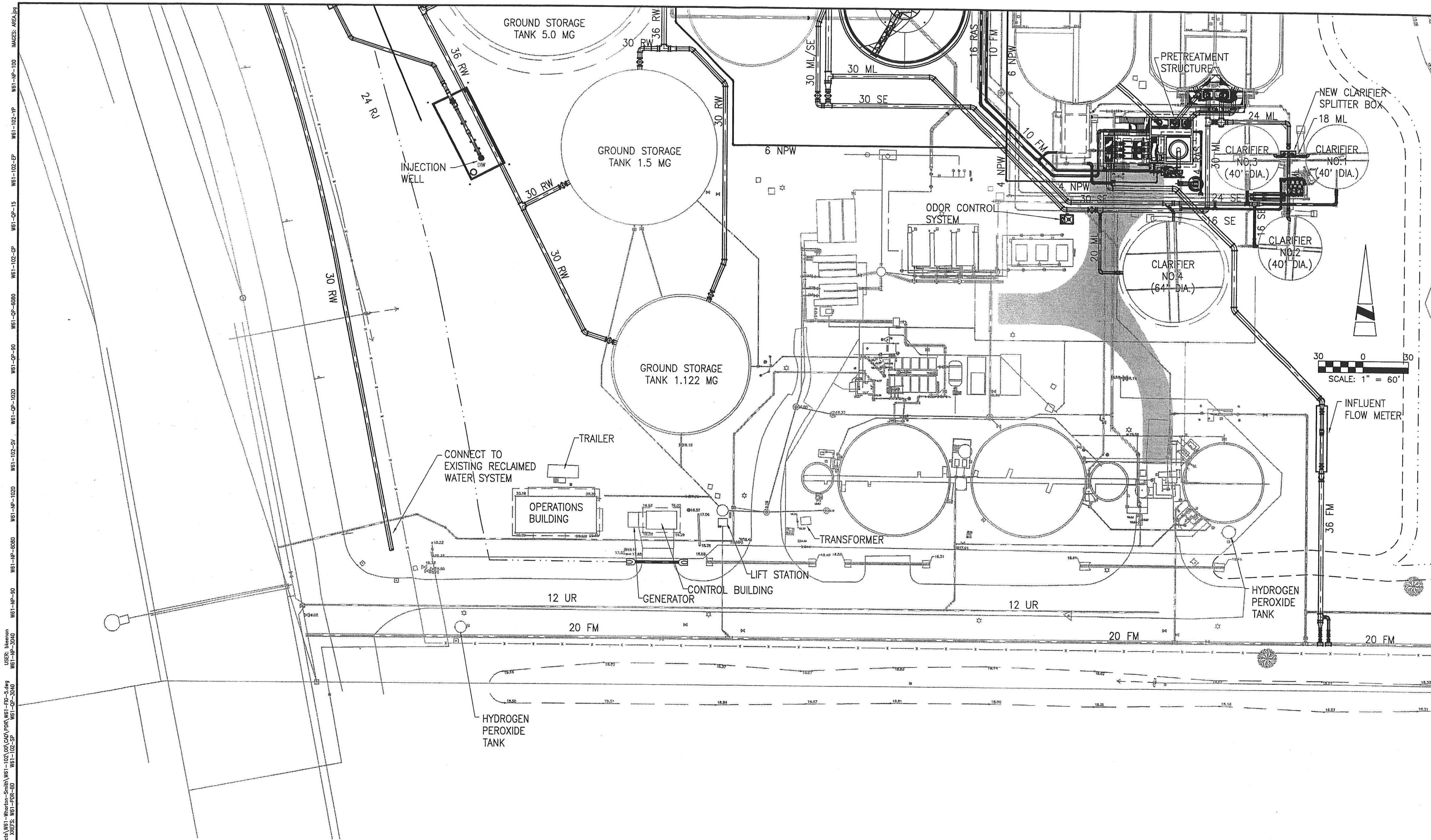
| THREE OAKS WWTP EXPANSION PRELIMINARY ENGINEERING REPORT | BEC PROJECT NO. | FIGURE |
|---|---|---|
| PROPOSED SITE PLAN | FE-W61-102-00 | 4 |

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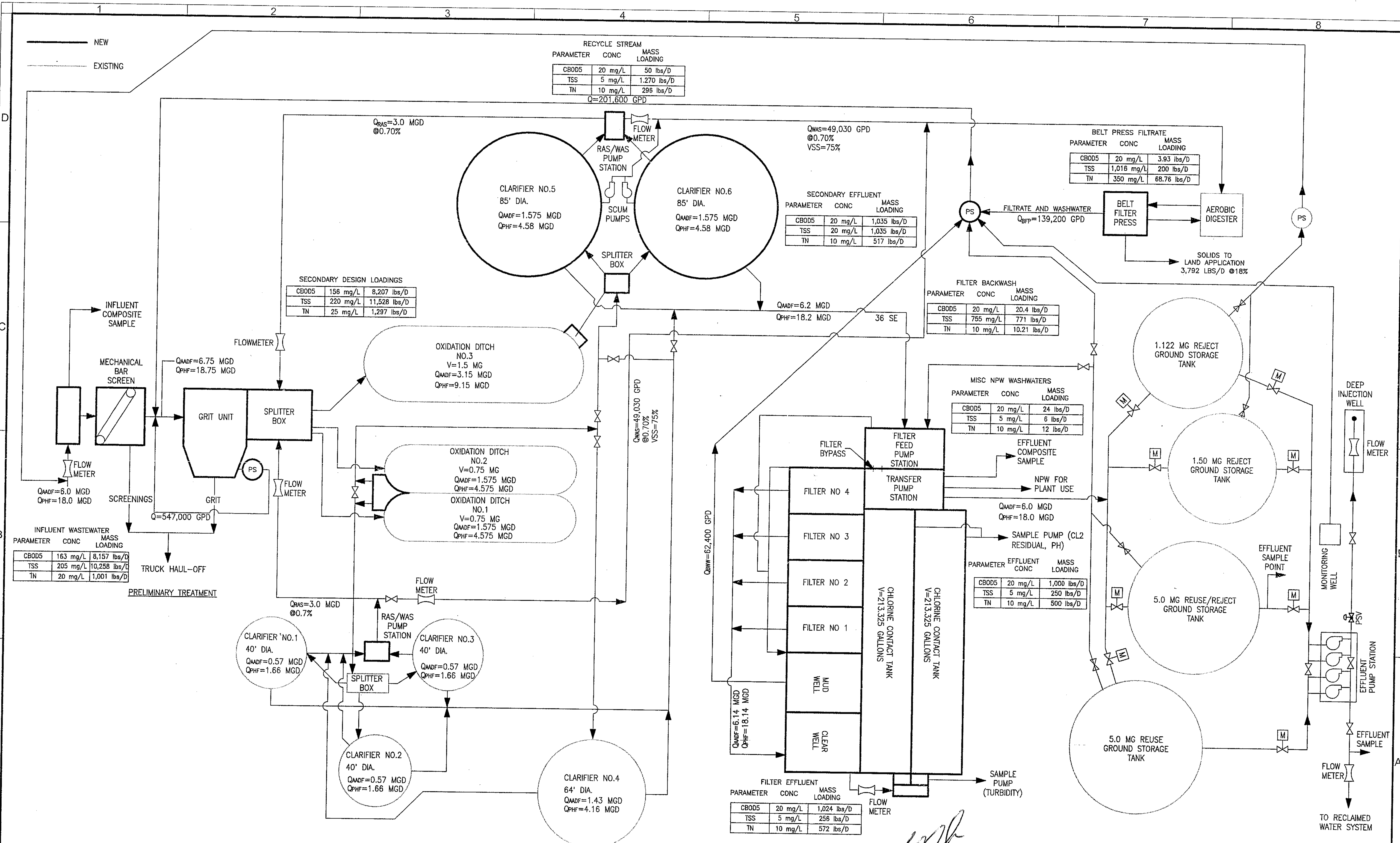
THREE OAKS WWTP EXPANSION
PRELIMINARY ENGINEERING REPORT

PROPOSED SITE PLAN

BEC PROJECT NO.: FE-W61-102-00

FIGURE 5




**RECYCLE STREAM**

| PARAMETER | CONC | MASS LOADING |
|---|---|---|
| CBOD5 | 20 mg/L | 50 lbs/D |
| TSS | 5 mg/L | 1,270 lbs/D |
| TN | 10 mg/L | 296 lbs/D |

**BELT PRESS FILTRATE**

| PARAMETER | CONC | MASS LOADING |
|---|---|---|
| CBOD5 | 20 mg/L | 3.93 lbs/D |
| TSS | 1,016 mg/L | 200 lbs/D |
| TN | 350 mg/L | 68.76 lbs/D |

**SECONDARY EFFLUENT**

| PARAMETER | CONC | MASS LOADING |
|---|---|---|
| CBOD5 | 20 mg/L | 1,035 lbs/D |
| TSS | 20 mg/L | 1,035 lbs/D |
| TN | 10 mg/L | 517 lbs/D |

**SECONDARY DESIGN LOADINGS**

| | | |
|---|---|---|
| CBOD5 | 156 mg/L | 8,207 lbs/D |
| TSS | 220 mg/L | 11,528 lbs/D |
| TN | 25 mg/L | 1,297 lbs/D |

**FILTER BACKWASH**

| PARAMETER | CONC | MASS LOADING |
|---|---|---|
| CBOD5 | 20 mg/L | 20.4 lbs/D |
| TSS | 755 mg/L | 771 lbs/D |
| TN | 10 mg/L | 10.21 lbs/D |

**MISC NPW WASHWATERS**

| PARAMETER | CONC | MASS LOADING |
|---|---|---|
| CBOD5 | 20 mg/L | 24 lbs/D |
| TSS | 5 mg/L | 6 lbs/D |
| TN | 10 mg/L | 12 lbs/D |

**INFLUENT WASTEWATER**

| PARAMETER | CONC | MASS LOADING |
|---|---|---|
| CBOD5 | 163 mg/L | 8,157 lbs/D |
| TSS | 205 mg/L | 10,258 lbs/D |
| TN | 20 mg/L | 1,001 lbs/D |

| PARAMETER | EFFLUENT CONC | MASS LOADING |
|---|---|---|
| CBOD5 | 20 mg/L | 1,000 lbs/D |
| TSS | 5 mg/L | 250 lbs/D |
| TN | 10 mg/L | 500 lbs/D |

**FILTER EFFLUENT**

| PARAMETER | CONC | MASS LOADING |
|---|---|---|
| CBOD5 | 20 mg/L | 1,024 lbs/D |
| TSS | 5 mg/L | 256 lbs/D |
| TN | 10 mg/L | 572 lbs/D |

REV | DATE | DESCRIPTION | APPR

VERIFY SCALES
BAR IS ONE INCH ON ORIGINAL DRAWING
IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY

DESIGNED BY BLK
DRAWN BY ESM
CHECKED BY VJG
DATE JAN 2006

PROJECT ENGINEER VICTOR J. GODLEWSKI JR., PE
REG NUMBER 33139 EXP DATE 02/28/07
PROJECT NUMBER FE-W61-102-00
CADD STANDARDS BOYLE

Wharton-Smith, Inc. CONSTRUCTION GROUP

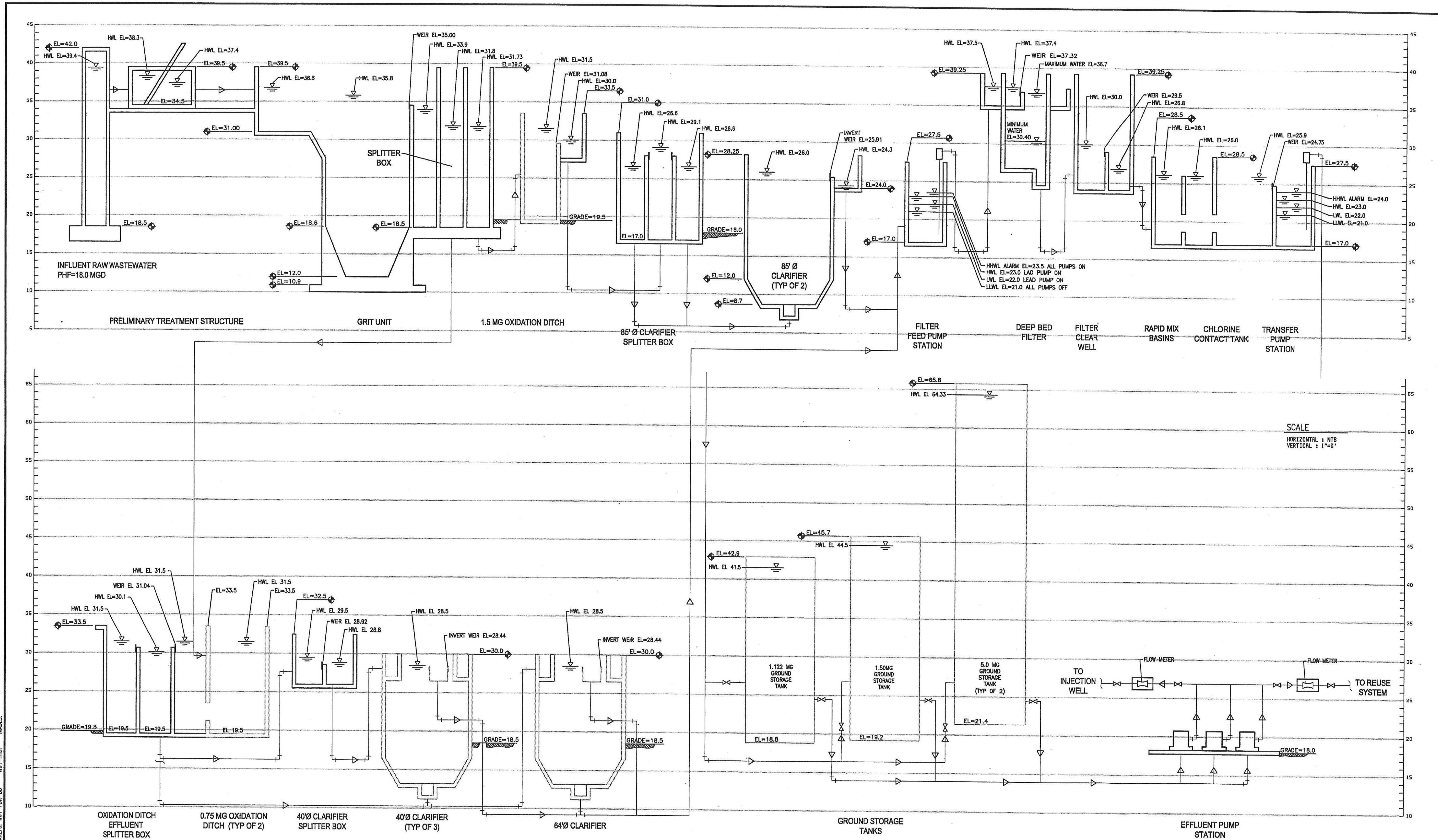
BOYLE ENGINEERING CORPORATION
4415 METRO PARKWAY, SUITE 404
FORT MYERS, FLORIDA 33916
FL. ENGINEERING BUS. NO. 2005
(239) 278-7996

33139 2/19/07

# THREE OAKS WASTEWATER TREATMENT PLANT EXPANSION

## PROCESS FLOW DIAGRAM

DRAWING G-006
SHEET 6 of 000 SHEETS

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THREE OAKS WWTP EXPANSION
PRELIMINARY ENGINEERING REPORT

HYDRAULIC PROFILE

BEC PROJECT NO.
FE-W61-102-00

FIGURE
7

DWG: B:\Orlando\Orlando - Projects\W61-Wharton-Smith\W61-102\00_CAD\PDR\W61-FIG-7.dwg USER: rmatthews
DATE: Oct 21, 2005 3:46pm XREFS: W61-PDR-BD W61-HYDP IMAGES: