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# Nipomo Community Services District

Revised Draft Southland Wastewater Treatment Facility Master Plan

November 2008

**Nipomo Community Services District**

| | |
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# 1.0 INTRODUCTION

## 1.1 Background

The Nipomo Community Services District (District) owns and operates the Southland Wastewater Treatment Facility (WWTF), located east of Highway 101 in the southern portion of San Luis Obispo County, California. The WWTF treats a mixture of domestic and industrial wastewater from part of the Nipomo community under Waste Discharge Requirements Order No. 95-75 (attached as Appendix A) with a permitted capacity of 900,000 gallons per day (gpd) based on the maximum monthly demand. A site plan is included as Figure 1-1.

On February 7, 2006, the District received a Notice of Violation (NOV) from the Regional Water Quality Control Board (RWQCB) for several effluent water quality violations reported during 2005. The letter included directives to investigate the dependability of analytical results, investigate treatment facility improvements, and submit a report of actions needed to correct wastewater treatment deficiencies and discharge violations. To facilitate response to the NOV, the District directed Boyle to perform the following services:

- Prepare an Action Plan for submittal to the RWQCB (completed May 2006);
- Prepare a technical memorandum to address operational improvements to be made in the immediate future (completed July 2006); and
- Prepare a WWTF Master Plan to assist in the strategy for future capital improvements. This report comprises the Master Plan.

## 1.2 Objectives and Scope of Work

The purpose of this study is to identify improvements needed for the WWTF and the Frontage Road trunk line to meet existing and projected demands and to develop a comprehensive Capital Improvements Program. This Master Plan will consider alternative treatment technologies and provide design criteria for a new treatment facility, allowing the District to design and construct improvements necessary to meet the discharge requirements and ultimate build-out demand. Specific tasks performed within this study included:

*Review of plant performance and capacity*: Monitoring data from September 2006 to August 2008 were analyzed to determine flow demands, peaking factors, loading rates, and solids production. This information was used to evaluate the historical performance of the plant. The existing hydraulic and process capacities of the pumps, pipes, ponds, and aeration systems were evaluated.

*Development of design criteria*: Projected build-out flow demands for the years 2010, 2015, 2020, 2025, and 2030 and anticipated future water quality standards were used to develop design criteria. Population and wastewater flow projections from the District's Water and Sewer Master Plan were used to develop flow demands. Peaking factors were developed for use in this analysis, as well.

*Determination of needed facility improvements*: The Study included evaluation of current facility capacity (process, hydraulic, and solids handling) and identification of improvements needed to meet current demands and treatment requirements. These improvements include screening and grit removal facilities, replacement of the Frontage Road Trunk Main, electrical improvements, and sludge handling facilities and strategies. Cost opinions were provided for solar power and for sludge removal from the drying beds, as well.

*Evaluation of alternatives for future plant improvements*: Four treatment processes were evaluated based on the ability to meet future demands. Process flow diagrams, site plans, schematics, and planning-level conceptual cost opinions are provided for each alternative.

*Development of a Capital Improvements Plan*: The schematic diagram, site plan, schedule, and cost are outlined for the recommended improvements.

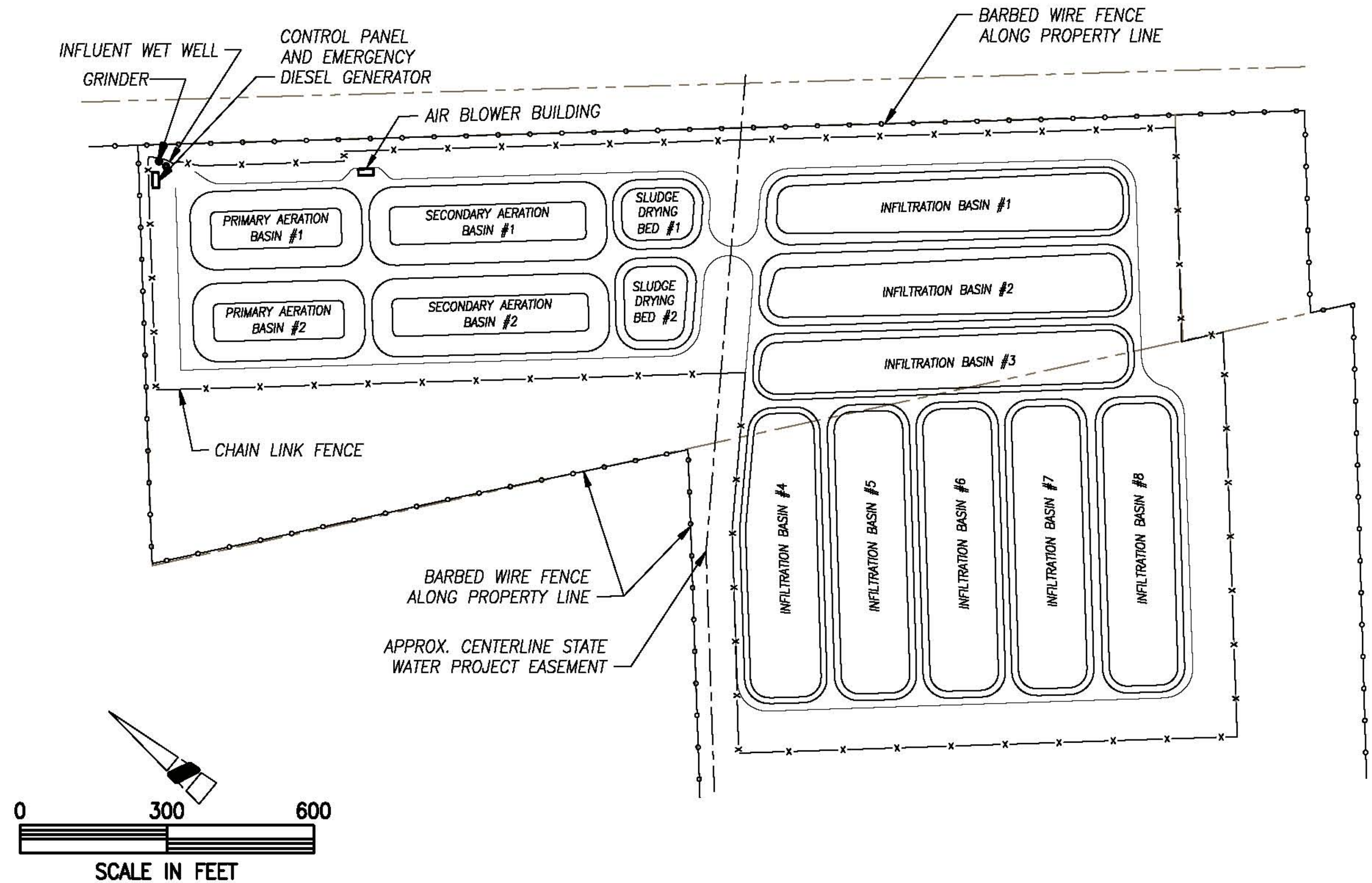


FIGURE **1-1**

**NCSD SOUTHLAND WWTF MASTER PLAN**

**SITE PLAN**

BEC PROJECT NO. **19996.17**

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# 2.0 EXISTING LOADS

## 2.1 Flow Analysis

Several flow rates were analyzed in this study. The *Average Annual Flow (AAF)* is the flow rate averaged over the course of the year and is the base flow for the WWTF. Collection and analysis of 2 years of historical flow data (September 2006 through August 2008) yielded an AAF of 0.59 million gallons per day (mgd).

*Average Wet Weather Flow (AWWF)* was defined as the average daily flow during “wet” months, or months that experience a total rainfall greater than 0.5 inches. San Luis Obispo County provided rainfall data, collected from a gauge at the WWTF. Flow and rainfall records indicate the service area has an AWWF of 0.59 mgd.

*Maximum Month Flow (MMF)* is an important design flow for the Waste Discharge Requirements (WDR’s) since it is the basis of the plants permitted capacity. MMF is the average daily flow during the maximum month. Flow records indicate a MMF of 0.64 mgd over the past two years (January 2007).

*Peak Day Flow (PDF)* is the maximum daily flow rate experienced at the WWTF. Flow records show the PDF to be 1.19 mgd (June 23, 2007).

*Peak Hour Flow (PHF)* is the maximum one-hour flow experienced by the system, and can usually be derived from WWTF records, flow monitoring, or empirical equations used to estimate PHF based on service area population. It is important for hydraulically limited facilities such as pumps, pipes, screens, flow meters, grit removal devices and clarifiers.

*Peak Dry Weather Flow (PDWF)* is the maximum daily flow rate recorded at the WWTF during months when less than 0.5 inches of rain occurs. PDWF for the WWTF is 1.185 mgd (June 23, 2007).

*Peak Wet Weather Flow (PWWF)* is the maximum daily flow rate recorded at the WWTF during months when 0.5 inches or more rain is recorded. The larger of the PWWF and the PDWF is used as the PDF. PWWF for the City is 0.993 mgd (December 22, 2006).

Table 2-1 summarizes the average and peak daily flows for each month. Also included are the monthly precipitation and peak and average flows. Table 2-2 summarizes existing flows and peaking factors.

**Table 2-1 Historic Flow and Precipitation Data**

| Month | ADF (mgd) | PDF (mgd) | Precipitation (in) |
|---|---|---|---|
| Sep-06 | 0.570 | 0.737 | 0.00 |
| Oct-06 | 0.584 | 0.772 | 0.01 |
| Nov-06 | 0.586 | 0.785 | 0.42 |
| Dec-06 | 0.597 | 0.993 | 2.85 |
| Jan-07 | 0.638 | 0.829 | 0.14 |
| Feb-07 | 0.623 | 0.835 | 0.87 |
| Mar-07 | 0.599 | 0.917 | 0.48 |
| Apr-07 | 0.589 | 0.772 | 0.59 |
| May-07 | 0.580 | 0.756 | 0.08 |
| Jun-07 | 0.596 | 1.185 | 0.00 |
| Jul-07 | 0.585 | 1.083 | 0.00 |
| Aug-07 | 0.572 | 0.850 | 0.10 |
| Sep-07 | 0.583 | 1.184 | 0.00 |
| Oct-07 | 0.575 | 0.803 | 0.15 |
| Nov-07 | 0.578 | 0.775 | 0.01 |
| Dec-07 | 0.594 | 0.739 | 3.72 |
| Jan-08 | 0.583 | 0.752 | 8.70 |
| Feb-08 | 0.573 | 0.796 | 3.71 |
| Mar-08 | 0.570 | 0.760 | 0.12 |
| Apr-08 | 0.578 | 0.767 | 0.48 |
| May-08 | 0.569 | 0.842 | 0.05 |
| Jun-08 | 0.613 | 0.903 | 0.00 |
| Jul-08 | 0.583 | 0.818 | 0.00 |
| Aug-08 | 0.570 | 0.745 | 0.00 |
| | **AAF = 0.587** | **PDF = 1.185** | **MMF = 0.638** |
| | ADWF = 0.585 | mean PDWF = 0.862 | Max PDWF = 1.185 |
| | AWWF = 0.593 | mean PWWF = 0.815 | Max PWWF = 0.993 |
| Precipitation data collected from onsite rain gauge and provided by SLO County. | | | |

**Table 2-2 Summary of Peaking Factors**

| Flow Condition | Existing Flow (mgd) | Peaking Factor |
|---|---|---|
| Average Annual Flow (AAF) | 0.59 | -- |
| Maximum Monthly Flow (MMF)[2] | 0.64 | 1.09 |
| Peak Daily Flow (PDF) | 1.19 | 2.00 |
| Peak Hourly Flow (PHF) | 1.77 | 3.00 |

[2] The February 2007 Draft Southland WWTF Master Plan reported a MMF peaking factor of 1.34, based on flow records from September 2004 – August 2006. This report has been updated to reflect flow data from Sept 2006 – Aug 2008.

## 2.2 Loading Rates and Solids Production

The loading of organic material and solids in domestic wastewater are important to determine the process capacity of a wastewater treatment facility. The loading can be obtained through monitoring the flow rate, biological oxygen demand ($BOD_5$), and total suspended solids (TSS) of the influent wastewater. Though influent TSS was not regularly monitored, weekly measurements of influent $BOD_5$ at the Southland WWTF began in December 2005. To estimate loading conditions (lbs/day) over the past two years (September 2006 – August 2008), the average $BOD_5$ concentrations were multiplied by the daily flow rates for the month. Table 2-3 summarizes the results and shows the average and maximum values.

**Table 2-3 Influent $BOD_5$ Concentrations and Loading**

| Month-Year | Monthly Average $BOD_5$ (mg/L) | Average Daily Flow (mgd) | Average Daily $BOD_5$ loading (lb/day) |
|---|---|---|---|
| Sep-06 | 320 | 0.570 | 1,521 |
| Oct-06 | 270 | 0.584 | 1,315 |
| Nov-06 | 295 | 0.586 | 1,443 |
| Dec-06 | 273 | 0.597 | 1,357 |
| Jan-07 | 278 | 0.638 | 1,479 |
| Feb-07 | 308 | 0.623 | 1,598 |
| Mar-08 | 250 | 0.599 | 1,246 |
| Apr-07 | 291 | 0.589 | 1,428 |
| May-07 | 310 | 0.580 | 1,500 |
| Jun-07 | 287 | 0.596 | 1,424 |
| Jul-07 | 311 | 0.595 | 1,545 |
| Aug-07 | 285 | 0.572 | 1,361 |
| Sep-07 | 297 | 0.583 | 1,444 |
| Oct-07 | 272 | 0.575 | 1,304 |
| Nov-07 | 393 | 0.578 | 1,892 |
| Dec-07 | 243 | 0.594 | 1,205 |
| Jan-08 | 238 | 0.583 | 1,156 |
| Feb-08 | 262 | 0.573 | 1,251 |
| Mar-08 | 290 | 0.570 | 1,379 |
| Apr-08 | 247 | 0.578 | 1,192 |
| May-08 | 252 | 0.569 | 1,195 |
| Jun-08 | 242 | 0.613 | 1,236 |
| Jul-08 | 237 | 0.583 | 1,150 |
| Aug-08 | 264 | 0.570 | 1,255 |
| AVERAGE | 280 | 0.587 | **1,370** |
| MAXIMUM | | | **1,892** |

As the solids layer, including grit, sludge, and screenings, builds up on the bottom of the ponds, the retention time decreases and the effluent water quality is reduced. Over the past three years, sludge has been removed from each aeration pond and transferred to the sludge drying beds. The WWTF has been operating with all four ponds since August 2008. An estimation of volume and weight of the sludge, and cost to remove it from the beds and dispose of it is included in Section 8.8.

### 2.3 Inflow and Infiltration

The potential impact from inflow and infiltration was investigated. *Infiltration* is the water entering a sewer system and service connections from groundwater, through such means as defective pipes, pipe joints, connections, or manhole walls. Infiltration does not include inflow and is relatively constant over a period of days, weeks, or even months if high groundwater conditions persist near the sewer system. *Inflow* is the water discharged into a sewer system and service connections from such sources as roof and foundation drains, manhole covers, cross connections from storm sewers, and catch basins. Inflow does not include infiltration. Inflow varies rapidly with rainfall conditions, with flows rising and falling within minutes or hours of a severe storm event with significant runoff.

Figure 2-1 compares the total precipitation, as measured by San Luis Obispo County at the WWTF, with the average daily flow for each month between September 2006 and August 2008. Typically, potential influence of infiltration on treatment plant flow rates can be estimated by observing patterns in the total rainfall plotted with the average daily flows for each month. Since the flow meter is considered adequate for long-term average flows, it is considered a reliable source of data for this infiltration study. Based on comparison of rainfall and monthly flows (Figure 2-1) it appears infiltration is not significant.

The impact of inflow can be estimated by the difference between wet weather and dry weather peak daily flows. Although the meter is not considered reliable for short-term peak flow measurements, plant records indicate peak day flows during wet weather months are generally less than dry weather peak day flows, suggesting that inflow is not a significant contribution to wastewater flow.

For these reasons, inflow/infiltration (I/I) is not considered significant in this capacity analysis. The annual average flow (AAF), peak daily flow (PDF), and peak hourly flow (PHF) were used to analyze existing and future capacity and it was assumed these peaks would occur during dry weather periods.

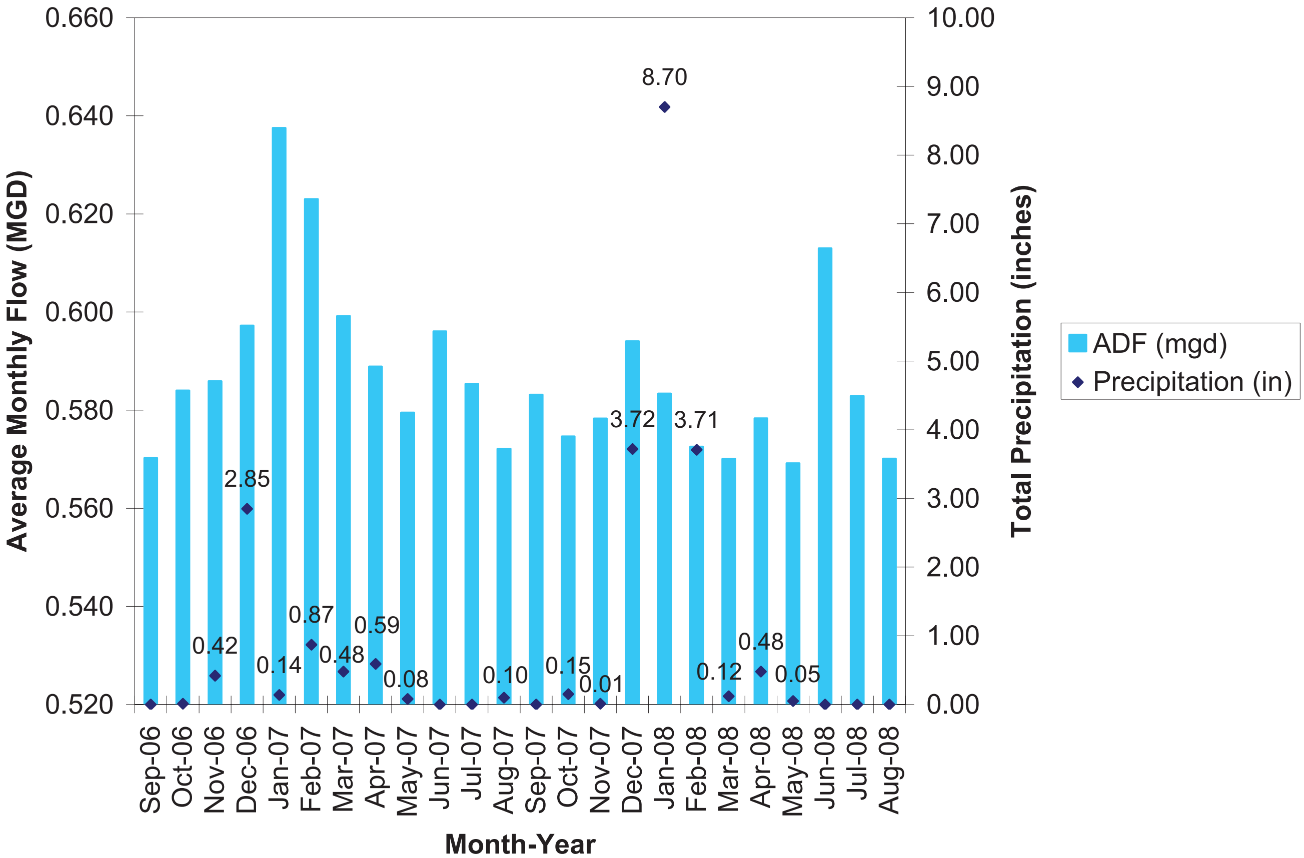



**Figure 2-1 Southland Monthly Average Daily Flows and Total Precipitation (Sept 2006 – Aug 2008)**

# 3.0 PROJECTED LOADS

### 3.1 Projected Future Flow Demands

Plant records from the past 2 years revealed an AAF of 0.59 mgd. This number is comparable to the AAF, 0.63 mgd, found in the NCSD Water and Sewer Master Plan Update (December 2007, Cannon Associates), which determined sewer duty factors based on land-use planning to project sewer flow rates. Based on direction from NCSD, this study used the projected 2030 AAF from Scenario 1 (using existing land use designations) of the Water and Sewer Master Plan and derived intermediate future AAFs assuming a linearized growth between existing and 2030 flow rates. Table 3-1 shows the existing and projected flow rates under the design flow conditions discussed in Section 2.0. The permitted capacity (MMF = 0.9 mgd) could be reached by December 2010 according to this conservatively high growth projection. However, based on current growth rates it may not be reached until 2011 or possibly later. The theoretical BOD reduction capacity of the ponds (discussed in Section 5.0) may allow the plant to operate at higher flows than the permitted capacity. In any event, the plant is operating close to its permitted capacity and the District should begin planning and designing a plant expansion.

**Table 3-1 Projected Flow Rates**

| Flow Condition | Peaking Factor | Existing Flow (mgd) | Projected Flow (mgd)[1] | | | | |
|---|---|---|---|---|---|---|---|
| | | | 2010 | 2015 | 2020 | 2025 | 2030 |
| Average Annual Flow (AAF) | -- | 0.59 | 0.73 | 0.97 | 1.20 | 1.44 | 1.67 |
| Maximum Monthly Flow (MMF)[2] | 1.09 | 0.64 | 0.80 | 1.06 | 1.31 | 1.57 | 1.82 |
| Peak Daily Flow (PDF) | 2.00 | 1.19 | 1.46 | 1.94 | 2.40 | 2.88 | 3.34 |
| Peak Hourly Flow (PHF) | 3.00 | -- | 2.19 | 2.91 | 3.60 | 4.32 | 5.01 |

[1] Projected AAF based on Water and Sewer Master Plan Update (Cannon Assoc., December 2007)
[2] The February 2007 Draft Southland WWTF Master Plan reported a MMF peaking factor of 1.34, based on flow records from September 2004 – August 2006. This report has been updated to reflect flow data from Sept 2006 – Aug 2008.

### 3.2 Projected Future Plant Loading

In evaluating future improvements, both plant $BOD_5$ loading and concentration are important parameters for sizing biological treatment and solids handling processes.

Loading: The projected $BOD_5$ loadings were determined by dividing the existing average annual and maximum monthly $BOD_5$ loadings (see Table 2-3) by the AAF and MMF, respectively. This provides the loadings in terms of pound of $BOD_5$ per million gallons.  These terms were multiplied by the projected flow rates to find the projected $BOD_5$ loadings shown in Table 3-2.

**Table 3-2 Projected $BOD_5$ Loading Rates**

| Year | 2006 | 2010 | 2015 | 2020 | 2025 | 2030 |
|---|---|---|---|---|---|---|
| AAF (mgd) | 0.59 | 0.73 | 0.97 | 1.20 | 1.44 | 1.67 |
| Average Annual $BOD_5$ Loading (lb/day) | 1,370 | 1,700 | 2,250 | 2,790 | 3,340 | 3,880 |
| MMF (mgd) | 0.64 | 0.80 | 1.06 | 1.31 | 1.57 | 1.82 |
| Maximum Monthly $BOD_5$ Loading (lb/day) | 1,892 | 2,350 | 3,130 | 3,870 | 4,640 | 5,380 |

Concentration:  Frequency diagrams are useful for determining design conditions when planning wastewater treatment plant improvements.  Figure 3-1 is the frequency diagram illustrating the monitoring test results for the influent $BOD_5$ for September 2006 through August 2008.  The frequency diagram reveals that 90% of the time the influent $BOD_5$ concentration is less than 360 mg/L.  The use of the 90% frequency value for design $BOD_5$ concentration is recommended for planning and design purposes, because it provides a reasonable level of confidence in the treatment plant performance relative to the actual wastewater conditions.

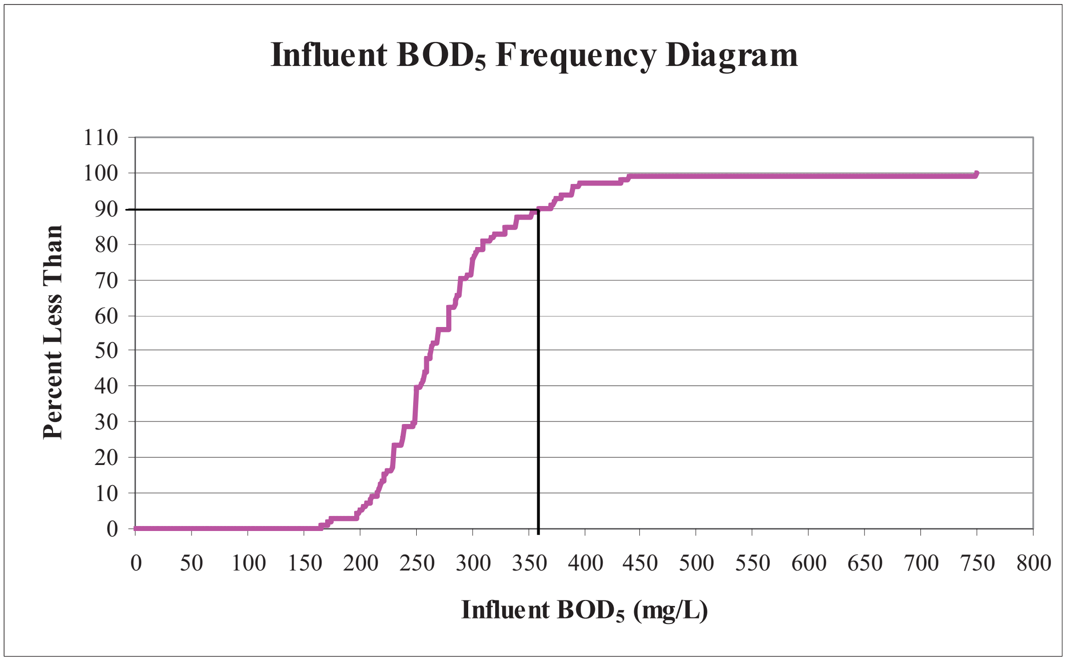



**Figure 3-1 Influent $BOD_5$ Frequency Diagram**

Future sludge production was estimated for a 5-year period at the projected 2030 AAF. Without influent TSS concentration data, the average influent TSS was assumed to be the same as the average influent $BOD_5$ concentration, 265 mg/L, based on similarly sized domestic wastewater plants. Assuming a density of 15%, approximately 2.7 million gallons of sludge is expected to accumulate over 5 years. This is equivalent to 21% of the existing pond system volume. Calculations are included in Appendix B.

# 4.0 EXISTING WASTEWATER TREATMENT FACILITY

## 4.1 Waste Discharge Requirements

The Nipomo CSD operates the Southland WWTF under Waste Discharge Requirements Order No. 95-75 (attached as Appendix A). The permitted capacity of the plant is 900,000 gpd, which is based on the maximum monthly flow. Table 4-1 summarizes the effluent quality requirements for the facility.

**Table 4-1 Effluent Water Quality Requirements**

| Parameter | **Max 30-Day Mean** | **Max Daily** |
|---|---|---|
| Settleable Solids (SS) – mL/L | 0.2 | 0.5 |
| Total Suspended Solids (TSS) – mg/L | 60 | 100 |
| Biochemical Oxygen Demand, 5-day ($BOD_5$) – mg/L | 60 | 100 |
| Dissolved Oxygen - mg/L | Minimum 1.0 | |
| **Additional Limits/Requirements** | | |
| pH | 6.5 -- 8.4 | |
| Receiving Groundwater | Nitrate levels shall not exceed 10 mg/L downstream of the disposal area. Groundwater samples upstream and downstream of the sprayfields shall not demonstrate a statistically significant increase in nitrate, sodium, chloride, and TDS. | |

## 4.2 System Components

The Southland WWTF process flow diagram is included as Figure 4-1 for the existing treatment facilities. The main system components are as follows:

**Headworks:** The purpose of the headworks is to grind large solids in the influent and pump the wastewater into treatment. The Southland WWTF headworks consist of a Parshall flume, two grinders, and two Fairbanks Morse submersible influent pumps.

| **Grinders** | |
|---|---|
| Number of grinders | 2 |
| Type | Vertical inline |
| Horsepower | 10 |
| Reducer | 43:1 |
| Capacity of each, gpm | 2500 |

| **Influent Pumps** | |
|---|---|
| Number of pumps | 2 |
| Capacity of each, gpm | 2331, 2421 |
| Motor horsepower, each | 35 |
| Pump speed, rpm | 1180 |
| TDH, ft | 45 |

| Parshall Flume | |
|---|---|
| Throat width, in | 9 |
| Min flow rate, gpm | 1.2 |
| Max flow rate, gpm | 5,599 |

**Aeration Ponds:** The aeration ponds provide a zone for solids settling and aerobic treatment for the wastewater. The ponds were retrofitted in 1999 with a total of 116 submerged Ramco 12/8 MASP aerators; 46 in each of Ponds 1 and 2, and 12 in each of Ponds 3 and 4. Ponds 3 and 4, the larger two ponds, were originally constructed with floating baffles to isolate a settling zone for additional removal of solids. Due to repeated complications (plugging, etc.), the submerged aerators have been replaced with mechanical aerators. Additionally, the baffles were removed in 2007 to increase aeration volume in Ponds 3 and 4.

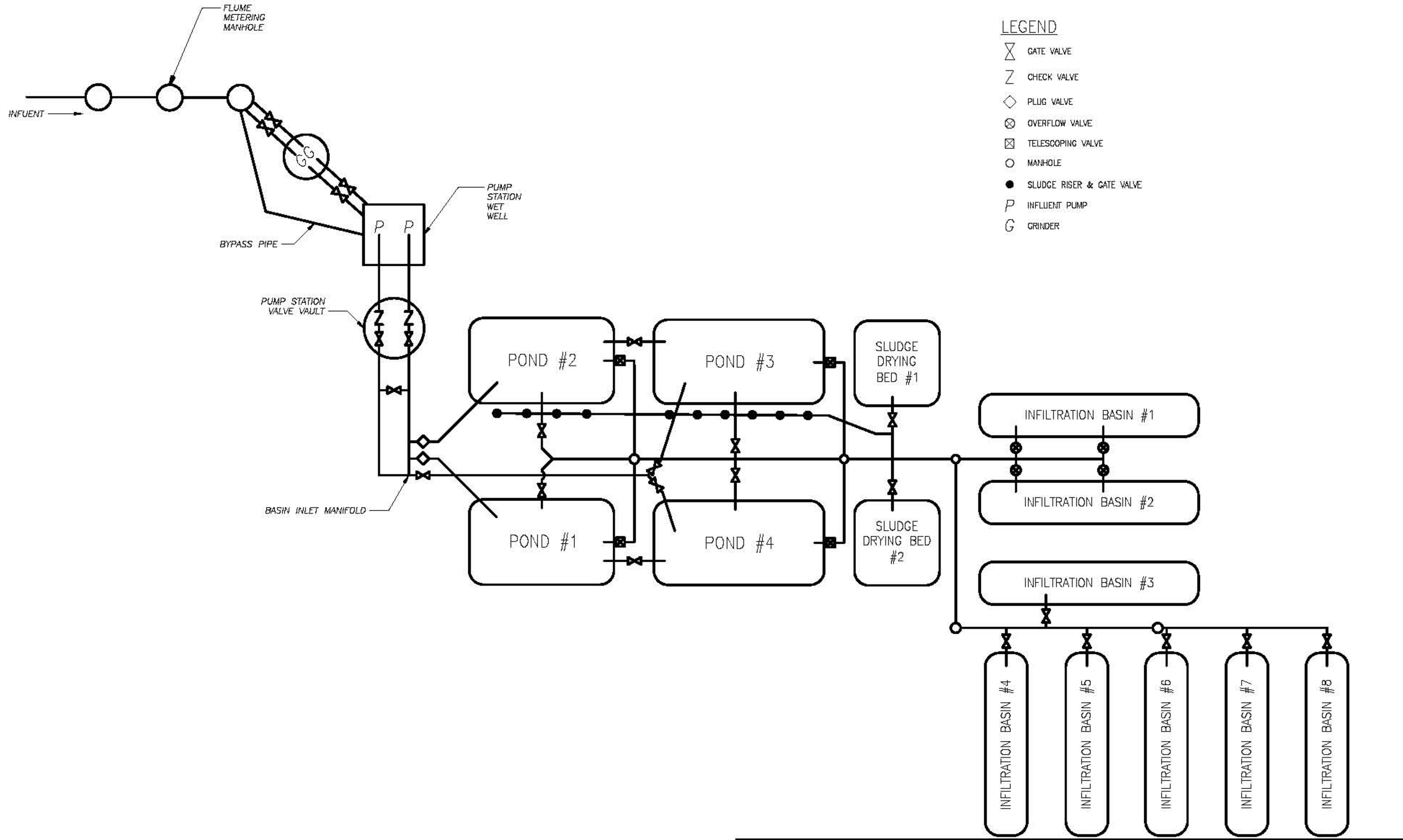
| Aerated Ponds | |
|---|---|
| Number of Ponds | 4 |
| Design Average Flow, mgd | 0.94 |
| Normal Operating Depth, ft | 14 |
| Total Surface Area, acres each | (2) @ 1.09, (2) @ 1.49 |
| Total Liquid Volume, MG | 10.7 |
| Total Aeration Blower Power, hp | 150 |
| Mechanical Aerators, total hp (# of units) | 120 (15) |
| Pond 1 | (2) 5 hP + (3) 10 hP |
| Pond 2 | (2) 5 hP + (2) 10 hP |
| Pond 3 | (2) 5 hP + (1) 10 hP |
| Pond 4 | (3) 10 hP |

**Infiltration Basins:** Further treatment is provided as the aeration pond effluent percolates through the soil beneath the infiltration basins. Several mechanisms work to improve the water quality. Filtration and adsorption through the soil remove suspended solids, bacteria, and viruses. Biodegradation reduces organic material and may have the potential to provide denitrification. The groundwater beneath the infiltration basins is monitored (for boron, sodium, chloride, total nitrogen, total dissolved solids, and sulfate) to ensure that adequate treatment is provided. As described in other studies, the District recently discovered that a mound of plant effluent is growing underneath the plant, supported by an aquitard at 60 to 100 feet below the ground surface.

| Infiltration Basins | |
|---|---|
| Number of Basins | 8 |
| Annual Loading, ft | 73 |
| Total Area, acres | 14.46 |
| Application period, days/basin | 7 |
| Drying Period, days/basin | 49 |

**Sludge Drying Beds:** The sludge drying beds provide an area for evaporation of liquid weight from sludge before disposal. This is important to reduce hauling costs as it is usually based on total weight of the bulk sludge. The beds also provide room for the operators to mix and turn sludge piles as they dry, in order to facilitate more efficient evaporation and thus accelerate the drying process.

| Sludge Drying Beds | |
|---|---|
| Number of Beds | 2 |
| Combined capacity, MG | 1.9 |

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**NCSD SOUTHLAND WWTF MASTER PLAN**

EXISTING PROCESS FLOW DIAGRAM

BEC PROJECT NO.

FIGURE **4-1**

### 4.3 Effluent Quality

Table 4-2 summarizes the WWTF effluent monitoring results for the past 2 years. Results exceeding effluent water quality limits are underlined. One potential cause for violations is insufficient retention time and/or aeration due to one pond being offline for cleaning and maintenance (approx 2004 through July 2008). Evaluation of the previously installed Ramco subsurface aeration system revealed limitations that could result in poor BOD removal. Phased replacement of the subsurface aeration system began in spring of 2004. The baffles in Ponds 3 and 4 were removed in 2007 to increase aerated volume, and all subsurface diffusers were replaced with mechanical surface aerators by July 25, 2008.

During maintenance of the system, District staff discovered an open bypass valve that caused short-circuiting between the primary ponds and the outlet from the secondary ponds, near the effluent sampling station. The valve has since been closed.

The vertical position of outlets in the aeration ponds influences the solids concentration in the effluent. Floating debris on top may interfere with effluent quality; therefore the outlet should be submerged. Also, the outlet should be located above the sludge/solids blanket at the bottom (approximately 6 feet from the water surface). Ideal outlet location is 2 to 3 feet from the top of the water surface where optimal water quality is expected. The outlets from Ponds 1 and 2 were set at 5 feet from the bottom, but the outlet from Pond 1 was raised by approximately 3 feet in 2004. The outlets from Ponds 3 and 4 were designed as floating outlets that adjust with the water to remain at approximately 2 to 3 feet below the water surface. However, the floating outlets were observed by operators to not work properly resulting in the outlets settling to the bottom of the ponds. This likely resulted in solids being decanted directly to the downstream ponds. The District recently replaced the outlets from Ponds 3 and 4 with fixed 90-degree elbows at a depth 2 to 3 feet below the water surface. Effluent monitoring data from August 2008 and on will reflect operations with all four ponds online and the outlets on Ponds 3 and 4 replaced.

Another challenge faced by the operators is the inability to direct effluent from either Pond 3 or Pond 4 to the inlet of the other secondary pond. Therefore, if either primary pond (1 or 2) is removed from service, the other three ponds cannot be operated in series (Ponds 3 and 4 must be operated in parallel).

## Table 4-2 Historical Plant Effluent

| Month/ Year | Flow | | | $BOD_5$ | | | TSS | | | DO | | | SS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Min. (mgd) | Max. (mgd) | Mo. Avg. (mgd) | Min. (mg/L) | Max. (mg/L) | Mo. Avg. (mg/L) | Min. (mg/L) | Max. (mg/L) | Mo. Avg. (mg/L) | Min. (mg/L) | Max. (mg/L) | Mo. Avg. (mg/L) | Mo. Avg. (mg/L) |
| Sep-06 | 0.348 | 0.737 | 0.570 | 21.8 | **150** | **71.2** | 36 | 51 | 44.2 | 3.6 | 4.3 | 3.9 | <0.05 |
| Oct-06 | 0.371 | 0.772 | 0.584 | 54 | 85 | **68.3** | 24 | 59 | 38.3 | **0.8** | 3.6 | 2.5 | <0.05 |
| Nov-06 | 0.38 | 0.785 | 0.586 | 30 | 100 | 57.8 | 20 | 100 | 41.4 | 3.9 | 5.4 | 4.7 | <0.05 |
| Dec-06 | 0.368 | 0.993 | 0.597 | 29 | 68 | 41.8 | 18 | 31 | 25.3 | 3.8 | 6.0 | 4.9 | <0.05 |
| Jan-07 | 0.318 | 0.829 | 0.638 | 20 | 32 | 24.7 | 14 | 40 | 26.8 | 3.6 | 4.3 | 3.9 | <0.05 |
| Feb-07 | 0.326 | 0.835 | 0.623 | 22 | 30 | 26.2 | 4.3 | 33 | 22.8 | 3.5 | 6.1 | 4.8 | <0.05 |
| Mar-07 | 0.361 | 0.917 | 0.599 | 26 | 28.8 | 27.7 | 23 | 40 | 31.5 | 5.0 | 5.6 | 5.3 | <0.05 |
| Apr-07 | 0.398 | 0.772 | 0.589 | 28 | **111** | 51.8 | 30 | 41 | 37.8 | 4.1 | 4.5 | 4.3 | <0.05 |
| May-07 | 0.422 | 0.756 | 0.580 | 25 | **158** | **65.6** | 28 | 56 | 41.4 | 3.9 | 4.5 | 4.1 | <0.05 |
| Jun-07 | 0.287 | 1.185 | 0.596 | 36 | **112** | **73** | 20 | 50 | 33.4 | 3.4 | 4.5 | 4.1 | <0.05 |
| Jul-07 | 0.277 | 1.083 | 0.585 | 21 | 36 | 28.3 | 24 | 36 | 29.3 | 3.9 | 4.4 | 4.2 | <0.05 |
| Aug-07 | 0.284 | 0.85 | 0.572 | 2 | **123** | 53.3 | 18 | 42 | 29 | 3.9 | 4.6 | 4.3 | <0.05 |
| **Avg** | | | **0.593** | | | **49.1** | | | **33.4** | | | **4.3** | **<0.05** |
| **Max** | | **1.185** | | | **158** | **73** | | **100** | **44.2** | | **6.1** | | |
| **Min** | **0.277** | | | **2** | | | **4.3** | | | **0.8** | | | |
| | | | | | | | | | | | | | |
| Sep-07 | 0.067 | 1.184 | 0.583 | 2.15 | **107** | 43.5 | 10 | 28 | 17 | 4.2 | 4.5 | 4.4 | <0.05 |
| Oct-07 | 0.365 | 0.803 | 0.575 | 9.3 | 14.7 | 11.7 | 10 | 13 | 11.7 | 4.2 | 4.7 | 4.4 | <0.05 |
| Nov-07 | 0.319 | 0.775 | 0.578 | 58 | **185** | **92.8** | 17 | 24 | 19.5 | 3.9 | 5.3 | 4.7 | <0.05 |
| Dec-07 | 0.305 | 0.739 | 0.594 | 21.6 | **147** | **94.2** | 16 | 60 | 36.5 | 1.8 | 8.4 | 4.9 | <0.05 |
| Jan-08 | 0.302 | 0.752 | 0.583 | 23 | 37 | 28.9 | 17 | 29 | 24.2 | 7.1 | 8.1 | 7.7 | <0.05 |
| Feb-08 | 0.338 | 0.796 | 0.573 | 8.1 | 26.7 | 13.4 | 18 | 40 | 27 | 7.3 | 8.5 | 8.1 | <0.05 |
| Mar-08 | 0.359 | 0.76 | 0.570 | 12.5 | 37 | 27.9 | 24 | 60 | 40 | 6.9 | 7.9 | 7.2 | <0.05 |
| Apr-08 | 0.428 | 0.767 | 0.578 | 54 | **175** | **108** | 31 | 50 | 43.4 | 5.3 | 6.4 | 5.8 | <0.05 |
| May-08 | 0.376 | 0.842 | 0.569 | 63 | **162** | **103** | 16 | 70 | 36 | 5.8 | 6.6 | 6.1 | <0.05 |
| Jun-08 | 0.288 | 0.903 | 0.613 | 42.9 | **168** | **108** | 31 | 70 | 53.2 | 3.1 | 7.3 | 5.7 | <0.05 |
| Jul-08 | 0.391 | 0.818 | 0.583 | 66.1 | **171** | **121** | 38.4 | 100 | 68 | 5.0 | 6.2 | 5.5 | <0.05 |
| Aug-08 | 0.371 | 0.745 | 0.570 | 30.8 | 56 | 39.2 | 37 | 47 | 73 | 5.5 | 7.0 | 5.9 | <0.05 |
| **Avg** | | | **0.581** | | | **66.0** | | | **37.5** | | | **5.9** | **<0.05** |
| **Max** | | **1.184** | | | **185** | **121** | | **100** | **73** | | **8.5** | | |
| **Min** | **0.067** | | | **2.15** | | | **10** | | | **1.8** | | | |

# 5.0 PLANT PERFORMANCE AND CAPACITY

### 5.1 Ability of Existing System to Meet Current Demand

Hydraulic Capacity of Trunk Main

A hydraulic analysis was performed on the Frontage Road trunk main from Division Street to the WWTF to examine the ability to handle existing flow demands as part of this study (Figure 5-1). Water surface elevations were estimated for both AAF and PHF conditions to develop the hydraulic profile. Figure 5-2 displays the estimated water levels and flow rates for each section, and identifies those that are undersized. The ratio of water depth to pipe diameter (d/D) was used to evaluate the pipe sizes under various flow conditions with the following criteria:

| **Flow Condition** | **Allowable Water Depth (d/D)** |
|---|---|
| AAF | 0.5 |
| PHF | 0.75 |

Flow rates for each section of the Frontage Road trunk main were adjusted for incoming wastewater flows. The percent of total flow in each contributing pipeline was estimated based on the number of dwelling units on the incoming line. There are three incoming pipelines between Division Street and the WWTF: an 8-inch pipe at Southland Street, and two 12-inch pipes at Story Street. An approximate dwelling unit count was performed for each contributing sub-area using an aerial photo taken in 2006. Flow rates were calculated assuming 3.34 people per dwelling unit and an average of 60 gallons per capita per day, based on total measured flow and population. Table 5-1 displays the estimated contributing flow rates for each incoming pipeline.

**Table 5-1 Estimated Contributing Flows to Frontage Road Trunk Main**

| Wastewater Pipeline | Percent of Total Flow | AAF (mgd) | PHF (mgd) |
|---|---|---|---|
| Frontage Rd at WWTF | 100 | 0.60 | 1.8 |
| Southland St | 5 | 0.03 | 0.09 |
| Story St (NE inlet) | 20 | 0.12 | 0.36 |
| Story St (NW inlet) | 10 | 0.06 | 0.18 |

DWG: W:\Nipomo CSD (19996)\19996.38 Southland WWTF Master Plan Update\CADD\Figures\FIGURE 5-1.dwg USER: ShieldsE
DATE: Nov 25, 2008 3:32pm XREFS: IMAGES: Frontage Road Trunk Site Plan.tif

LEGEND

PROPERTY BOUNDARY

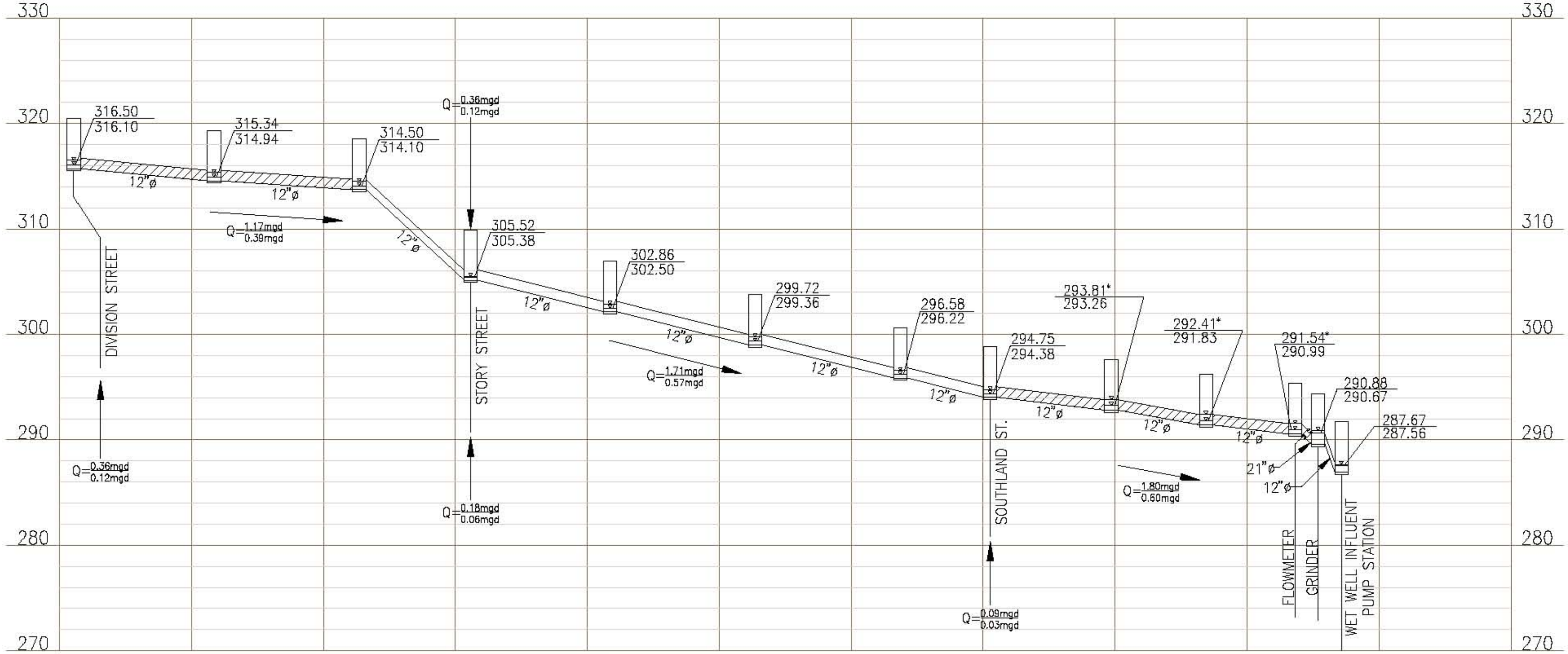
SEWER LATERAL

STUDY AREA


N

NOT TO SCALE

| FIGURE | NCSD SOUTHLAND WWTF MASTER PLAN | BEC PROJECT NO. | BOYLE ENGINEERING CORPORATION |
|---|---|---|---|
| 5-1 | FRONTAGE ROAD TRUNK MAIN STUDY AREA | 19996.17 | 1194 Pacific St., Suite 204 San Luis Obispo, CA. 93401 Tel. 805-542-9840 Fax 805-542-9990 WWW.BOYLEENGINEERING.COM |




*DEPTH IS 82–100% OF PIPE DIAMETER (SHOWN AS 100%)

LEGEND

PHF / AAF

d/D>0.5 DURING AAF AND >0.75 PHF

d/D>0.75 DURING PHF

d/D>0.5 DURING AAF

Q= PHF / AAF

SCALE:
HORIZ. 1"=400'
VERT. 1"=10'

| | | | |
|---|---|---|---|
| BOYLE ENGINEERING CORPORATION<br>1194 Pacific St., Suite 204 San Luis Obispo, CA. 93401 Tel. 805-542-9840 Fax 805-542-9990<br>WWW.BOYLEENGINEERING.COM | NCSD SOUTHLAND WWTF MASTER PLAN<br>FRONTAGE ROAD TRUNK MAIN HYDRAULIC PROFILE FOR EXISTING DEMANDS | BEC PROJECT NO.<br>19998.17 | FIGURE<br>5-2 |

Influent Pump Station

The influent pump station was examined for hydraulic capacity. Two Fairbanks-Morse submersible pumps were installed in 2000. They are rated at approximately 2300 gpm each, providing enough capacity to handle the current peak hour flow of approximately 1230 gpm with one pump as a backup. System and pump curves were generated which confirmed this for the specific system conditions (Figure 5-3).

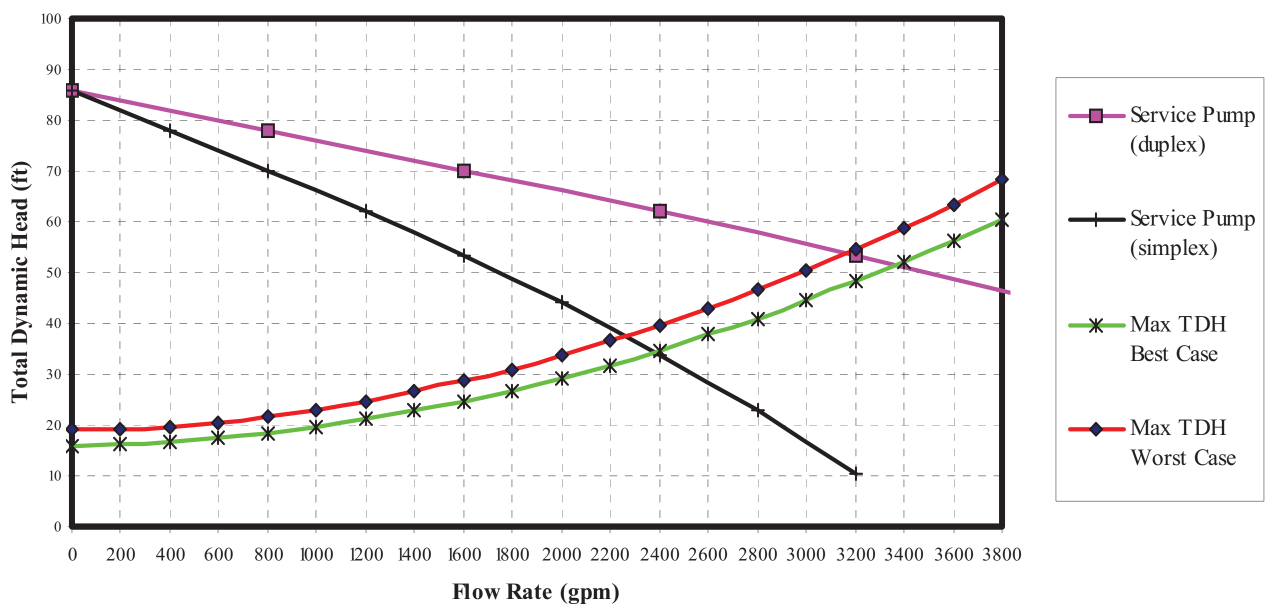



**Figure 5-3 Composite Service Pump Curve and System Curve**

It is important that influent wetwells are sized with the correct volume and controls for optimized pump station operation. Wet wells should be large enough to prevent rapid pump cycling, which wears the motor and electronics, and small enough to reduce residence time and minimize odors and settling/accumulation of solids. The influent wet well is 8-feet in diameter. Analysis indicates that the wet well is undersized. The following equation is used to determine the recommended storage volume for a wet well[1]:

$$V = \frac{Tq}{4}$$

Where, T is the allowable minimum cycle time between starts, q is the rated capacity of a single pump, and V is the active volume of the wet well. The active volume is defined as the amount of storage available between pump cycles. To protect the pumps, the recommended minimum cycle time is 10 minutes per pump. Under this condition, the desired wet well active volume for the pump station is 2875 gallons, or 370 ft$^3$. With 3.7 feet

[1] Sanks, Robert L. *Pumping Station Design,* 2$^{nd}$ Edition. Butterworth-Heinemann: (1998), 370.

between the levels when the lead pump turns on and off, the current active volume is 186 $ft^3$, half the volume recommended for existing conditions.

Treatment Capacity

The ability to treat the current influent wastewater was evaluated using various historic flow and temperature conditions. First-order rate kinetics were used to estimate $BOD_5$ degradation in the aeration ponds. The analysis showed that the current treatment system is able to handle existing conditions and treat incoming wastewater to acceptable levels provided adequate aeration is accomplished and transfer of clarified effluent between the primary ponds to the secondary ponds is withdrawn from proper level above sludge blanket and below pond surface. The $90^{th}$ percentile $BOD_5$ (360 mg/L) was applied and the analyses were run under two assumed configurations: four ponds in series and two ponds in series (two parallel flow trains). Both configurations were examined under different combinations of temperature and flow conditions (summer and winter temperatures, and high, low, and maximum month daily flow rates).

Analyses show the configuration using four ponds in series theoretically performs better than the series of two ponds, providing a 92 – 96% reduction in $BOD_5$ concentration (from 360 mg/L to 15 – 29 mg/L). The two ponds in series configuration also shows the ability for adequate levels of treatment, providing effluent $BOD_5$ concentrations between 30 and 48 mg/L, or an 87 – 92% reduction of $BOD_5$. However, several other factors can hinder the ponds' capability to reduce BOD. Extended detention times can result in poorly settled sludge in the final aeration steps. This sludge may be suspended in the ponds and may cause an increase in effluent BOD. For this reason, we recommend using the parallel model to determine if the ponds provide sufficient retention time as opposed to the ponds in series. The modeling cannot provide an accurate prediction of effluent $BOD_5$ concentrations, but is useful in evaluating retention time and determining appropriate pond volumes. Table 5-2 summarizes the results of the analysis and indicates that sufficient retention time and pond volume are available under existing conditions. Calculations are included in Appendix B.

**Table 5-2 Modeled Effluent Quality under Existing Flow Conditions**

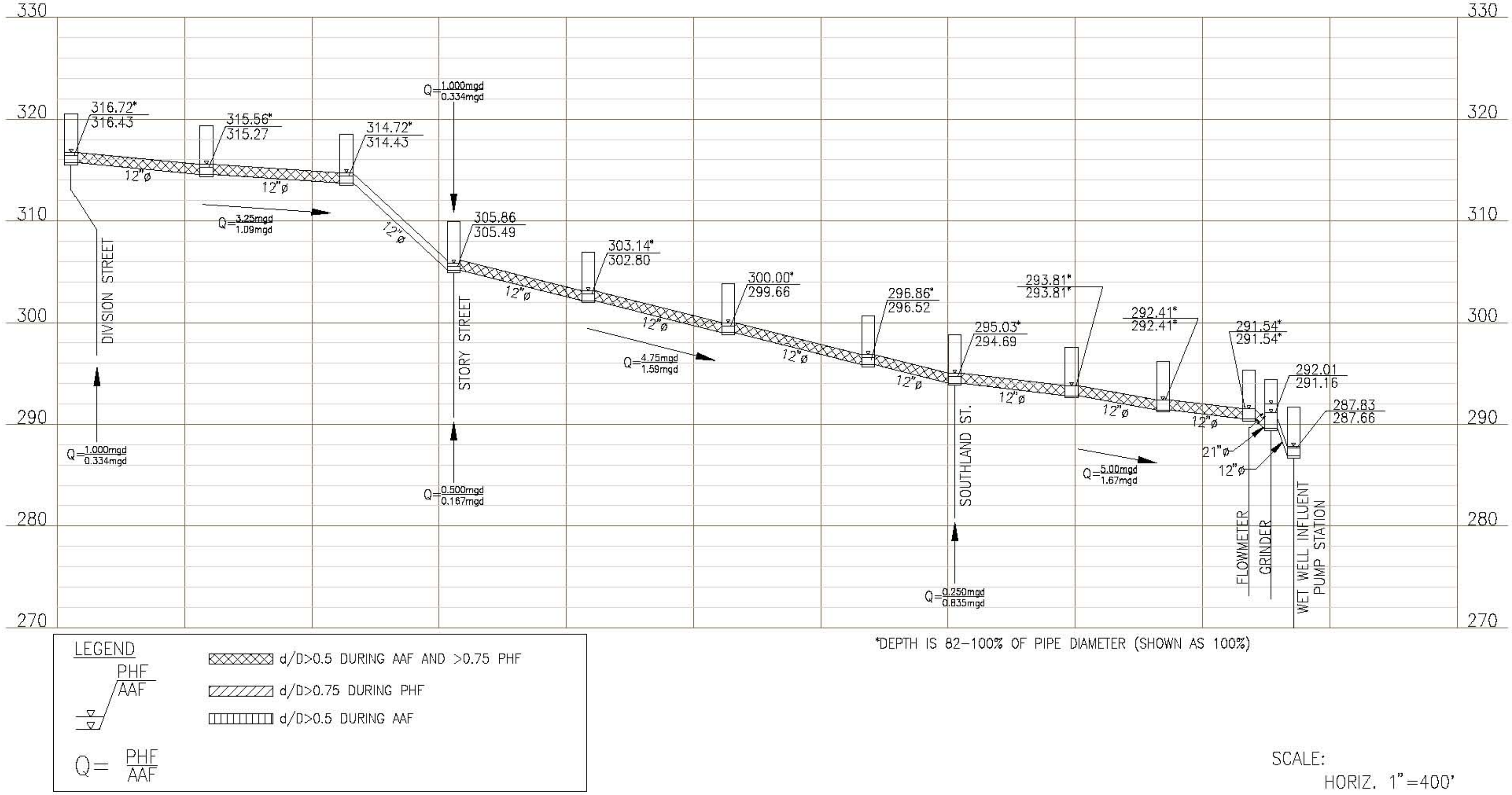
| | Temperature (T) and Flow (Q) Conditions | | |
|---|---|---|---|
| | Low T, Low Q | High T, High Q | High T, MMF |
| 4 Ponds in Series [$BOD_5$] (mg/L) | 28 | 15 | 29 |
| 2 Parallel Trains of 2 Ponds [$BOD_5$] (mg/L) | 47 | 30 | 48 |
| WDR Effluent $BOD_5$ Limits: Daily = 100 mg/L; Monthly = 60 mg/L | | | |

## 5.2 Ability of Existing System to Meet Future Demand

Frontage Road Trunk Main

The Frontage Road Trunk Main from Division Street to the WWTF was examined to determine the ability to handle future flow demands. The water surface elevations were estimated using the projected AAF and PHF to form the hydraulic profile, included as Figure 5-4. Flow rates were adjusted for incoming wastewater pipelines, using the same method as previously discussed.

The same d/D criteria as for the existing hydraulic capacity analysis were used to identify undersized pipe. The entire stretch of 12-inch pipeline examined was found to be undersized for both AAF and PHF, except one section immediately above the Story Street intersection where the slope is 2.1%, nearly 3.5 times that of the next greatest slope in the study reach. If the other pipes are replaced, it is recommended that this pipe be replaced as well.




SCALE:
HORIZ. 1"=400'
VERT. 1"=10'

| | | | |
|---|---|---|---|
| BOYLE ENGINEERING CORPORATION<br>1194 Pacific St., Suite 204 San Luis Obispo, CA. 93401 Tel. 805-542-9840 Fax 805-542-9990<br>WWW.BOYLEENGINEERING.COM | NCSD SOUTHLAND WWTF MASTER PLAN<br>FRONTAGE ROAD TRUNK MAIN<br>HYDRAULIC PROFILE FOR 2030 PROJECTED DEMANDS | BEC PROJECT NO.<br>19998.17 | FIGURE<br>5-4 |

WWTF Hydraulic Capacity

Available record drawings were used to develop a hydraulic grade line through the wastewater treatment facility for future peak day flow. Overflow weirs and outlet control devices dictate the water levels in the secondary ponds. A hydraulic analysis was performed through the pipes and valves connecting the primary to the secondary ponds to determine the water levels in the primary ponds. Hydraulically, the current pond system has sufficient capacity to meet future flow demands. Treatment capacity is addressed in the subsequent section. Figure 5-5 displays the hydraulic grade line through the treatment facility.

Influent Pump Station

The influent pump station was analyzed for future capacity. Based on the pump and system curves, included as Figure 5-3 above, the pumps are undersized to handle the year 2030 PHF of 3500 gpm. The duplex pump curve indicates that the two existing pumps pumping together will be capable of delivering the flow. However, an upgrade is required to maintain 100% redundancy in the future.

Since the desired wet well volume is dependent on pump capacity, the wet well volume should be increased when the pumps are replaced with larger pumps. Assuming two 3500-gpm pumps are installed to meet PHF, the future required active wet well volume should be 585 $ft^3$ to maintain a 10-minute cycle time per pump during PHF. It should be noted that the analysis is based on the existing system. If changes are made to the headworks the analysis will need to be revisited to properly size influent pumps and wet well. The addition of screening and grit removal systems will add to system head loss, potentially requiring additional pump capacity.

Treatment Capacity

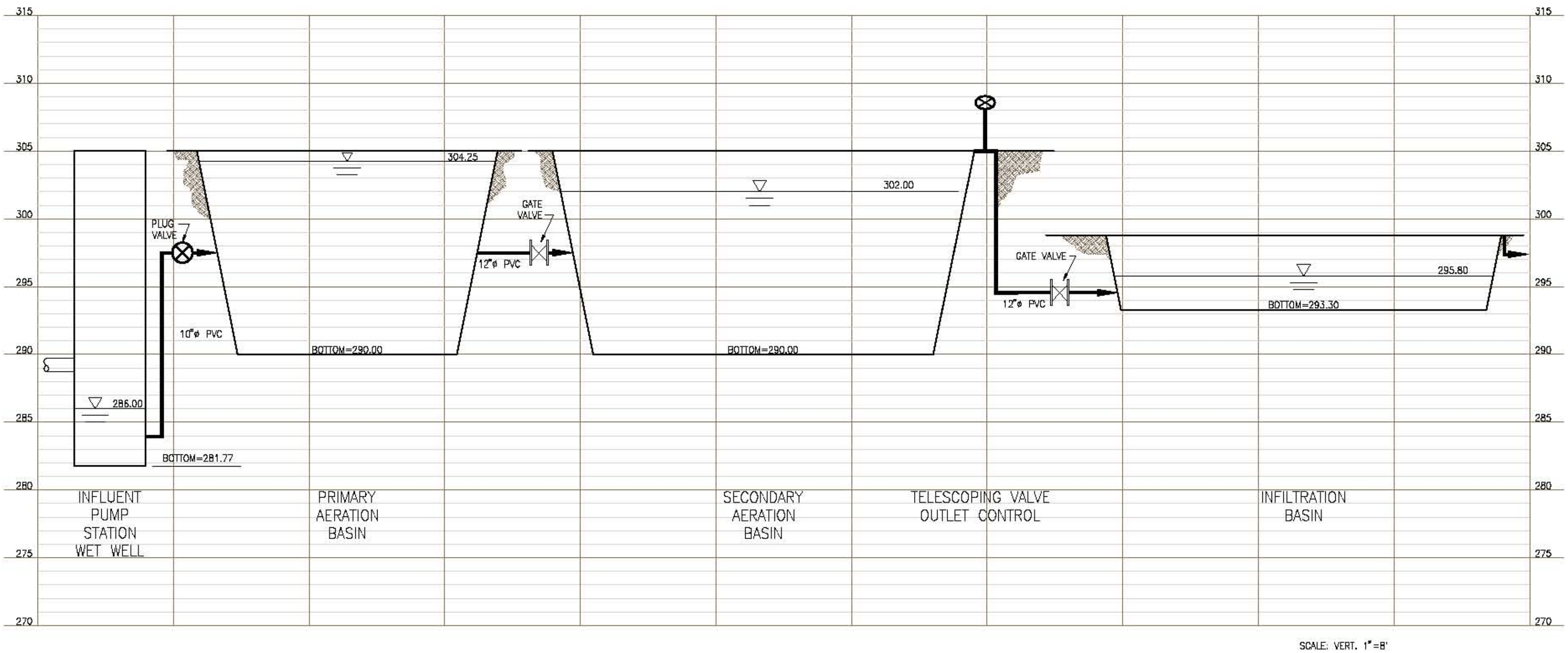
The ability of the existing system to treat future wastewater flow was evaluated using projected hydraulic demands for applicable 2030 flow rates (PDF, AAF, and MMF), the $90^{th}$ percentile $BOD_5$ concentration (360 mg/L), and two boundary temperature conditions (summer and winter). Two configurations were examined: four ponds in series, and two parallel trains with two ponds in each train. First-order rate kinetics were applied to predict $BOD_5$ degradation. Table 5-3 summarizes the results of the analysis. Neither configuration appears to provide sufficient treatment under any boundary flow condition. Full calculations are included in Appendix B.

**Table 5-3 Treatment Capacity of Existing System under Future Flow Conditions**

| | Temperature (T) and Flow (Q) Conditions | | |
|---|---|---|---|
| | **Low T, Low Q** | **High T, High Q** | **High T, MMF** |
| 4 Ponds in Series [$BOD_5$] (mg/L) | 124 | 155 | 108 |
| 2 Parallel Trains of 2 Ponds [$BOD_5$] (mg/L) | 139 | 167 | 125 |
| WDR Effluent $BOD_5$ Limits: Daily = 100 mg/L; Monthly = 60 mg/L | | | |

If the ponds are operated in two parallel trains of two, the treatment modeling indicates that permitted $BOD_5$ effluent limit is expected to be reached by 2011 during high temperature, high flow conditions. If the ponds are run in series, the permitted $BOD_5$ limit may be reached in 2015 but sludge settleability becomes a concern in series operation, as discussed elsewhere in this study.

Regardless, the District should begin planning and design of a wastewater treatment plant upgrade as soon as possible since the facility is nearing the permitted hydraulic capacity (see Section 3.0).

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| | | | |
|---|---|---|---|
| **NCSD SOUTHLAND WWTF MASTER PLAN** | BEC PROJECT NO. | FIGURE |
| TREATMENT PLANT HYDRAULIC PROFILE FOR 2030 PEAK DAY FLOW | [illegible] | **5-5** |

# 6.0 WATER QUALITY GOALS

### 6.1 Recycled Water Usage

Currently, the Southland Wastewater Treatment Facility (WWTF) discharges to eight infiltration basins and eventually to groundwater. The selection of treatment processes, associated plant improvements, pumping stations, pipelines, and storage facilities depend on the end user or final destination of the wastewater. Depending on the usage option chosen, different regulatory requirements will be enforced; also, the WDRs will need to be revised for recycled water use. The usage options considered in this section are as follows: 1) Unrestricted Urban Usage, 2) Groundwater Recharge, and 3) Maintain Current Discharge Practices. Depending on the usage option chosen, the WWTF may need to meet recycled wastewater regulations (i.e. California Code of Regulations (CCR) Title 22).

### 6.2 Option 1 - Unrestricted Urban Reuse (Disinfected Tertiary Recycled Water)

Regulatory Requirements

The California Code of Regulations (CCR) Title 22, Division 4, Chapter 3, Sections 60301 through 60355 are used to regulate recycled wastewater and are administered jointly by California Department of Health Services (CDHS) and RWQCB.

Disinfected tertiary recycled wastewater requires a level of treatment that meets the most stringent requirements for all uses allowed under the Title 22 criteria. Potential users include farmlands, parks and playgrounds, schoolyards, unrestricted access golf courses, roadway landscaping, and residential and commercial landscaping. This study focuses on landscaping application for parks. Owners of these facilities, CDHS, RWQCB, County, and possibly local authorities will be involved in wastewater reuse contracts and permitting. The Waste Discharge Requirements for the WWTF would need to be revised to allow reuse of plant effluent for unrestricted urban use. Disinfected tertiary treatment requires oxidation, coagulation[2], filtration and disinfection. These treatment stages will need to be added to the WWTP as part of the upgrades if this reuse option is pursued. According to Title 22 requirements, the median total coliform limit in reclaimed water is 2.2 MPN/100mL, and the maximum total coliform standard is 23 MPN/100mL. The median total coliform number is determined from samples of bacteria collected from the last 7-days of analysis. The maximum total coliform should not be exceeded in one sample over 30 consecutive days.

Contracts with end users are typically required for guaranteeing a demand for treated wastewater. In addition, facilities and appurtenances needed for recycling include transmission pipelines, pump stations, storage reservoirs, and property or easements for locating these facilities.

Water Quality Objectives

Water quality objectives for unrestricted urban use are primarily driven by public safety and suitability for application. Safety assurances are written into Title 22 requirements through standards for effluent coliform concentrations and usage restrictions, such as pipeline distance from potable water pipelines, proximity to groundwater, and restrictions near eating facilities and drinking fountains.

There have been multiple studies to determine constituents of concern in reclaimed water used for irrigation. Suitability of water for irrigation is directly related to the concentration and kind of chemical constituents present. The water constituents that may affect recycled water suitability for irrigation of grasses and ornamental plants include electrical conductivity of the irrigation water ($EC_W$), sodium adsorption ratio (SAR), bicarbonates, chlorides, and boron. General irrigation water quality guidelines are shown on Table 6-1. A summary of the effluent[3] (treated wastewater) quality from the Nipomo Southland Wastewater Treatment Facility (WWTF) is presented in Table 6-2. Crop specific tolerance limits are presented in Table 6-3.

Electric Conductivity/TDS

Salinity can be indirectly measured by electrical conductivity. The units of conductance are typically decisiemens per meter (dS/m), which is equivalent to millimhos per centimeter (mmhos/cm). Multiple devices and protocols exist for the monitoring/measuring of electrical conductivity, including in-office and in-field measurements.

$EC_w$ is the electrical conductivity of the irrigation water. It is a measure of the total salt content of the irrigation water and is used to quantify its salinity. Since the EC of the treatment plant effluent is not currently monitored, no conclusions can be drawn as to the suitability of the effluent's salinity for irrigation. If the effluent salinity (measured as EC) is within the water quality guidelines summarized in Table 6-1 for irrigation water salinity (measured as $EC_w$), there should be no EC associated effluent reuse restrictions. However, if the effluent salinity tends toward the "Increasing Problems" or "Severe Problems" range, intensive irrigation management may be required in order to control soil salinity levels. Adequate rainfall will assist the salt leaching process and help to mitigate the accumulation of soluble salts in the soil profile.

---

[2] Coagulation is not typically required if membrane filtration is used and/or turbidity requirements are met.

[3] Effluent is currently secondary

## Table 6-1 Guidelines for Interpretation of Water Quality for Irrigation

| Problem and Related Constituent | References | Water Quality Guidelines<br>No Problem | <br>Increasing Problems | <br>Severe Problems |
|---|---|---|---|---|
| Salinity[1] | | | | |
| $EC_w$ of irrigation water (mmhos/cm) | 1,2 | <0.75 | 0.75-3.0 | >3.0 |
| TDS (mg/l) or (ppm) | 2 | <450 | 450-2000 | >2000 |
| Permeability | | | | |
| $EC_w$ of irrigation water (mmhos/cm) | 1 | >0.5 | <0.5 | <0.2 |
| adj.SAR[2] | 1 | <6.0 | 6.0-9.0 | >9.0 |
| Specific ion toxicity from root absorption[3] | | | | |
| Sodium (evaluated by adj.SAR) | 1,2 | <3.0 | 3.0-9.0 | >9.0[4] |
| Chloride (meq/l) | 1 | <4 | 4.0-10.0 | >10 |
| Chloride (mg/l) | 1,2 | <142 | 142-355 | >355 |
| Boron (mg/l) | 1 | <0.5 | 0.5-2.0 | 2.0-10.0 |
| Specific ion toxicity from foliar absorption[5] (sprinkler irrigation) | | | | |
| Sodium (meq/l) | 1 | <3.0 | >3.0 | -- |
| Sodium (mg/l) | 1,2 | <69 | >69 | -- |
| Chloride (meq/l) | 1 | <3.0 | >3.0 | -- |
| Chloride (mg/l) | 1 | <106 | >106 | -- |
| Miscellaneous[6] | | | | |
| Total Nitrogen ($NH_4$-N and $NO_3$-N) (mg/l) for sensitive crops | 1,2 | <5 | 5-30 | >30 |
| (The following apply only for irrigation by overhead sprinklers) | | | | |
| Bicarbonate ($HCO_3$) (meq/l) | 1 | 1.5 | 1.5-8.5 | >8.5 |
| Bicarbonate ($HCO_3$) (mg/l) | 1,2 | <90 | 90-520 | >520 |
| Residual Chlorine (mg/l) | 2 | <1.0 | 1.0-5.0 | >5.0 |
| PH | 1,2 | Normal range = 6.5-8.4 | | |

[1] Assumes water for crop plus needed water for leaching requirement will be applied. Crops vary in tolerance to salinity

[2] adj.SAR (adjusted sodium absorption ratio) is calculated form a modified equation developed by U.S. Salinity Laboratory to include added effects of precipitation or dissolution of calcium in soils and related to $CO_3$ + $HCO_3$ concentrations. Permeability problems, related to low EC or high adj.SAR of water, can be reduced if necessary by adding gypsum.

[3] Most tree crops and woody ornamentals are sensitive to sodium and chloride. Most annual crops are not sensitive.

[4] Shrinking-swelling type soils (montmorillonite type clay minerals); higher values apply for others.

[5] Leaf areas wet by sprinklers may show a leaf burn due to sodium or chloride absorption under low-humidity / high-evaporation conditions. (Evaporation increases ion concentration in water films on leaves between rotations of sprinkler heads.)

[6] Excess N may affect production of quality of certain crops, i.e., sugar beets, citrus, avocados, apricots, and grapes. $HCO_3$ with overhead sprinkler irrigation may cause a white carbonate deposit to form on fruit and leaves.

Reference 1: Ayers, Robert S., Quality of Water for Irrigation, Journal of the Irrigation and Drainage Division, ASCE, June 1977. (Table 1, page 136)

Reference 2: Irrigation with Reclaimed Municipal Wastewater – A Guidance Manual, California State Water Resources Control Board, Report Number 84-1 WR, July 1984. (Table 3-4, page 3-11)

Note: Interpretations are based on possible effects of constituents on crops or soils or both. Guidelines are flexible and should be modified when warranted by local experience or special conditions of crop, soil, and method of irrigation.

**Table 6-2 Summary of Effluent Quality from NCSD Southland WWTF**

| Constituent | Units | Range of Results[1] | Comparison to Table 6-1 Guidelines |
|---|---|---|---|
| Bicarbonate | mg/l or ppm | -- | -- |
| Boron | mg/l | -- | -- |
| Chloride | mg/l | 208 – 234 | Increasing problems for root and foliar absorption[2] |
| Total Nitrogen | mg/l | 28 – 46 | Increasing to severe problem for sensitive crops[2] |
| pH | -- | 7.4 – 7.7 | Within normal range |
| TDS | mg/l | 980 – 1180 | Within increasing problems range[2] |
| EC | dS/m or mmhos/cm | -- | -- |
| Sodium | mg/l | 184 – 209 | Increasing problems for foliar absorption[2] |
| SAR | -- | -- | -- |
| $SAR_{adjusted}$ | -- | -- | -- |

-- Indicates constituents are not currently monitored
[1]Effluent quality data is based on Discharger Self Monitoring Reports from July 2004 through August 2006.
[2]Crops vary in tolerance to the constituents above in Table 6-2. Table 6-1 summarizes general irrigation water guidelines as published by the quoted references. Care should be taken in interpretation and application of this data.

Sodium Adsorption Ratio

The sodium adsorption ratio (SAR) is the most reliable index of sodium hazard to crops and soils. A moderately high SAR will not generally result in a toxic effect to most plants. However, some crops are sensitive to excess sodium. Foliar toxicity may exist due to elevated sodium concentrations: however, it is a site/crop-specific phenomenon.

A reduction in soil permeability is a major problem that occurs with high-sodium irrigation water. Applying water with an SAR below 6 does not usually result in permeability problems. If the SAR is between 6 and 9, permeability problems can occur on fine-textured soils. An SAR above 9 will likely result in permeability problems on all mineral soils except course, sandy soils.

Bicarbonates and Adjusted Sodium Adsorption Ratio ($SAR_{adj}$)

Bicarbonates in irrigation water applied to the soil will precipitate calcium from the cation exchange complex as relatively insoluble calcium carbonate. As exchangeable calcium is lost from the soil, the relative proportion of sodium is increased with a corresponding increase in the sodium hazard (SAR). Bicarbonates in the irrigation water contribute to the overall salinity, but, more importantly, they may result in a previously calcium-dominant soil becoming sodium dominant by precipitating the exchangeable calcium, which, in turn, will reduce soil permeability.

A measure of the bicarbonate hazard in irrigation water can be expressed as the adjusted SAR. See Table 6-1. The adjusted SAR takes into account the concentration of bicarbonates in irrigation water in relation to their effect on potential increases in soil SAR. When the adjusted SAR is less than 6, soil permeability problems generally do not occur. If the adjusted SAR is between 6 to 9, permeability problems can occur on fine-textured soil. An adjusted SAR above 9 will likely result in permeability problems in mineral soils except course, sandy soils, where adverse impacts to soil permeability are not a major concern. Periodic soil treatment (i.e. deep ripping or disking) or water treatment may be required to maintain favorable water infiltration characteristics in project soils.

Bicarbonates in irrigation water may also cause potential problems in micro-irrigation systems as a result of lime precipitation, which can cause emitter plugging. These potential problems are accentuated in alkaline irrigation water.

Chlorides

Chlorides are necessary for plant growth in relatively small amounts. However, high concentrations of chlorides can inhibit growth and result in toxicity to foliage if applied by sprinkler irrigation. Chlorides in irrigation water are toxic to some plant species. The tolerances of select herbaceous crops and ornamentals to chloride are shown on Table 6-3. The chloride concentration of the treatment plant effluent (see Table 6-2) is within the range of increasing problems for root and foliar absorption when compared to the guidelines in Table 6-1. If a sprinkler wets the leaf areas, foliage toxicity (leaf burn) problems may also be apparent as a result of the effluent having a slightly higher-than-desired chloride concentration level (Table 6-2).

## Table 6-3 Crop Specific Tolerance Limits for Irrigation Water Quality

### Herbaceous Crops & Ornamentals

| Crop | Constituent Limits: Salt tolerance: In Sat. Soil Extracts $EC_e$[1] (dS/m) or (mmhos/cm) | Salt tolerance: In Irrigation Water $EC_w$[2] (dS/m) or (mmhos/cm) | Chloride tolerance ($Cl^-$): In Sat. Soil Extracts[3] (mol/m^3) | Chloride tolerance ($Cl^-$): In Sat. Soil Extracts[4] (mg/l) | Boron tolerance: In Soil Water[5] (mg/l) |
|---|---|---|---|---|---|
| **Herbaceous Crops (grasses,grain,forage):** | **Threshold values** | | **Threshold values** | | **Threshold values** |
| Alfalfa | 2.0 | 1.3 | 20 | 700 | 4.0 - 6.0 |
| Barley (forage) | 6.0 | 4.0 | 60 | 2100 | 3.4 |
| Bermuda Grass | 6.9 | 4.6 | 70 | 2450 | -- |
| Fescue Tall Grass | 3.9 | 2.6 | 40 | 1400 | -- |
| Sorghum | 6.8 | 4.5 | 70 | 2450 | 7.4 |
| **Ornamental shrubs and trees:** | **Max. Permissible Values** | | | | **Threshold values** |
| Bougainvillea | > 8 | 5.3 | -- | -- | -- |
| European Fan Palm | 6 - 8 | 4 - 5.3 | -- | -- | -- |
| Southern Magnolia | 4 - 6 | 2.7 - 4 | -- | -- | -- |
| Strawberry Tree | 3 - 4 | 2 - 2.7 | -- | -- | -- |
| Oleander | 6 - 8 | 4 - 5.3 | -- | -- | 2.0 - 4.0 |
| Japanese Boxwood | 4 - 6 | 2.7 - 4 | -- | -- | 2.0 - 4.0 |
| Juniper | -- | -- | -- | -- | <0.5 |

**--** Indicates data not available

[1] $EC_e$ data adapted from Tables 13.1a, 13.1b, & 13.3 of reference #1 below:

[2] $EC_w$ is the electrical conductivity of the irrigation water. Irrigation water salinities exceeding the stated threshold or maximum permissible values may cause leaf burn, loss of leaves, and/or excessive stunting. $EC_w$ is approximated from the $EC_e$ as follows:

$EC_e/1.5 = EC_w$

This relationship should be valid for normal irrigation practices.

[3] $Cl^-$ tolerance data adapted from Table 13.6 of Reference #1 below:

[4] To convert $Cl^-$ concentrations to mg/l, multiply threshold values by 35. $Cl^-$ concentrations in saturated soil extracts sampled in the rootzone.

[5] Boron tolerance data adapted from Tables 13.7 & 13.9 of Reference #1 below:

**Reference 1:** ASCE Manuals and Reports on Engineering Practice No. 71, Agricultural Salinity Assessment and Management, 1996 corrected edition

Boron

Boron in irrigation water does not have an effect on soil physical conditions, but in high concentrations it can have a toxic effect on some plants. The tolerance of some crops to boron is shown in Table 6-3. As indicated in Table 6-2, boron is currently not monitored, as it is not a regulated contaminant in the treatment plant's WDR.

Recommendations For Monitoring

In order to fully evaluate the suitability of the wastewater treatment plant effluent for unrestricted use in urban applications, the following constituents/parameters should be monitored, recorded, and evaluated on a quarterly or semiannual basis.

- Effluent Electrical conductivity ($EC_w$) as previously discussed in this report
- SAR and $SAR_{adj}$ to evaluate the water sodium hazard
- Boron to evaluate potential toxicity to plants
- Fecal coliform

This data is invaluable in fully understanding, evaluating, and identifying potential soil management and crop production problems that can arise as a result of irrigating with the effluent in question.

### 6.3 Option 2 - Groundwater Recharge

In December 1994, CDHS prepared a draft document to regulate groundwater recharge reuse projects (GRRP) called the Groundwater Recharge Reuse Draft Regulations. This document proposed guidelines for maximum percentage of recycled water, retention time, horizontal distance to extraction, and maximum contaminant levels (MCLs). Though the regulations are still in draft form and the ultimately adopted criteria are unknown, the document provides useful guidelines for potential groundwater recharge reuse projects. CDHS, RWQCB, local agencies, and landowners will be involved if this usage option is pursued.

The general requirements of the draft regulations indicate that for each GRRP the wastewater management agency shall administer an industrial pretreatment and pollutant source control program. Contaminants for the program will be specified by CDHS based on a review of an engineering report (discussed below) and other available data. The source control program shall include:

1) An assessment of the fate of specified contaminants,
2) A source investigation and monitoring program focused on specified contaminants,

3) An outreach program to the public within service area to manage and minimize discharge of compounds of concern, and
4) A program for maintaining an inventory of compounds discharged into the wastewater collection system.

Upon proposal of a GRRP an engineering report is required for CDHS and RWQCB that includes a comprehensive investigation and evaluation of the GRRP, characterization of the recycled and diluent water quality, evaluation of the impacts on the existing potential uses of the impacted groundwater basin, the proposed means for achieving compliance, and an operations plan. Prior to the operation of a new GRRP, an approved plan shall be in place for providing an alternative source of domestic water supply or an approved treatment if drinking water sources are determined to be unsafe as a result of the GRRP. CDHS will conduct public hearings for the proposed GRRP prior to making recommendations to the RWQCB regarding permitting.

Recycled water used for groundwater recharge must meet the definition of filtered, disinfected tertiary wastewater as defined by CDHS. The median and maximum total coliform limits are the same as for the disinfected tertiary wastewater for unrestricted urban use. Pathogenic microorganisms are controlled through the draft regulations regarding travel time and minimum distances to extraction locations that are dependent on the recharge delivery method. Filtration will be required to meet turbidity requirements. For surface spreading projects, the required minimum travel time for the recycled water is six months prior to extraction for use as a drinking water supply. Extraction shall not be within 500 feet of any GRRP surface spreading area. For subsurface injection projects, the minimum travel time is 12 months, and extraction shall not be within 2000 feet of any GRRP.

All GRRP must dilute the recycled water to be used as recharge with an approved source of water. The water source must be a potable source of water and cannot contain treated municipal wastewater. The ratio of recycled water to diluent water is regulated through a value termed the “recycled water contribution” (RWC). The maximum average RWC is specified by CDHS for each GRRP based on its review of the engineering report (Section 60320.080) and information presented during hearings on the GRRP. The average RWC cannot exceed 0.50, as calculated over a 60-month period, without specific approval by CDHS. If the RWC does exceed 0.50, the entire wastewater stream shall be treated by reverse osmosis.

Total organic carbon (TOC) is monitored in the filtered wastewater. TOC is not to exceed 0.5 mg/L divided by the CDHS-specified RWC, or the recycled water is to be treated by reverse osmosis to achieve this TOC level. For one year after initial startup, samples are to be collected and analyzed twice per week for TOC. Subsequently, based on review of the first year data, the CDHS may allow weekly sampling.

Three options are available to demonstrate the control of organic and inorganic nitrogen compounds. Table 6-4 details each option. Tables 6-5 through 6-10 summarize the maximum contaminant levels (MCLs) for constituents of concern in GRRPs. To determine compliance, samples are to be collected and analyzed quarterly for inorganics, organics, lead and copper, radioactivity, and disinfection byproducts. Once per year, samples are to be collected and analyzed for secondary constituents.

**Table 6-4 Three Options to Demonstrate Control of Nitrogen Compounds**

| | Option 1 | Option 2 | Option 3 |
|---|---|---|---|
| Compliance point | Recycled water or blend of recycled and diluent, in or above mound | - Recycled water or blend of recycled and diluent, in or above mound for total N<br>- Recycled water or recharge water in or above mound, for ammonia, org-N, nitrate, nitrite, and DO in excess of the BOD as required<br>- Groundwater down-gradient of the recharge area for DO as required | Groundwater down-gradient of the recharge area |
| Standards | - 5 mg/L total N as an average<br>- 10 mg/L total N at a max frequency | Recycled Water:<br>10 mg/L total N<br>As established by engineering report for:<br>- Total N at some level <10 mg/L when used as part of a comprehensive nitrogen control scheme<br>- Ammonia, nitrite, and/or org-N<br>- Minimum DO in excess of BOD<br>Groundwater:<br>- Min DO as established in the engineering report | Drinking water MCLs for $NO_3$ and $NO_2$ |
| Frequency of sampling | 2 per week | As established in Engineering report | 2 per month |
| Engineering Report | - Identification of criteria for suspending recharge<br>- Baseline monitoring and operations plan<br>- Monitoring plan | - Identify chemical or surrogate concentrations that will ensure that $NO_2$ and $NO_3$ MCLs are not exceeded in the groundwater down-gradient of the recharge area<br>- Identify criteria for suspending recharge<br>- Baseline monitoring and operations plan<br>- Monitoring plan | - Evidence that local recharge of water containing similar N levels over at least 10 yrs has not caused a problem & that recharge water can be tracked<br>- Monitoring plan<br>- Baseline monitoring and operations plan |
| Consequences of Failure | Investigate, correct, and notify based on average of 2 consecutive samples >5 mg/L and suspend recharge of recycled water based on an average of all samples collected during ensuing 2 weeks >5mg/L. Suspend recharge if more than 25% of samples collected in any 2 week period exceed 10 mg/L. | Investigate, correct, and notify based on average of 2 consecutive samples over the Total N standard, any standard for another form of N, or under the DO-BOD level or DO level. Suspend recharge of recycled water based on an average of a number of consecutive samples over the total N standard, any standard for another form of N, or under the DO-BOD level or DO level, as identified and justified in the engineering report. | Notify and either demonstrate compliance with MCLs or suspend recharge of recycled water, based on the average of 2 consecutive samples over an MCL |

| | Option 1 | Option 2 | Option 3 |
|---|---|---|---|
| Rationale | Option relies on such a low limit for the Total N in recycled water that the chance that the $NO_3$ or $NO_2$ MCL could be exceeded is minute. | Option relies on:<br>1. A low enough limit for Total N in the recycled water that the chance that the $NO_3$ or $NO_2$ MCL could be exceeded is minute, combined with<br>2. Some set of limits determined for specific GRRP and explained in the engineering report for nitrite, org-N, and/or ammonia necessary to limit oxidation to $NO_3$ or $NO_2$, and some set of min levels for an excess DO over BOD requirement in the recycled water and/or a DO requirement in the groundwater as necessary to prevent reduction of $NO_3$ to $NO_2$. | Option relies on:<br>1. A demonstration that historic recharge with water containing comparable levels of nitrogen has not caused a problem,<br>2. Evidence that recharge water can be tracked and monitored throughout the flow path, and<br>3. Monitoring to show that MCLs for $NO_2$ and $NO_3$ are met in the groundwater. Relatively frequent monitoring at locations between the recharge area and down-gradient domestic wells is required. |
| Adapted from CA DHS Draft regulations for Groundwater Recharge Reuse. 12/01/04. | | | |

**Table 6-5 Maximum Contaminant Level (MCL) for Inorganic Compounds**

| *Inorganic Chemicals* | *MCL (mg/L)* |
|---|---|
| Aluminum | 1 |
| Antimony | 0.006 |
| Arsenic | 0.05 |
| Asbestos | 7 MFL* |
| Barium | 1 |
| Beryllium | 0.004 |
| Cadmium | 0.005 |
| Chromium | 0.05 |
| Cyanide | 0.15 |
| Fluoride | 2 |
| Mercury | 0.002 |
| Nickel | 0.1 |
| Selenium | 0.05 |
| Thallium | 0.002 |
| MFL = million fibers per liter, for fibers exceeding 10 um in length | |

**Table 6-6 Maximum Contaminant Levels for Radioactivity**

| *Radioactivity* | *MCL (pCi/l)* |
|---|---|
| Combined Radium-226 & Radium-228 | 5 |
| Gross Alpha particle activity (including Radium-226, but excluding Radon & Uranium) | 15 |
| Tritium | 20,000 |
| Strontium-90 | 8 |
| Gross Beta particle activity | 50 |
| Uranium | 20 |

**Table 6-7 Reporting Limits and Action Levels for Lead and Copper**

| *Constituent* | *DLR[a] (mg/L)* | *Action Level[b] (mg/L)* |
|---|---|---|
| Lead | 0.005 | 0.015 |
| Copper | 0.050 | 1.3 |
| [a] DLR = Detection limit for reporting purposes<br>[b] Action level is based on the 90th percentile level | | |

**Table 6-8 Maximum Contaminant Levels for Organic Compounds**

| *Non-Volatile Synthetic Organic Chemicals* | *MCL (mg/L)* | *Volatile Organic Compounds* | *MCL (mg/L)* |
|---|---|---|---|
| Alachlor | 0.002 | Benzene | 0.001 |
| Atrazine | 0.001 | Carbon Tetrachloride | 0.0005 |
| Bentazon | 0.018 | 1,2-Dichlorobenzene | 0.6 |
| Benzo(a)pyrene | 0.0002 | 1,4-Dichlorobenzene | 0.005 |
| Charbofuran | 0.018 | 1,1-Dichloroethane | 0.005 |
| Chlordane | 0.0001 | 1,2-Dichloroethane | 0.0005 |
| 2,4-D | 0.07 | 1,1-Dichloroethylene | 0.006 |
| Dalapon | 0.2 | cis-1,2-Dichloroethylene | 0.006 |
| Dibromochloropropane (DBCP) | 0.0002 | trans-1,2-Dichloroethylene | 0.01 |
| Di(2-ethylhexyl)adipate | 0.4 | Dichlrormethane | 0.005 |
| Di(2-ethylhexyl)phthalate | 0.004 | 1,2-Dichloropropane | 0.005 |
| Dinoseb | 0.007 | 1,3-Dichloropropene | 0.0005 |
| Diquat | 0.02 | Ethylbenzene | 0.3 |
| Endothall | 0.1 | Methyl-*tert*-butyl ether | 0.013 |
| Endrin | 0.002 | Monochlorobenzene | 0.07 |
| Ethylene Dibromide (EDB) | 0.00005 | Styrene | 0.1 |
| Glyphosate | 0.7 | 1,1,2,2-Tetrachloroethane | 0.001 |
| Heptachlor | 0.00001 | Tetrachloroethylene | 0.005 |
| Heptachlor Epoxide | 0.00001 | Toluene | 0.15 |
| Hexachlorobenzene | 0.001 | 1,2,4-Trichlorobenzene | 0.005 |
| Hexachlorocyclopentadiene | 0.05 | 1,1,1-Trichloroethane | 0.200 |
| Lindane | 0.0002 | 1,1,2-Trichloroethane | 0.005 |
| Methoxychlor | 0.03 | Trichloroethylene | 0.005 |
| Molinate | 0.02 | Trichlorofluoromethane | 0.15 |
| Oxamyl | 0.05 | 1,1,2-Trichloro-1, 2,2-Trifluoroethane | 1.2 |
| Pentachlorophenol | 0.001 | Vinyl Chloride | 0.0005 |
| Picloram | 0.5 | Xylene | 1.750* |
| Polychlorinated Biphenyls | 0.0005 | | |
| Simazine | 0.004 | | |
| Thiobencarb | 0.07 | | |
| Toxaphene | 0.003 | | |
| 2,3,7,8-TCDD (Dioxin) | $3x10^{-8}$ | | |
| 2,4,5-TP (Silvex) | 0.05 | * MCL is either for a single isomer or the sum of isomers | |

**Table 6-9 Maximum Contaminant Levels for Disinfection Byproducts**

| ***Disinfection Byproduct*** | ***MCL (mg/L)*** | ***Detection Limit for Reporting Purposes (mg/L)*** |
|---|---|---|
| Total Trihalomethanes (TTHM) | 0.080 | |
| Bromodichloromethane | | 0.0005 |
| Bromoform | | 0.0005 |
| Chloroform | | 0.0005 |
| Dibromochlorormethane | | 0.0005 |
| Haloacetic acids (five) (HAA5) | 0.060 | |
| Monochloroacetic Acid | | 0.002 |
| Dichloroacetic Acid | | 0.001 |
| Trichloroacetic Acid | | 0.001 |
| Monobromoacetic Acid | | 0.001 |
| Dibromoacetic Acid | | 0.001 |
| Bromate | 0.010 | 0.005 |
| Chlorite | 1.0 | 0.02 |

**Table 6-10 Maximum Contaminant Levels for Secondary Constituents**

| ***Secondary Constituents*** | ***MCL/Units*** |
|---|---|
| Aluminum | .2 mg/L |
| Copper | 1.0 mg/L |
| Foaming Agents (MBAS) | 0.5 mg/L |
| Iron | 0.3 mg/L |
| Manganese | 0.05 mg/L |
| Methyl-*tert*-butyl ether (MTBE) | 0.005 mg/L |
| Odor - Threshold | 3 Units |
| Silver | 0.1 mg/L |
| Thiobencarb | 0.001 mg/L |
| Turbidity | 5 NTUs |
| Zinc | 5.0 mg/L |
| Total Dissolved Solids (TDS)* | 1,000 mg/L |
| or | |
| Specific Conductance | 1,600 microohms |
| Chloride* | 500 mg/L |
| Sulfate* | 500 mg/L |

* Constituents currently regulated under WDR at a lower concentration than specified here.

The two delivery options typically considered for groundwater recharge are direct injection with groundwater wells or surface spreading and percolation. The latter option may be preferred because it will allow natural filtration of the percolated wastewater throughout the geological subsurface or vadose zone, allowing further biological and filtration treatment. Direct injection is often energy intensive, requires high capital costs due to the requirement for RO treatment, may present public perception concerns, and may require an additional level of treatment to assure the public that contamination is not a significant risk.

The District is currently investigating potential sites for groundwater recharge. To be effective, the land must have proper soil characteristics for percolation and be located where recharge would increase availability of water in the aquifer. The project will require treatment process improvements, transmission pipelines, pump stations, and property for percolation ponds. Additionally, the District must identify a source of diluent water to blend with the recycled water prior to spreading or injection.

### 6.4 Option 3 Maintain Current Discharge Practices

Operating improvements made over the past two years have generally improved the wastewater effluent quality. However, groundwater monitoring and hydrogeological studies have indicated a clay layer between 60 and 140 feet beneath the site. This layer appears to be restricting percolation to groundwater and a mound of treated effluent is growing horizontally and upwards beneath the site. An investigation into various disposal alternatives has been initiated by the District. The Revised Draft Preliminary Evaluation of Southland WWTF Disposal Alternatives was completed in November 2008 (Boyle). The disposal/reuse alternatives considered included the current disposal practice (which was determined to be fatally flawed based on capacity and regulatory considerations), infiltration offsite using surface basins or subsurface systems, and irrigation of landscape or agricultural lands with recycled water. It may be possible to utilize onsite infiltration followed by pumping, for infiltration and storage before transporting the treated effluent offsite. The report provides a ranking to assist the District determine which alternatives to continue investigation.

Potential groundwater impacts are an important consideration if the District chooses to pursue infiltration. The Central Coast Regional Water Quality Control Board Basin Plan provides median groundwater water objectives for selected ground waters. These are intended to serve as a baseline for evaluating water quality management, and for establishing limits for discharge permits. The following values are given for the Lower Nipomo Mesa:

- Total Dissolved Solids (TDS) = 710 mg/L
- Chlorides (Cl) = 95 mg/L
- Sulfate ($SO_{4)}$ = 250 mg/L

- Boron (B) = 0.15 mg/L
- Sodium (Na) = 90 mg/L
- Total Nitrogen (TN) = 5.7 mg/L

It is important to note that aerated or facultative ponds (similar to Nipomo's current treatment process) are not capable of meeting any of the water quality goals listed in the Basin Plan for the Lower Nipomo Mesa, nor is it adequate pretreatment for nitrogen removal or salts reduction processes.

Therefore, it is recommended that the District explore treatment technologies in their next treatment plant expansion that will, at a minimum, provide adequate pretreatment for future process improvements to meet these parameters.

### 6.5 Recommendations

Water quality goals will dictate the appropriate level of treatment for the future wastewater treatment plant. Recommendations to assist in that determination are as follows:

- Sample effluent for constituents that may effect reuse as irrigation: $EC_w$, SAR & $SAR_{adj}$, boron, and fecal coliform.
- Sample effluent for constituents that may effect reuse as recharge: TOC, turbidity, organic and inorganic nitrogen.
- Perform a user survey to determine the potential market for reclaimed water. This will need to be done in conjunction with a public information campaign.
- Select a future treatment plant process which will provide adequate pretreatment for filtration. If uses such as park/school irrigation, groundwater recharge, or infiltration (under more stringent permit limits than the plant's current permit) are pursued for the expanded treatment facility, aerated ponds will not provide adequate treatment or pretreatment.

# 7.0 SYSTEM IMPROVEMENTS

## 7.1 Frontage Road Trunk Main

A hydraulic analysis based on Manning's equation was performed on the Frontage Road trunk main from Division Street to the WWTF. The analysis allowed identification of trunk main sections that are insufficiently sized to handle existing and/or future flows based on the allowable water depth, or d/D as discussed in Section 5.1 (See Figures 5-2 and 5-4). Several sections currently fail to meet the criteria for PHF and the majority of the line is expected to fail for both average and peak future flow rates. The minimum pipeline diameters needed to meet both existing and projected demand were calculated. A 15-inch pipeline will handle existing flow rates, but a 21-inch replacement is recommended to meet future peak demand. The 15-inch upgrade is estimated to cost approximately $1,800,000. The 21-inch upgrade is estimated to cost about 20% more, at $2,200,000. The cost opinions are based on open trench construction. Pipe bursting may be an option, but a geotechnical study and identification of nearby utilities would be required to determine feasibility. Additional assumptions are listed with the detailed cost opinions, included in Appendix C.

## 7.2 Influent Pump Station

<u>Electrical Supply Reliability</u>

The WWTF uses two influent pumps to pump incoming wastewater to treatment ponds. The Fairbanks Morse submersible pumps are 35 HP each and rated at an approximate 2300 gallons per minute (gpm) capacity. Occasionally, the WWTF experiences an imbalance in the utility power supply, which causes temporary pump failure. This causes submergence of the trunk sewer and the Parshall flume throat, resulting in false meter readings. The electrical problem is likely a result of the plant's position as the end user on the distribution line, where many "up-stream" residential developments, which are single-phase loads, create an imbalance in the line's three-phase voltage. This theory was substantiated by a data logger that revealed voltage differences of up to 12-15 volts between phases. While this is a problem for the District, it is within the delivery tolerances allowed by Pacific Gas & Electric (PG&E) for their customers. The District has installed motor savers on the pumps, to protect the motors during voltage imbalances, but this results in deactivating the motors and causing surcharges. A small voltage imbalance can create a large current imbalance, and may thereby increase heat in the motors and lead to premature motor failure.

Several methods were considered to reduce or eliminate the electrical problem at the pumps, as follows:

1. Variable-Frequency Drives (VFDs) convert the three-phase power to a direct current and then convert it back to an adjustable frequency three-phase voltage. By slightly oversizing the VFD, the VFD can

accommodate a severe input voltage imbalance and produce a completely balanced output voltage to the motor. Disadvantage is high cost and complexity.

2. The solid-state starter (Allen Bradley Dialog Plus) has a unique feature called a phase re-balance feature. In lieu of bypassing the solid state starter once it gets the motor up to speed, as is conventionally done, the solid state starter remains in the circuit and reduces the voltage of the high phase(s) to balance it with the other phases(s). We recommend a bypass contactor also be installed as a backup to the solid state starter with a hand switch with "soft-start only, bypass only and normal" positions. This option appears to be the most favorable with regard to cost and operability.

3. A larger motor on the same pump could handle the voltage imbalances without overloading any of the three motor phases since the rating of the motor phases would be higher. Disadvantage is that pump and wiring must also be replaced resulting in a high cost. However, if District is planning on a pump replacement for other reasons, this is the simplest and least technical option at about the same cost as the solid state starter.

<u>Wetwell and Pumping Capacity</u>

Analyses show the existing influent pumps have capacity to handle existing flow, but will need to be upgraded to maintain redundancy while meeting future demands. The wetwell volume calculations also showed that the wet well is undersized for existing conditions. The cycle time was calculated to be 3 minutes for existing peak hour conditions. However, staff has estimated that the pumps are cycling every 15 minutes during peak hour flow. Additional investigation is recommended to fully evaluate the existing pump station and determine appropriate alternatives to meet future demand. An excessive number of pump starts per hour (greater than 4 or 5) results in shorter useful life for starters and motors.

On a short-term basis, assuming no pump station upgrades are performed for several years, retrofiting the existing pumps with VFDs was investigated as an option to reduce required capacity of the wet well. VFDs will allow the pumps to run at a reduced speed. They also assist with the voltage imbalances as discussed above. The disadvantages are cost, some decreased efficiency, and complexity of operation. In order to retrofit the pumps with VFDs, the minimum flow must be determined. It is not recommended to operate pumps at flows less than 30% below their best efficiency point to maintain sufficient shaft speed for discharge against the static head. Review of the pump curve indicates the highest efficiency point for the existing influent pumps is at 2000 gpm. Therefore the recommended minimum flow rate is 1400 gpm, at an operating speed of 850 rpm. At this flow the required active volume to provide a 10-minute cycle time per pump at peak flow is 1750 gallons or 220 $ft^3$. Though this is nearly half the volume needed without VFDs, the existing wet well is still smaller than desired for pump cycling (existing active volume of 186 $ft^3$).

Installing VFDs on the existing pumps is not recommended at this time, since pump capacity will eventually need to be increased to meet 2030 flow. The existing pumps are each rated at 2300 gpm, or 3.3 mgd. Peak demand with the existing pumps (while maintaining 100% redundancy) is projected to occur in 2018. Therefore, it is recommended that new pumps be installed by 2015 (at the latest – constructing a new pump station could be accomplished sooner, while upgrading the Frontage Road trunk main to reduce construction cost and minimize plant service outages) to provide a "planning buffer" since flow projections are imprecise. Either the existing pumps could be replaced with two new pumps, or a third pump could be installed to meet peak demands while operating in parallel with one of the existing pumps.

Solids Handling

Alternatives to the existing submersible solids-handling pumps warrant investigation. Operators have reported problems with the existing pumps clogging from rags and other large materials. There are no screens upstream of the pumps, only grinders, which pass material through the influent pump station and into the wastewater treatment facility. Screw-centrifugal pumps (such as a Wemco Hidrostal® or approved equal) combine the high efficiency of a centrifugal pump (80% or greater) with the clog-free advantage of a vortex pump. The screw impeller provides a smooth flow and low turbulence, reducing hydraulic losses, keeping power costs down. The large screw channel from suction to discharge reduces clogging and maintenance.

To further enhance solids removal and continual cleaning of the wetwell, a prerotation basin can be installed in the wet well. Wemco offers the Prerostal® System with the Hidrostal® pump. The basin is constructed with a partial weir to induce rotation towards an inclined tangential entrance channel, where a bellmouth suction pipe draws water into the pump and causes the liquid to enter the impeller at a different angle than the pump was originally designed for. The result is a lower head-capacity curve and a reduction in energy consumption. The higher the velocity in the prerotation basin, the greater the decrease in capacity from original design. With the geometry of the prerotation basin and gravity as the control mechanism, the discharge flow automatically matches the influent flow rate without changing pump speed. Using a constant pump and motor speed the flow can be varied to as low as 35% of it's design capacity. A major benefit to the system is that the pump will automatically draw floating and settled solids, which will reduce odors and eliminates the need for cleaning the wet well.

Screenings and floatables would then be removed by a downstream screening and grit removal system (see Section 7.3)

<u>Recommended Influent Lift Station Improvements</u>

At this time we recommend that the District budget for a pump station replacement, including a new wet well with a prerotation basin and three screw centrifugal pumps, sized so that any two could handle the PHF at 2030. The budget for this work is summarized in Table 7-1:

**Table 7-1 Cost Opinion for Influent Pump Station Upgrade**

| Item | Estimated Installed Cost |
|---|---:|
| Flow Metering Manhole | $40,000 |
| 3 Screw Centrifugal Pumps | $140,000 |
| Valves and Piping | $150,000 |
| Wetwell | $200,000 |
| Demolish/Salvage Existing Facility | $20,000 |
| Electrical, Controls, and Instrumentation | $70,000 |
| Engineering/Admin (20% of Subtotal) | $124,000 |
| Contingency (30% of Total) | $223,200 |
| Total | $967,200 |

### 7.3 Screening and Grit Removal

Two screen technologies were investigated for headworks improvement: shaftless spiral and in-channel moving screens. Each screen would feature 6-mm openings, all stainless steel hardware and wetted parts, pressure wash capability, and capacity for future (2030) PHF. We also recommend using two screens in parallel (each with 100% PHF capacity) for process redundancy. The costs are compared in Table 7-2, with a detailed breakdown in Appendix C, and product information in Appendix D.

Shaftless spiral screens (such as the Parkson Hycor® Helisieve® or approved equal) are in-channel, units that combines screening, conveying, and dewatering (Figure 7-1). They are typically mounted in a concrete channel with a grated cover. A bypass channel should be provided in case the units become clogged and the screen stops functioning. The spiral conveyor is fitted with a steel brush for continuous cleaning of the screen surface.

The conveyor operates intermittently, based on time, differential level, or manual initiation of the screen cleaning cycle. A bagger unit can be added for collection of screenings. The shaft pivots out of the channel for maintenance accessibility. This equipment requires no submerged end bearings or intermediate hanger bearings.

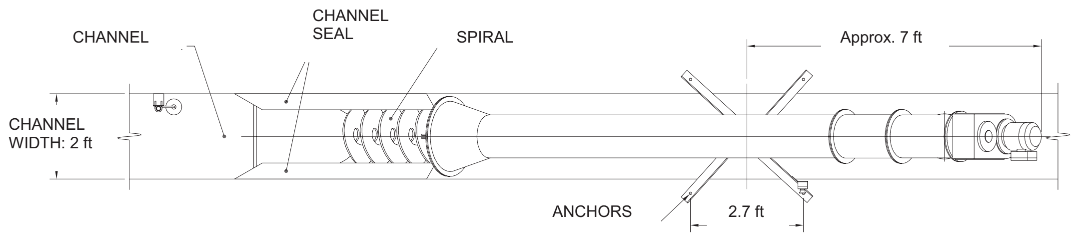



**Figure 7-1 Top view Hycor® Helisieve®**

An alternative is an in-channel, moving screen (such the Parkson Aqua Guard® or approved equal), as shown in Figure 7-2. Similar to the shaftless spiral screen, the moving screen operates intermittently, based on time, differential level, or manual initiation of screen cleaning cycle. This reduces power consumption and wear on the equipment. It is self cleaning and all moving parts can be accessed above water level. The screen pivots out of the channel for ease of maintenance.

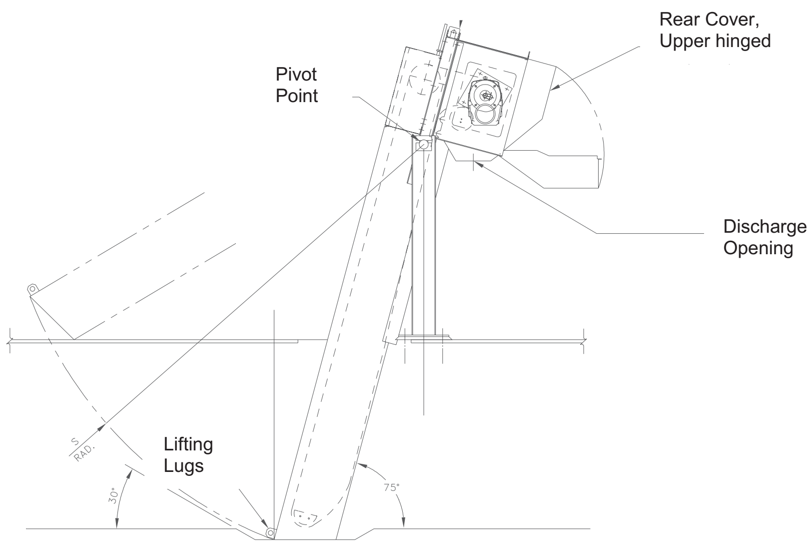



**Figure 7-2 Profile view AquaGuard®**

Alternatives for Grit Removal

Two systems were investigated for grit removal: vortex and aerated systems. Costs are included in Table 7-2. The Jones & Attwood® Jetair is a vortex flow and tangential entry grit trap (Figure 7-3). Coupled with a Jones & Attwood Screw Classifier, the system is designed to separate inorganic solids from influent wastewater. Either two units could be installed, each able to handle 50% of the projected 2030 PHF and allow temporary operation with one unit while maintenance is performed on the other, or one unit with a bypass could be provided to handle 100% of PHF.

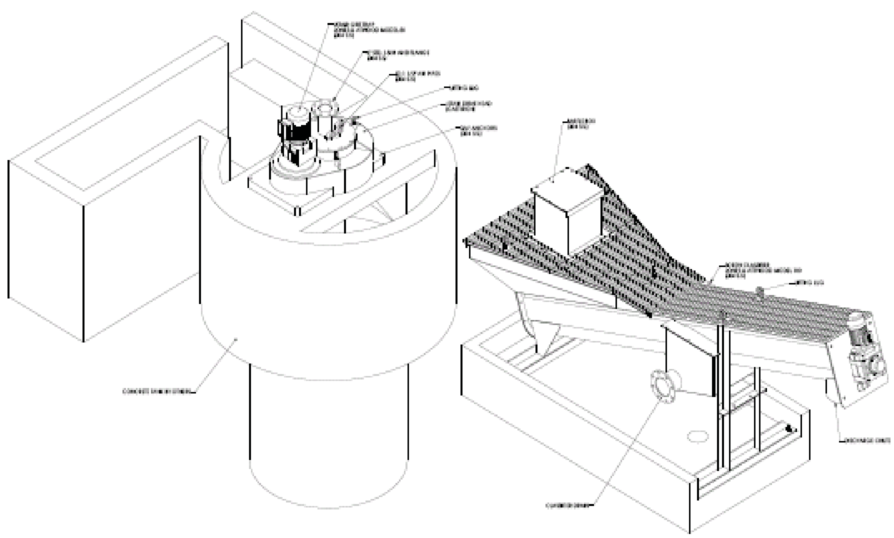


**Figure 7-3 Jones & Attwood JetAir® and Screw Classifier**

(Detailed photographs and drawings included in Appendix D)

An aerated grit chamber is an economical alternative to vortex grit removal. Air is introduced from one side of a rectangular chamber, perpendicular to the wastewater flow to create a spiral flow pattern through the tank. Heavier grit particles settle to the bottom of the chamber, while lighter particles – primarily organics – remain suspended and pass through. When compared to the vortex grit removal system, aerated grit chambers require more air piping, diffusers, and mixing, which demand more power and maintenance, but are typically less expensive to construct. Aerated grit chambers require blowers to blow air through the water and overcome static head from the depth of diffusers. Since the District already has blowers onsite, and an air line is near the existing headworks, they already have aeration capability for the chambers. Aerated grit chambers sometimes contribute to odors and headworks corrosion through the creation and release of hydrogen sulfide.

Drum Screens

A potential alternative to screening and grit removal systems is a drum screen. A drum screen will remove more material than a mechanical screen alone, but less than a combined system as presented above. The advantage to this option is having only one headworks system to maintain, assumedly simplifying operations. However, drum screens often require more maintenance than other screens, since they typically have a smaller opening than mechanical screens (3 mm verses 6 mm) and can clog more frequently. Though more expensive than other types of screens, when comparing to a dual screen and grit removal system, the capital costs are similar. Drum screens require continuous wash water at higher flow rates than required for coarser screens (described above) and conveying, dewatering, and bagging must be performed separately.

**Table 7-2 Cost Opinions for Screening and Grit Removal Systems**

| Improvement Option | Estimated Installed Cost |
|---|---|
| **Screens** | |
| (2) Parkson HLS500 Hycor® Helisieve® | $512,000 |
| (2) Parkson Aqua Guard® AG-MN-A | $855,000 |
| **Grit Removal** | |
| (2) Jones & Attwood JetAir 100 Grit Trap + Model 100 Screw Classifier | $629,000 |
| (2) Aerated Grit Chambers[4] | $588,000 |

Recommendations for Screening and Grit Removal Systems

Two (2) shaftless screw screens are recommended for screening, since they require lower capital cost and provide better dewatering and compaction of solids than a mechanical screen.

A vortex grit removal system (such as the Jones & Attwood JetAir® grit trap) is recommended as part of the headworks improvements at the WWTF. The capital costs are higher than an aerated grit chamber, but the system requires less maintenance than an aerated grit chamber which requires regular repair and replacement of air valves, fittings, diffusers and piping in the basins

[4] Includes cost for grit classifier, which is estimated at $150,000 for the grit chambers.

## 7.4 Sludge Removal

Currently, ponds are drained by temporary pump systems to remove sludge and convey it through buried sludge pipes to the drying beds. Draining a pond is a time-consuming task and the WWTF must take the pond out of service, requiring operation using the remaining ponds until the sludge removal is complete.

Two alternative removal methods were investigated to reduce maintenance time and avoid taking the ponds out of service. One alternative is to retrofit the pond with a central sump and submersible pump, as shown in Figure 7-4. This improvement would be done in conjunction with the addition of a pier/walkway to the center of the pond. The pond floor would be sloped towards the center to encourage settling towards the center sump for sludge removal, where a submersible pump would transport the sludge through a pipeline that would be routed along the walkway to the drying beds.

Several problems are anticipated with this option. First, long-term effectiveness is questionable. Once the pump removes the sludge in the immediate area, water would fill the void much faster than the surrounding sludge and the pump would start drawing mainly water. Second, even if a design were created to render this option effective, the economic impact of re-grading is likely to be significantly greater than that of other sludge removal alternatives. Construction cost is estimated at approximately $220,000 - $275,000 per pond.

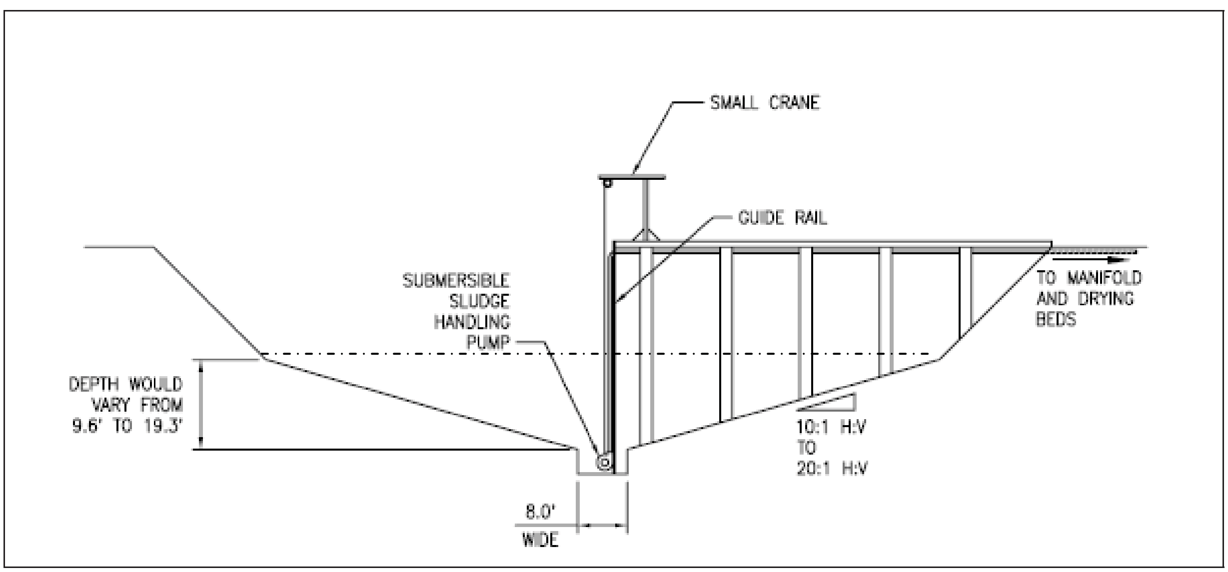



**Figure 7-4 Conceptual schematic of pond with sump**

A second alternative is to dredge the ponds. Crisafulli offers a dredge rental program. Other vendors may provide a similar service. The Crisafulli system and rental service was evaluated in this study, but competitors should be identified and consulted if the District wishes to proceed with this alternative. The FLUMP® (floating lagoon pumper) is an unmanned, remote-controlled electric dredge. The Model ST-3 standard duty Flump® offers a sludge discharge capacity of up to 25 cubic yards per hour and a dredging depth of 0 – 8 feet, though it can be customized for greater depths. A floating dredge allows the basin to remain full during the sludge removal process. The cutterhead can be fitted with a cage for liner protection. It uses a patented floating discharge system and is able to discharge sludge from distances of up to 500 feet from shore. The dredges are moved, manually or automatically, along a tensioned steel cable extending across the pond and fixed to steel posts. The ST-3 runs on 460 volts and can be powered by a 75 hp generator.

Maneuvering around the surface aerators is one of the challenges in using a cable-directed dredging unit. However, if aerators were relocated in approximately ½ of the pond, the dredge could operate within that area while the aerators in the other ½ of the pond continue to function.

**Figure 7-5 Severe duty Flump® operating on traverse system to dredge a pig lagoon**

The rental package for the standard ST-3 Flump® includes the control panel, 200 feet of floating discharge pipe, a 4 post manual traverse system, and 500 feet of power and control cord. The estimated cost is shown in Table 7-3. Additional product information can be found in Appendix D.

**Table 7-3 ST-3 FLUMP® Cost Opinion**

| | |
|---|---|
| 1 month rental package (+ 100' additional float pipe) | $7,070 |
| Round-trip freight | $5,350 |
| Installation + 2-day training | $3,960 |
| Damage deposit | $3,345 |
| Total estimated cost for 1st month (with deposit) | $19,725 |
| Cost per month for subsequent dredging (with deposit) | $15,765 |
| Cost based on January 2007 quote | |

### 7.5 Operability and Automation

Automation and Controls

The Southland Wastewater Treatment Facility is on the District's read-only Supervisory Control and Data Acquisition network. The following systems are transmitted by radio across the District's web-based system:

- Influent flow (gpm)
- Influent pump 1 on
- Influent pump 2 on
- High wetwell level
- Each aerator on
- Grinder 1 on
- Grinder 2 on
- Power outage
- Generator on

The level of automation and controls at the plant is relatively low. Influent pumps are activated by float switches in the wetwell. This is the only pumping facility on site – flow through the ponds, and to the percolation ponds, is gravity-driven. In the event of a power failure, an automatic transfer switch will activate the onsite diesel generator, which provides power to the aerators, lift station, and blowers.

<u>Monitoring/Analytical Capabilities</u>

The District has an influent flow meter, dissolved oxygen (DO) probes in the primary ponds (1 each), and 2 staff gauges to monitor levels in 2 of the percolation ponds. The District does not have a laboratory, but uses some portable analytical kits for measuring some parameters such as nitrate and nitrite levels.

It is our understanding that the District intends to install staff gauges in all of the percolation ponds. Staff also intends to construct a laboratory in the old shop, as well as a new transducer in the wetwell to replace the float switches. Another planned improvement is reconfiguration of the aerator controls and dissolved oxygen probes to control aerators by DO levels. Staff will develop a system to allow them to step-up or step-down the number of aerators in operation to maintain consistent DO levels. At a minimum, it is recommended that the aerators closest to the outlets be provided with DO controls since these aerators would face lower regular BOD loading than the inlet-side aerators.

In addition to these changes, we would recommend adding current meters to read and transmit amperage for each aerator, pumps, and grinders (if they remain in operation). This would allow operators to remotely detect problems that would increase or decrease load (and cause changes in current) on the motors, such as clogged pumps, “ragging” of aerators, and blockage in the grinders.

If a laboratory is constructed, equipment should be purchased to allow District staff to measure BOD as a “quality control” method to check laboratory results, since they have been questionable (as discussed previously). The lab could also be outfitted to perform sludge volume index (SVI) and total suspended solids (TSS). The laboratory should also have a vented hood, to allow the District to run Chemical Oxygen Demand (COD) tests and other tests which require ventilation for safety.

<u>Improved Pond Access</u>

Representative sampling is a goal for any wastewater treatment plant. Building piers for access into the pond interior area is a relatively simple improvement to gain better access for representative sampling. It is difficult to obtain representative samples at the shore due to floating and submerged debris build up caused by wind and pond circulation patterns. Construction of a pier would require draining the ponds and modification to the liners for installed footings or piles with columns for support. Placement should be near the pond outlet where the majority of the treatment has been accomplished, extending out to the deepest part of the pond to avoid collection of material from the sides when sampling. The side-slope ends approximately 42-feet from the edge of the pond. The walkway should be aluminum-framed with stainless steel handrails. Gatordock makes an aluminum fixed pier. A 40-foot long by 6 feet wide DuraDock® with handrails is expected to cost approximately $16,000. This includes the cost of four plastic coated wood pilings and shipping. It does not include costs associated with modification of the liner or installation of an anchoring system. The main disadvantages to a fixed pier include the disruption of service for construction, the potential for interference with pond retrofits or sludge removal, and the cost and potential problems with modifying the pond liner.

An alternative option is a floating pier with anchoring to the side of the pond. ShoreMaster's floating Polydock® is made from UV-resistant polyethylene (Figure 7-6). A straight 48-foot long Polydock® (6-feet wide) with handrails and an 8-foot long gangway is estimated to cost approximately $20,000, plus costs for an anchoring system.

**Figure 7-6 ShoreMaster's Polydock®**

Flow Direction in Ponds 3 and 4

District staff currently has plans to install a submersible pump in the telescoping valve vault in Pond 4. The pump will provide a means for transporting the effluent from Pond 4 to the front of Pond 3. After Pond 4 is put back online, Pond 2 will be drained for maintenance and water will be directed through the remaining ponds in series: Pond 1, to Pond 4, to Pond 3.

## 7.6 Recommendations for Facility Improvements

Several system improvements are recommended.

- *Frontage Road trunk main replacement*: Hydraulic analysis revealed deficiencies in the size of the Frontage Road trunk main. We recommend replacing the Frontage Road trunk main with a 21” pipeline to meet the projected demand for 2030. This project should be constructed in the next 2 years.
- *Influent pump station upgrade*: The influent pump station will need improvements to handle future conditions. Analysis indicates that though the existing pumps have the capacity to handle existing flow, the wet well is undersized, causing rapid cycling, which can prematurely wear the pumps. We recommend that the District budget for a wet well replacement and three new screw centrifugal pumps (such as Wemco Hidrostal® or equal) to meet 2030 demands. This project would be most efficiently constructed with the Frontage Road trunk main improvements, but should be in place no later than 2012 to prepare for 2015 projected demands.
- *Screening and grit removal*: Headworks improvements will increase effluent quality and significantly reduce maintenance issues (such as rag entanglement in the aerators) and wear on the plant equipment. Two parallel shaftless screw screens (such as Parkson Helisieve® or equal) is recommended for the fine screening, followed by two vortex grit removal systems (such as Jones & Attwood JetAir® or equal). We recommend installing screening and grit removal within the next 2 years.
- *Solids handling*: Rent a portable dredging unit (such as the Crisafulli Flump®) for sludge removal from the aerated ponds (after all subsurface equipment is removed).
- *Control and automation*: In addition to the upgrades the District has planned, we recommend adding current meters to aerators, pumps, and grinders to read and transmit amperage.
- *Increase pond access*: Fixed and floating piers were investigated. Floating piers can provide pond access at a reasonable cost without constructing a permanent structure or damaging the pond liner. If pond access is desired for sampling or monitoring, or for access to a new floating outlet (see below), we recommend installing a floating dock.

# 8.0 FUTURE PROCESS ALTERNATIVES

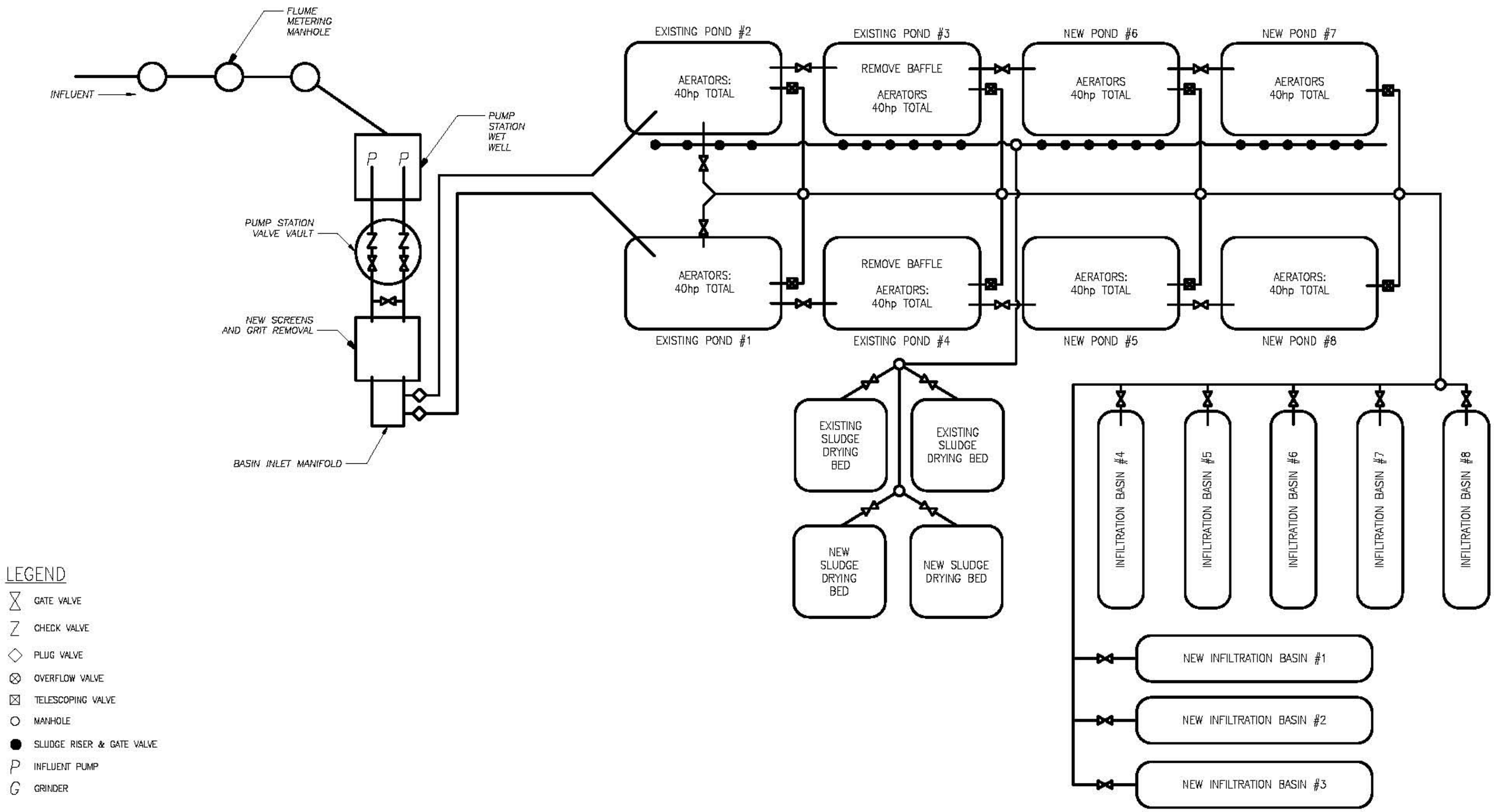
The anticipated effluent requirements for permitting and future flow increases necessitate investigation of treatment process alternatives.  Four alternatives were reviewed and are discussed below: expansion of the current treatment process with additional aerated ponds, a conversion to Biolac® Wave Oxidation System (an extended aeration technology), a conventional activated sludge system, and an oxidation ditch.  Most of these options could be implemented in phases, spreading the capital cost out over several years.  A summary of comparative cost opinions is shown in Table 8-2.  Cost details are included in Appendix C.  Sizing and cost opinions are based on meeting an AAF of 1.67 mgd, for 2030 demand.

## 8.1 Expansion of Aerated Ponds

The WWTF currently uses four aerated ponds for treatment.  Under normal operation, the wastewater flow from the influent pump station is split into the primary ponds, Ponds 1 and 2, then flows into the secondary ponds, Ponds 4 and 3, respectively.  The inlet and outlet ends of the secondary ponds were previously split with a baffle curtain to minimize short-circuiting and provide a quiescent zone.  The front 40% of each pond was aerated with two 5-hp mechanical surface aerators, and the back 60% was a stabilization basin, providing settling time.  In 2007, the baffle curtain was removed to maximize aerated volume.  The WWTF currently runs 3 aerators each in Ponds 3 and 4.  Pond 3 has two 5 hP aerators and one 10 hP.  Pond 4 contains three 10 hP aerators.  The District plans to replace all existing 5-hp aerators with 10-hp aerators.  Figure 4-1 shows the existing process flow diagram.

Based on the projected flows discussed in Section 3.0 and a $BOD_5$ effluent goal of 80 mg/L, four additional ponds would be needed, each with an equivalent liquid volume of the existing secondary ponds (approximately 3.1 million gallons).  Calculations were performed with the assumption that the baffling in the existing secondary ponds would be removed to provide additional aerated capacity for treating increased flows.  Appendix B contains the complete calculations.  Additional aerators, providing 195 hp more, will be needed for adequate aeration in the new ponds (total of 315 hp).  The process flow diagram for this option is provided as Figure 8-1.  A recommended layout for the four additional ponds is shown as a site plan in Figure 8-2.  Though there is open area behind the existing ponds, only two ponds of this size will fit.  We recommend constructing the four new aeration basins in place of the existing infiltration basins #1, 2, and 3.  Additional sludge drying beds could be constructed in the area behind the existing aeration ponds and there is room to the southwest, behind infiltration basins #4 through #8, to construct additional infiltration basins.  The improvements could be implemented in phases, as the demand requires.

One of the main disadvantages to constructing additional aerated ponds is the inability to meet a higher level of treatment than is currently required in the WDRs, as well as poor nitrogen removal.  In addition, aerated or facultative ponds will not produce effluent that can be efficiently filtered for recycled water applications such irrigation at parks or schools.  This option will sufficiently treat the wastewater with projected future hydraulic and loading demands with respect to current water quality goals.  However, more stringent water quality regulations are anticipated for the future and if the District chooses to pursue groundwater recharge or another reuse alternative, additional treatment to reduce nitrogen concentrations and other constituents in the effluent will be required.  The capital cost is for this option is one of the highest, due to the large amount of excavation and fill required.  The cost opinion does not include excavation and grading for additional infiltration basins or sludge drying beds, which are discussed in Sections 8.6 and 8.7.

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**NCSD SOUTHLAND WWTF MASTER PLAN**

**EXPANSION OF AERATED PONDS PROCESS FLOW DIAGRAM**

BEC PROJECT NO. 19996.17

FIGURE **8-1**

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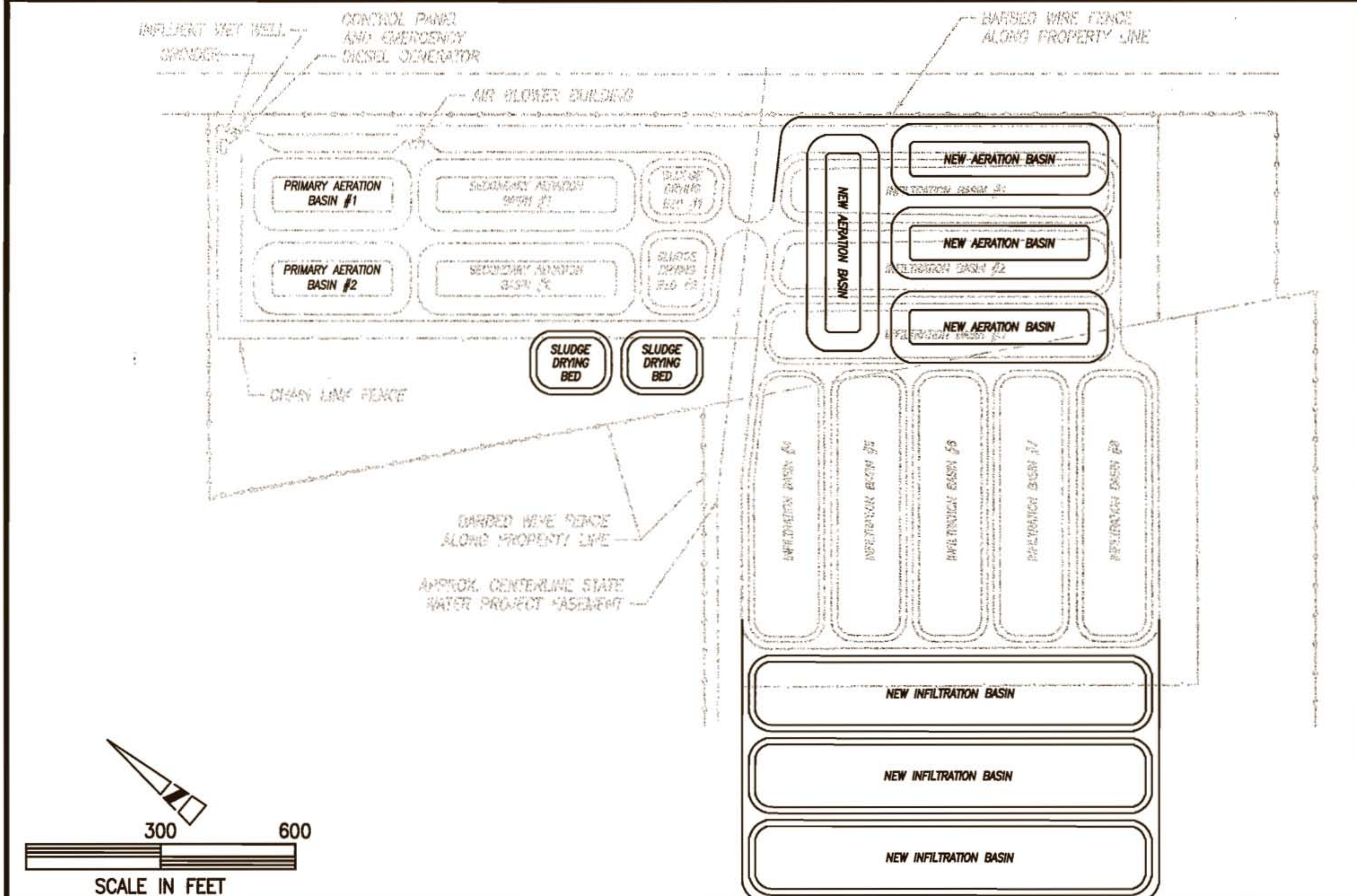


| FIGURE 8-2 | NCSD SOUTHLAND WWTF MASTER PLAN<br>ALTERNATE 1:<br>SITE PLAN FOR AERATED PONDS | BEC PROJECT NO.<br>19996.17 | BOYLE ENGINEERING CORPORATION |
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## 8.2 Biolac® Conversion

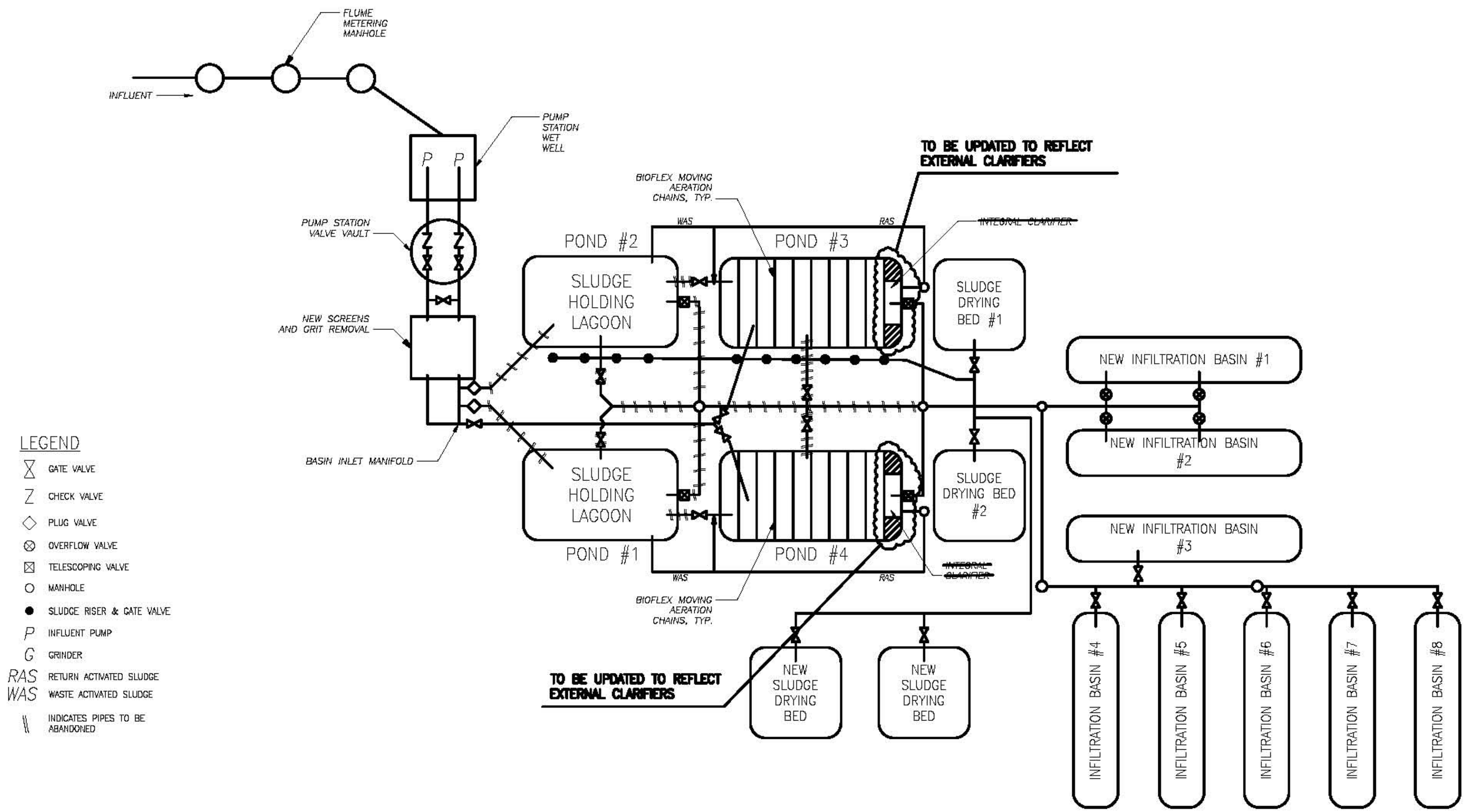
The Parkson Biolac® Wave Oxidation System is an extended aeration process that utilizes a longer solids retention time (SRT) and moving aeration chains to reduce BOD and TSS concentrations to below 15 mg/L and total nitrogen to less than 10 mg/L. The extended SRT increases the stability of the system, allowing for fluctuating loads under similar operating conditions. Airflow to the moving aeration chains can be controlled to create a wave of aerobic and anoxic zones, resulting in nitrification and denitrification. Multiple fine-bubble diffusers are mounted on the flexible air tubing suspended across the pond. The flexible Biolac aeration system maintains the required mixing and suspension of solids at 4 cubic feet per minute per 1000 cubic feet of aeration basin volume, half that required for a typical stationary aeration system. Appendix D contains additional product information.

The process flow diagram for a Biolac retrofit and site plan are shown as Figures 8-3 and 8-4. One main advantage to this option is the high level of treatment provided within a small footprint and relatively lower cost than comparable technologies. It can be retrofitted into the existing ponds with some piping modifications and can utilize the existing blowers. To handle the future projected flow rates, two secondary ponds will eventually need to be converted to Biolac systems. This would include installation of the Wave Oxidation system, which will each fit within the footprint of a pond, and new secondary clarifiers. A Biolac system in one pond will provide adequate treatment until the MMF reaches approximately 1.4 mgd, currently projected for 2020, allowing a phased upgrade. This would leave three aeration ponds for the facility to stay online during the retrofit. Otherwise, for redundancy, two ponds could be retrofitted with sufficient diffusers to meet the 2020 demands and additional diffusers could be added later. After the conversion, the unused primary ponds could be used for sludge holding and digestion. Sand or multi-media filtration can easily be added to the treatment train to provide a higher quality effluent if required, whereas conventional aerated or facultative pond systems do not produce effluent quality that is compatible with filtration equipment.

The main disadvantage to a Biolac upgrade is increased maintenance and control requirements, inherent in the higher level of technology. Blower controls are needed for aeration cycling. The diffuser sheets will need to be replaced approximately every 5 to 7 years and the air tubing will need replacing about every 7 to 10 years. The diffuser assemblies are designed for neutral buoyancy, and are lightweight and compact for easy retrieval. For the level of treatment, Biolac appears the most maintainable when compared with activated sludge and oxidation ditch systems – simple, accessible parts, relatively inexpensive to replace.

The life-cycle power and replacement costs for a Biolac system were compared to that of an aerated pond system. Power consumption and material needs to the year 2030 were determined assuming the systems were constructed to meet the projected 2030 demands. The cumulative present-worth costs for Biolac would be approximately $8,015,000, while a pond system would cost approximately $14,300,000. Figure 8-5 summarizes the comparative, cumulative life cycle costs, assuming the system is built this year. Costs for disposal systems and sludge drying beds were not included, since it is assumed these facilities would be the same cost for each alternative. Assumptions are included in the detailed cost opinion in Appendix C.

It should be noted that a Biolac system will require a Class II Wastewater Treatment Operator, whereas pond systems require only Class I certification. Therefore, the District must ensure that a Class II Operator directs plant operations if Biolac is selected.

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| NCSD SOUTHLAND WWTF MASTER PLAN | BEC PROJECT NO. | FIGURE |
|---|---|---|
| BIOLAC® CONVERSION PROCESS FLOW DIAGRAM | 19998.17 | 8-3 |

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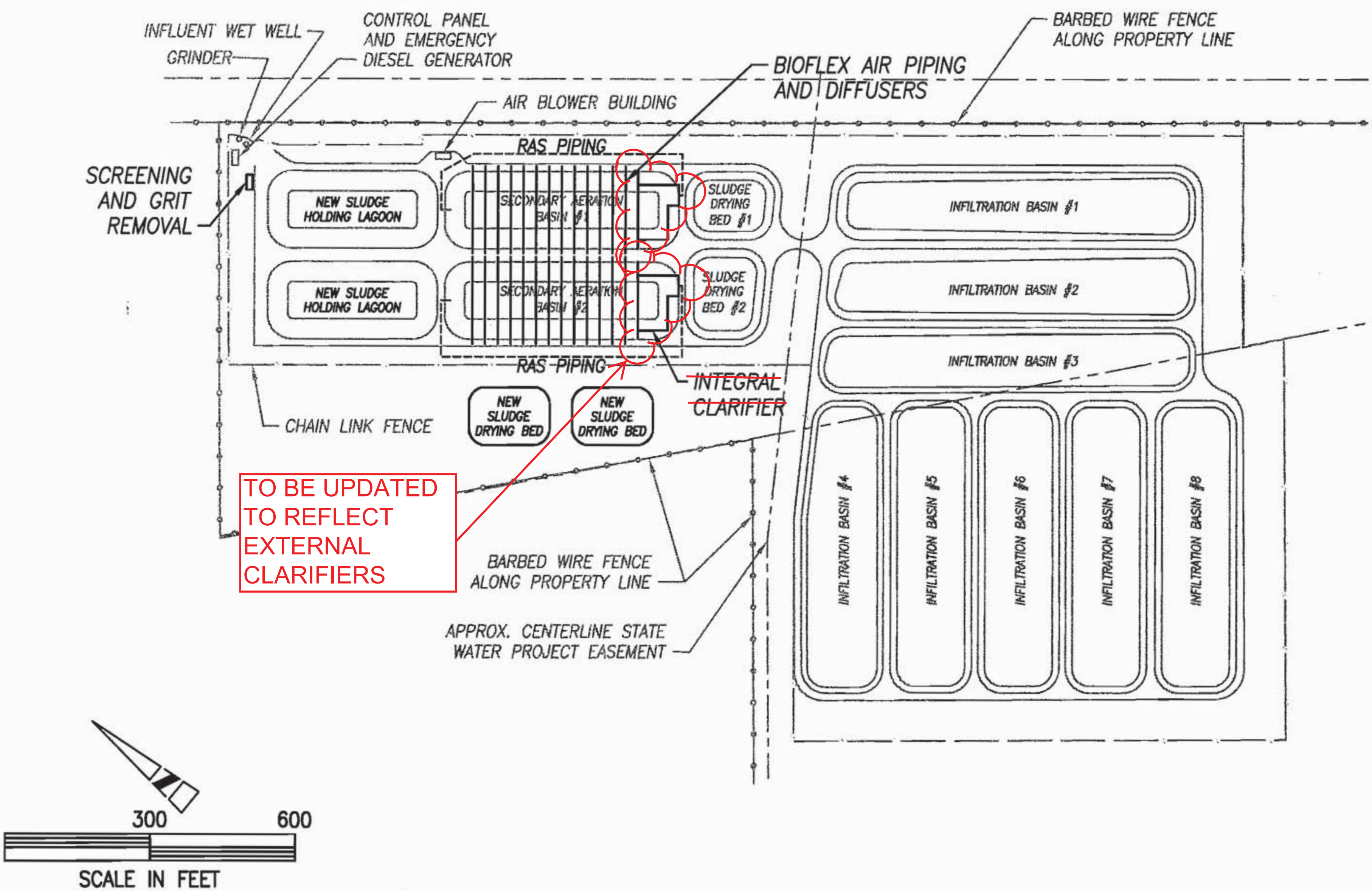



| | | | |
|---|---|---|---|
| FIGURE 8-4 | NCSD SOUTHLAND WWTF MASTER PLAN<br>ALTERNATE 2:<br>SITE PLAN FOR BIOLAC | BEC PROJECT NO.<br>19996.17 | BOYLE ENGINEERING CORPORATION<br>1194 Pacific St., Suite 204 San Luis Obispo, CA 93401<br>Tel. 805-542-9840 Fax 805-542-9990<br>WWW.BOYLEENGINEERING.COM |

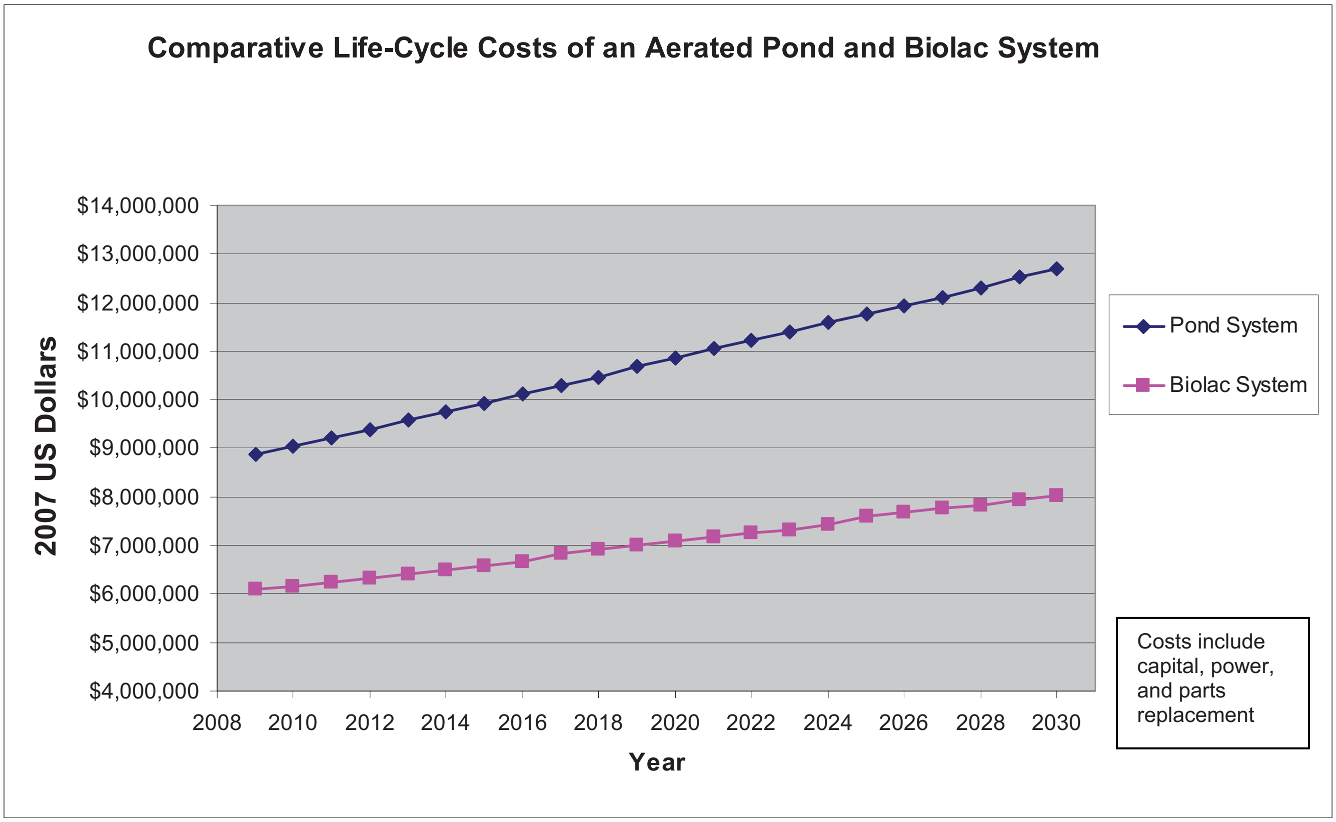



**Figure 8-5 Comparative Life-Cycle Costs of an Aerated Pond System and a Biolac® System**

### Activated Sludge

Activated sludge systems are constructed in various configurations, but three basic components are necessary: 1) a reactor for suspension and aeration of microorganisms, 2) primary and secondary clarifiers for liquid-solid separation, and 3) a system to recycle activated sludge from the secondary clarifier to the reactor influent[5]. The basic process flow diagram is shown as Figure 8-6.

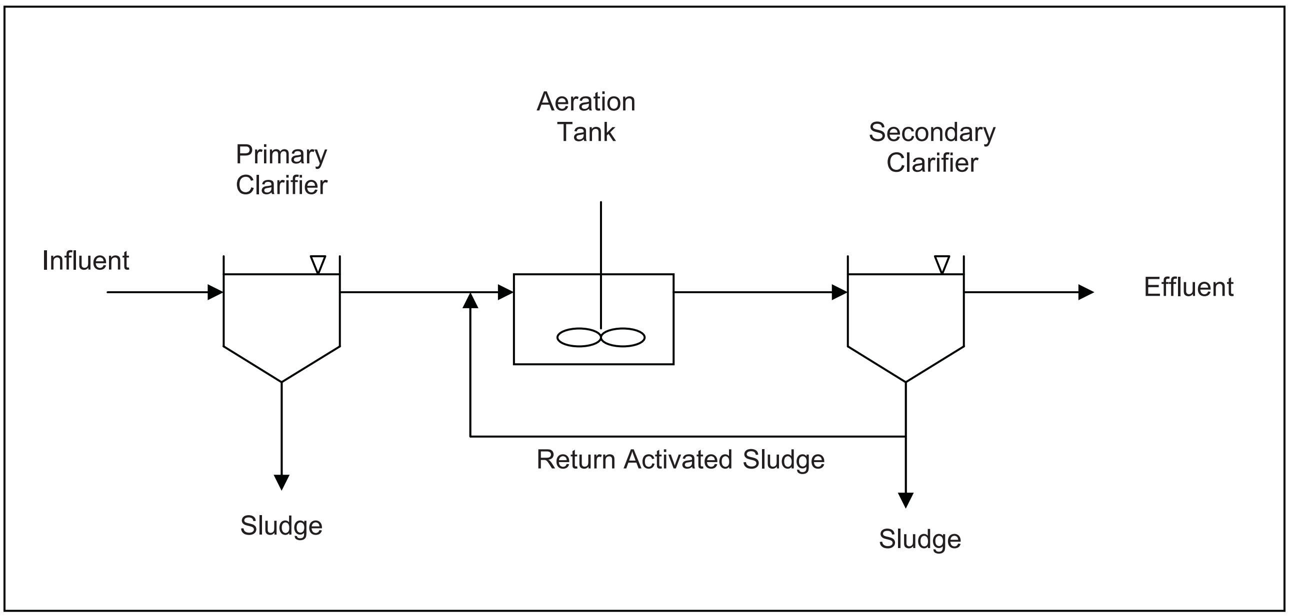



**Figure 8-6 Completely mixed activated sludge process flow diagram**

A typical system for projected 2030 flows would include two primary clarifiers, each with a 40-foot diameter, two aeration basins with a total volume of approximately 52,000 cubic feet (0.4 MG), two secondary clarifiers with 40-foot diameters, and a return activated sludge system. Some advantages to activated sludge include the small footprint, and the option to modify for nitrification, should a higher quality effluent be desired. It delivers a higher quality effluent than the existing aerated ponds. The main disadvantages are the high capital cost, mainly due to concrete and earthwork, and a relatively high operating cost, because of aeration requirements. Denitrification requires additional steps and recycling and may require the addition of a carbon source, such as methanol. Though operation and control is similar to the Biolac system discussed above, upsets in the microbial balance can

[5] George Tchobanoglous, et al. Wastewater *Engineering Treatment and Reuse,* 4th Edition. Tate McGraw-Hill Publishing Company Limited: New Delhi (2005).

cause operational problems like sludge bulking or foaming more frequently than expected with Biolac. The relative footprint for an activated sludge system is shown in Figure 8-7.

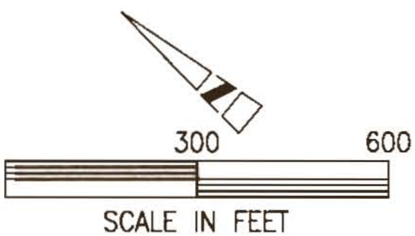
## 8.3 Oxidation Ditch

An oxidation ditch is a ring-shaped channel equipped with aeration and mixing devices. Influent wastewater is mixed with return activated sludge in an anoxic chamber to accomplish biological nutrient removal (nitrogen). The design mimics the kinetics of a completely mixed reactor in the aerated sections, with plug flow along the channels. The aeration zone, located at a turn in the channel, provides oxidation of BOD and ammonia and establishes constant flow, driving the mixed liquor along the channels. As wastewater leaves the aeration zone, oxygen concentrations decrease and denitrification occurs. The process flow diagram for this option is included as Figure 8-8 and the relative footprint is shown in Figure 8-7.

The Eimco Carrousel® System is an example of a closed loop oxidation ditch reactor. The configuration is custom designed based on influent characteristics, and aeration and effluent requirements. Aerators are placed in such a way as to ensure solids suspension in the entire channel. The Eimco Excell™Aerator incorporates a surface aerator on a common shaft with a lower turbine. The system is designed to be able to draw only 15-30 % of the nameplate power and maintain sufficient mixing throughout the channel. This allows for the build-out design to save energy during low influent loadings. Oxidation ditches provide a higher quality effluent than aerated ponds and can handle fluctuating loads. Disadvantages include the high capital cost due to the great amount of concrete required and relatively expensive equipment.

**Table 8-1 Cost Opinion and Relative Size for Future Treatment Options**

| Improvement Option | Total Capital Cost (2008 US $) | Total Estimated Footprint (acre) |
|---|---|---|
| **Treatment Processes** | | |
| Additional Aeration Ponds (4) | $8,680,000 | 7.8 + |
| Biolac® Wave Oxidation System | $6,014,000 | Within 2 existing secondary ponds |
| Eimco Carrousel 3000<br>+ 2 secondary clarifiers | $7,417,000 | 0.45 |
| Activated Sludge<br>+ primary & secondary clarifiers | $8,640,000 | 0.23 |

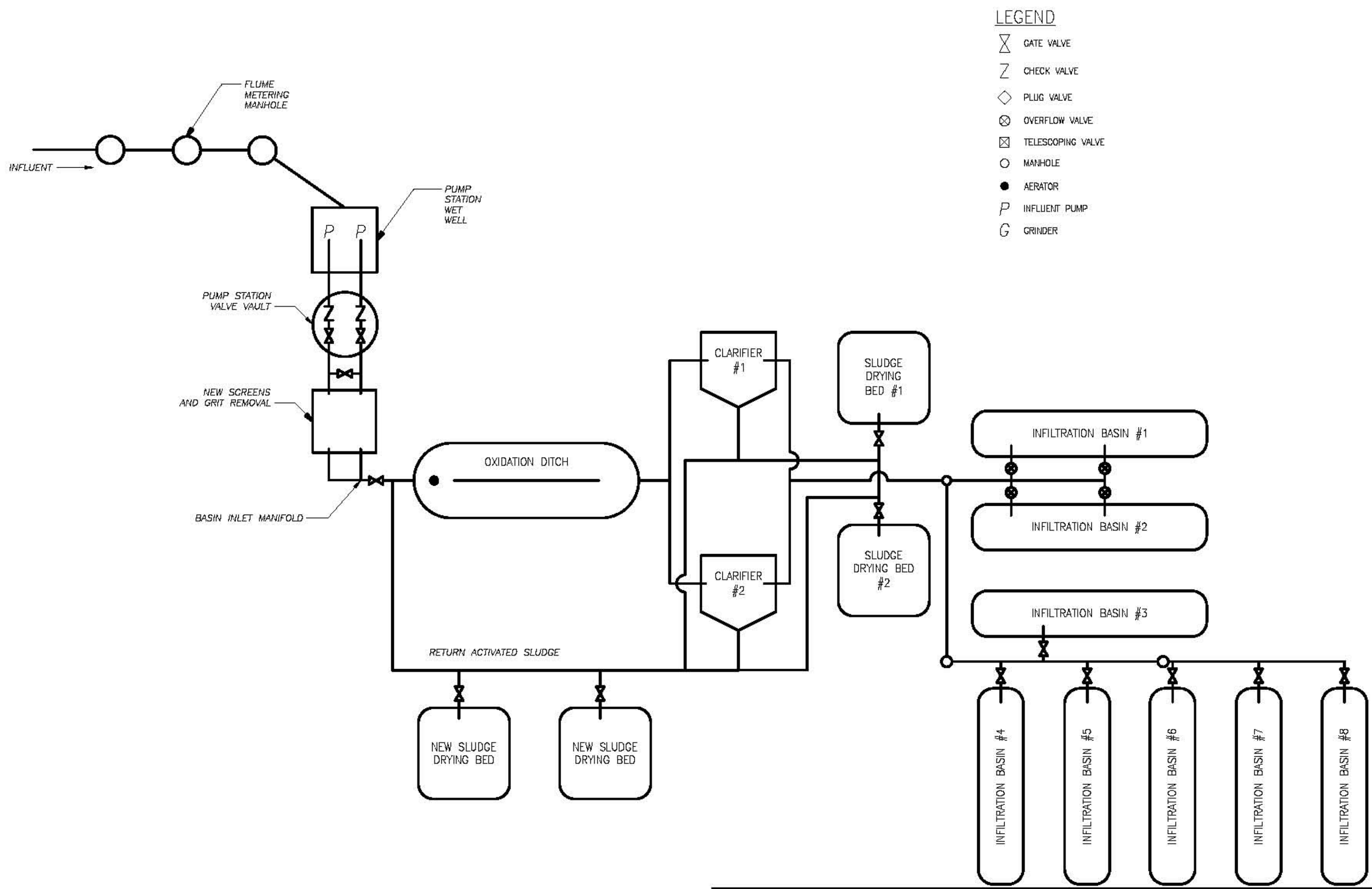




## LEGEND

OXIDATION DITCH SYSTEM

ACTIVATED SLUDGE SYSTEM

| FIGURE | NCSD SOUTHLAND WWTF MASTER PLAN | BEC PROJECT NO. | BOYLE ENGINEERING CORPORATION |
|---|---|---|---|
| 8-7 | ALTERNATES 3 & 4:<br>SITE PLAN FOR OXIDATION DITCH & ACTIVATED SLUDGE SYSTEMS | 19996.17 | 1194 Pacific St., Suite 204 San Luis Obispo, CA 93401 Tel. 805-542-9840 Fax 805-542-9990 WWW.BOYLEENGINEERING.COM |

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**NCSD SOUTHLAND WWTF MASTER PLAN**

OXIDATION DITCH PROCESS FLOW DIAGRAM

BEC PROJECT NO. 19998.17

FIGURE 8-8

## 8.5 Tertiary Treatment

The level of treatment will be dictated by water quality goals and regulations and the decided end use, as discussed in Section 6.0. Three end uses are proposed: unrestricted urban reuse (irrigation of parks), groundwater recharge reuse, and percolation (the current disposal method). The two reuse options will require tertiary treatment (coagulation, filtration, and disinfection) to meet Title 22 and additional regulatory requirements. Under the existing WDR, the current disposal method does not require tertiary treatment. However, the current trend in water quality regulations suggest a higher quality effluent and/or groundwater monitoring may be required to demonstrate that groundwater is not being negatively impacted at some point in the foreseeable future. Alternatives for filtration and disinfection were investigated and are discussed below. A detailed cost opinion is included in Appendix C, and Appendix D contains additional product information for the filtration and UV systems.

In order to provide relatively constant flows to the tertiary treatment systems discussed below, it is assumed the upstream treatment process will provide flow equalization in order to limit short-term peak flows (such as the PHF) to the peak day flow (PDF). Pumping facilities to transfer pond effluent to the filters would likely be required for either alternative, and are included in the cost opinions.

<u>Filtration</u>

Either filtration option would require coagulant feed and mixing equipment upstream of the filters for compliance with Title 22 requirements. It is assumed that coagulant feed and mixing facilities would cost approximately $100,000 for 2030 design flows.

Option 1: Advanced Sand Filtration (Parkson Dynasand)

The Dynasand filtration system consists of upflow, modular sand filters with integral backwash. The internal wash system does not require backwash pumps or wash water storage tanks, reducing energy costs, the need for clean water storage, and the system footprint. Each filter is continuously backwashed, eliminating the need for downtime to clean the filters. Dynasand filters have been approved for Title 22 compliance.

To meet 2030 PDF, a minimum of 10 modules are needed. Therefore, we recommend 6 filtration cells with 2 modules per cell. This way one cell could be taken offline at a time without exceeding the maximum allowable loading rate (5 gpm/ft$^2$) for Title 22 compliance. Arranging the cells in 2 columns with 3 rows, the total approximate footprint would be 45 feet long by 15 feet wide. The estimated capital cost is approximately $2,780,000. Construction could be phased with flow demand.

Option 2: Rotating Disk Filtration (Aqua-Aerobic Aquadisk)

The Aquadisk rotating disk filter system uses nylon pile cloth media. Backwashing occurs at a predetermined water level or time without interrupting treatment. Filters arrive completely assembled in a stainless steel tank. Each unit includes a vacuum backwash, a hopper-bottom tank, a solids removal manifold system, and a fully automatic PLC-based control system. Two 10-disk filters are recommended to provide 100% redundancy. The system was sized to meet 2030 PDF. Each unit is approximately 10 feet wide, 20 feet long and 10 feet high. The estimated capital cost for the system is approximately $2,020,000.

<u>Disinfection</u>

Option 1: Chlorine Contact Basin

For chlorine disinfection, 90-minutes of contact time (at PDF) is required to meet Title 22 standards. To provide this level of treatment, the basin will need a volume of 27,900 $ft^3$. We recommend two parallel channels for redundancy and ease of maintenance. Chlorine dosing and monitoring equipment will be needed. The dosing can be paced off the influent flow meter. The estimated capital cost for a chlorine disinfection system is approximately $1,750,000.

Option 2: UV Disinfection

The Trojan UV3000 Plus™ is a reliable and proven disinfection system that uses low pressure, high output variable power amalgam lamps. The system was designed with an emphasis on dependable performance and simplified maintenance. It is equipped with an automatic chemical/mechanical cleaning system, called ActiClean™, consisting of submersible wiper assemblies with on each UV module. ActiClean™ maintains 95% sleeve transmittance and works while the system is in operation, eliminating the need to go offline for cleaning. To meet design flow for 2030, a system with five banks (four duty, one redundant) is recommended, with nine 8-lamp modules per bank, for a total of 360 lamps. The total estimated capital cost for this option is approximately $4,550,000.

### 8.6 Solids Handling

The additional biological activity of any of the extended aeration processes discussed (Biolac®, oxidation ditch, or activated sludge) provides a higher level of treatment and produces a greater volume of sludge than the existing aerated pond system. This will require additional storage space for solids handling. If the District pursues activated sludge or oxidation ditch treatment, all of the existing aerated ponds will be available and could be used for sludge treatment and storage.

A Biolac system retrofit (least capital cost option) will leave the two primary ponds for use. Odor control can be provided by maintaining an aerated, 2- to 4-foot depth of water over the sludge. This would require the installation of two (2) 10-hp brush aerators in each pond. We recommend the District budget for approximately $100,000 ($50,000 per pond) for aerators and other miscellaneous equipment needed to convert the primary ponds to sludge lagoons.

The sludge produced from a Biolac system at Year 2030 conditions was calculated as an example. Biolac typically yields 0.6 pounds of solids per pound of BOD removed. Assuming the influent $BOD_5$ concentration is equal to the average $BOD_5$ concentration (265 mg/L), TSS is 265 mg/L (70% as fixed solids), and Biolac reduces $BOD_5$ to 5 mg/L, approximately 4700 pounds of sludge would be produced per day during average flow conditions. Assuming 2% solids, the volume of sludge produced would be approximately 3770 $ft^3$ per day. Over time, it is expected that the sludge concentration in the ponds would compress, resulting an average of 6% solids (assuming negligible anaerobic degradation of sludge).

At 2% solids, with three feet for freeboard each primary pond has a total volume of 424,000 $ft^3$, providing a minimum of 110 days of storage each (approximately 7 months total). If solids reach 6% within the first year of storage, the ponds may store approximately 1.3 years of sludge at 2030 flows. It is assumed the sludge would be removed by a portable pump and conveyed through onsite sludge piping to the District's sludge drying beds.

Although the District has used the existing drying beds successfully for many years, we recommend upgrading them. The beds are not lined, and any infiltration through the bottom of the beds could contribute to groundwater degradation. In addition, the beds will be used more regularly in the future and should be lined with concrete to allow vehicles and equipment to work in the ponds without getting stuck. Therefore, initially (during construction of the Phase I Biolac improvements – in the next 2 years) we recommend lining the ponds with concrete and installing a decanting pump station for dewatering the beds and conveying supernatant back to the plant's headworks for treatment. This will provide the District with maximum use of their drying beds, by regularly removing any liquid volume from the ponds and leaving more volume for receiving sludge from the holding ponds. Actively "working" the sludge in drying beds can remove 50-75% of the water from the sludge. At 2030 demands, one year of "dried" sludge (50% solids) would occupy approximately 50% of the proposed drying bed volume, and would require approximately 140 standard 10-cy truck trips for removal. If solids content is increased to 75% through continual compression, raking, and further evaporation, this would be reduced to 70 truck trips.

In the next phase of construction, it is recommended that the District construct two (2) new sludge drying beds by 2017 (simultaneously with Phase II upgrade of the Biolac system to meet 2030 demands) similar in size to the existing beds. All four (4) beds should be connected by common valves and piping from the existing sludge header adjacent to the ponds, and should be connected to the decanting pump station.
Cost opinion for Phases I and II is provided below:

**Table 8-2 Cost Opinions for Sludge Drying Beds**

**Phase I – Modify Existing Sludge Drying Beds**

| Item | Description | Unit | Unit Price | Quantity | Amount |
|---|---|---|---|---|---|
| 1 | Concrete Bed Liner | LS | $600,000 | 1 | $600,000 |
| 2 | Decant Pump Station and Piping | LS | $500,000 | 1 | $500,000 |
| 3 | Engineering/Admin (20% of earthwork) | | | | $220,000 |
| | *Subtotal* | | | | $1,320,000 |
| 4 | Contingency (30% of subtotal) | | | | $396,000 |
| | *Total* | | | | $1,716,000 |

**Phase II – New Sludge Drying Beds**

| Item | Description | Unit | Unit Price | Quantity | Amount |
|---|---|---|---|---|---|
| 1 | Excavation for 2 beds (160' x 200' x 5') | $YD^3$ | $25.00 | 11,860 | $296,500 |
| 2 | Concrete Bed Liner | LS | $600,000 | 1 | $600,000 |
| 3 | Piping (10% of Subtotal) | | | | $90,000 |
| 4 | Engineering/Admin (20% of Subtotal) | | | | $197,300 |
| | *Subtotal* | | | | $1,183,800 |
| 5 | Contingency (30% of subtotal) | | | | $355,140 |
| | *Total* | | | | $1,540,000 |

Note: Totals rounded to nearest $1,000

If odors are a concern in the future, the District should explore various sludge treatment processes such as belt press filtration and/or centrifuge to reduce volume prior to storage in the drying beds.

## 8.7 Wastewater Disposal

Various end-use options for treated wastewater were discussed in Section 6.0: reuse as irrigation for parks, groundwater recharge reuse, and maintain onsite percolation for filtration and potentially for seasonal storage before transporting offsite for infiltration or other reuse. The Revised Draft Preliminary Screening Evaluation of Southland WWTF Disposal Alternatives (Boyle, November 2008) further discusses potential disposal and reuse alternatives. If the District chooses to continue onsite percolation as a wet-weather disposal or secondary disposal method, additional infiltration basins will likely be needed, especially if additional aeration ponds are built as the future treatment alternative. Table 8-3 shows the approximate costs to construct three new infiltration basins. As discussed in previous sections of the report, percolation capacity of the site must be evaluated. At least three basins (approximately 110 ft by 650 ft) could fit on the District's property without requiring additional land.

**Table 8-3 Cost Opinion for Infiltration Basins**

| Item | Description | Unit | Unit Price | Quantity | Amount |
|---|---|---|---|---|---|
| 1 | Excavation for 3 basins (110' x 650' x 5') | $YD^3$ | $20.00 | 39,730 | $794,600 |
| 2 | Piping (10% of earthwork) | | | | $79,460 |
| 3 | Engineering/Admin (20% of Subtotal) | | | | $174,840 |
| | *Subtotal* | | | | $1,048,900 |
| 4 | Contingency (30% of subtotal) | | | | $314,700 |
| | *Total* | | | | $1,363,000 |

## 8.8 Removal of Sludge from Drying Beds during Construction

In a November 30, 2007, Technical Memorandum (Appendix E), Boyle evaluated various options for long-term sludge management at Southland and Blacklake WWTFs. The Memorandum developed costs for hauling sludge to a landfill, San Jose Composting (Kern County) or to Engel & Grey (Santa Maria).

Removing sludge from the drying beds will present a significant cost during construction. Assuming the existing drying beds (approximately 50,800 square feet of surface area) have depths of 5 feet or 8 feet of sludge, we would expect to have 254,000 cubic feet or 406,000 cubic feet of sludge. If the average density is in the range of 10-30% solids at a specific gravity of 1.06, we would anticipate the volumes are equivalent to 1,100 and 1,800 tons, respectively.

It is unlikely a composting facility will take these solids since there is no grit removal or screening at the plant, but the landfill might take them. Since landfill and composting facilities' policies may change in the next year, it is recommended that this analysis be reviewed and revised prior to beginning plant construction.

The budget numbers summarized below are considered to be an adequate, current planning-level cost for hauling solids to a landfill. Reducing volume by drying these solids will decrease hauling and tipping costs:

- Excavation of Sludge (5-ft Depth) = $100,000 ($10 per cubic yard)
- Excavation of Sludge (8-ft Depth) = $150,000 ($15 per cubic yard)
- Total Tipping and Hauling Cost per Truck Load = $1,500 ($1330 from 2007 Technical Memorandum with 10% Escalation)
- Total Sludge Disposal Cost (5-ft Depth = 45 Loads) = $170,000
- Total Sludge Disposal Cost (8-ft Depth = 72 Loads) = $260,000

## 8.9 Alternative Energy Supply

The District is interested in pursuing alternative energy to provide power for the expanded Southland WWTF. A proposal received from SPG Solar (See Appendix F) described a 500-kW solar array that could be placed on a 3.5 acre area adjacent to the existing plant. If implemented, the SPG project would cost approximately $4,010,000 in capital cost or a Power Purchase Agreement could be executed between the District and SPG Solar to provide approximately 1,000,000 kWh/yr at around $0.11/kWh with 3% annual escalation or $0.105/kWh with 4% annual escalation. The SPG proposal does not include site preparation, fencing, lighting, drainage, or other improvements beyond installation of the solar arrays and electrical conduits to the plant's control center.

Although this proposal is included for budgetary purposes, an evaluation of solar power alternatives should be performed prior to implementing a project. No analysis has been performed on the SPG Solar proposal and it is unknown if it would be appropriate for providing power to the proposed treatment project.

## 8.10 Recommendations

The WWTF will require an upgrade to handle future demands. Several processes were evaluated. When compared to the aerated pond system, a Biolac® system can provide a higher level of treatment at a lower capital and operating cost. It requires a higher degree of operator involvement than the current system, but routine operations and maintenance are less complex than the other, more expensive treatment technologies reviewed herein (oxidation ditch and activated sludge).

We recommend installing sufficient aeration capacity to meet 75% of 2030 demands in Phase I of plant upgrades, as well as lining the existing sludge drying beds and installing a decanting pump station [6]. Ponds 3 and 4 should be relined and retrofit with Biolac wave oxidation systems and clarifiers should be constructed. The existing primary ponds should be used for onsite sludge storage and anaerobic reduction prior to drying.

Phase II would involve upgrading the Biolac system capacity to meet 2030 demands and installing two additional lined sludge drying beds.

Three (3) infiltration basins, similar in size to the existing ponds, could fit on the existing WWTF site. The ultimate capacity of the existing and new ponds should be determined so the District can decide whether to use the onsite infiltration basins as filtration and potentially "wet-weather" storage prior to offsite infiltration or some reuse alternative.

[6] Phase I improvements meet Scenario 1 demands from the Water and Sewer Master Plan Update (Cannon Associates, December 2007).

# 9.0 CONCLUSIONS & RECOMMENDATIONS

## 9.1 Conclusions

The Southland WWTF is approaching the permitted capacity (MMF = 900,000 gpd). Flowrates could reach this limit as early as 2010 and the WWTF is expected to exceed effluent quality limits (average monthly $BOD_5$ = 60 mg/L) in 2011 during high flow conditions. An upgrade is required to handle future demands and water quality goals. The District should work with RWQCB to develop a phased approach to upgrading the Wastewater Treatment Facility. A schedule for this work is outlined in Section 10.0.

Water quality goals will dictate future plant process improvements. Usage options include groundwater recharge, direct reuse (irrigation), and offsite infiltration. Based on conversations with RWQCB staff, and review of Basin Plan criteria, more stringent discharge requirements to eliminate impacts on groundwater are inevitable. These requirements may include nitrogen limits and possibly salts limits in the future. The existing treatment process is not adequate to meet water quality goals that are more stringent than the current discharge requirements, including requirements for tertiary treatment (for park/school irrigation) or pretreatment requirements for future salts removal if required.

An examination of existing and future hydraulic demands on the system revealed deficiencies as discussed below:

- The capacity of the Frontage Road trunk main is inadequate for existing conditions;
- The influent pumps can meet projected flow demands through 2015, however the wetwell is undersized for existing demands and may cause excessive motor wear. The influent pump station will not meet 2030 demands.
- The plant is nearing its rated capacity, and could exceed permitted flow limits by 2010, according to the flow projections presented in this report.

Four alternatives were evaluated for the WWTF treatment upgrade: additional aerated ponds, Biolac® wave oxidation system, oxidation ditch, and conventional activated sludge. The first option is an extension of the current treatment process at the plant. The following three are variations of activated sludge technology, which provides a higher quality effluent and a basis for tertiary treatment. The Biolac system provides extended aeration at a lower cost than any of the other three alternatives examined. Life cycle costs are approximately half that of a pond system. Additional treatment can be easily added to the process train, providing flexibility for the potential of tertiary treatment.

## 9.2 Recommendations

As discussed in previous sections, we recommend the following as a result of our analysis in this Master Plan:

- Begin planning and permitting efforts for a wastewater treatment plant expansion as soon as possible;
- The District should consult with RWQCB to acquire either interim adjustment to effluent limits, or to permitted flows, during planning and design of a treatment facility expansion. They should also seek RWQCB support on the recommendations and schedule presented in this Master Plan. Details are discussed in Section 8.0.
- If reuse is an option, a user survey should be conducted to see if a viable market for irrigation is available. (See Revised Draft Preliminary Screening Evaluation of Southland WWTF Disposal Alternatives, ibid, for additional discussion).
- Since expansion of percolation area may be required on an interim basis, regardless of future reuse opportunities, we recommend assessing available onsite percolation capacity and evaluating groundwater conditions beneath the plant.
- Screening and grit removal systems will improve treatment and reduce wear on system components. We recommend installing two (2) shaftless screw screens and two (2) vortex-type grit removal vaults.
- Sludge in the drying beds will need to be removed before construction. As discussed in Section 8.8, volume is estimated between 254,000 and 406,000 $ft^3$ and the weight between 1,100 and 1,800 tons, respectively.
- Biolac® is the recommended wastewater treatment process based on capability to meet more stringent discharge limits; nitrogen removal capabilities; low level of complexity compared with activated sludge systems; and low capital/lifecycle costs compared with the other alternatives evaluated herein. Ponds 3 and 4 should be relined and retrofitted with the Biolac wave oxidation system. External clarifiers will also be required. The system should be constructed in two phases – Phase I would provide 75% of the 2030 capacity [7], and Phase II would meet 2030 demands.
- The District should have a Class II Operator managing the Biolac system.
- The primary treatment ponds should be converted to aerated sludge holding lagoons.
- The two existing drying beds should be lined and a decanting pump station should be provided. Two additional drying beds should be constructed to meet 2030 solids handling demands. If odors become a concern in the future, due to increase in development around the plant site, more rigorous solids processing may be required.

[7] Phase I improvements meet Scenario 1 demands from the Water and Sewer Master Plan Update (Canon Associates, December 2007).

# 10.0 RECOMMENDED CAPITAL IMPROVEMENTS PLAN & OPINION OF PROBABLE COST

The analysis presented in the previous sections addresses improvements required to meet existing demands, as well as future demands and water quality goals. Major capital improvements can be separated into two categories:

- Facility Improvements: Those projects which would improve plant operability without requiring major process improvements.
- Future Process Improvements (Schedule TBD): Process and capacity improvements to meet anticipated future water quality goals and demands through 2030. While the first phase of the Biolac® system should be installed before the plant reaches its permitted capacity (0.9 MGD), the tertiary treatment and disinfection improvement schedule would be dictated by future permitting limits and/or recycling opportunities.

A 4% annual cost escalation factor was applied to the 2008 project costs summarized below.

**Table 10-1 Conceptual Cost Opinions for Facility Improvements**

| Component | 2008 Project Cost | Year to be Completed | Escalated Project Cost to Midpoint of Construction |
|---|---|---|---|
| Frontage Rd. Trunk Main 21" Upgrade | $2,182,000 | 2011 | $2,361,000 |
| Influent Pump Station and Flowmeter Improvements | $967,000 | 2011 | $1,046,000 |
| Spiral Screening System | $512,000 | 2011 | $554,000 |
| Grit Removal System | $629,000 | 2011 | $681,000 |

Nov 2008 ENR (CCI) = 8602 in all Cost Opinions

Table 10-1 includes the Frontage Rd. Trunk Main Upgrade, which will remedy existing hydraulic deficiencies in the pipeline; Screening and Grit Removal Systems, as requested by District staff to improve operability of the plant and improve pond performance; and the Influent Pump Station and Flowmeter Improvements. Although the existing pump station capacity is adequate through 2015, as discussed in Section 7.0, it is recommended that this project be installed at the same time as the Frontage Road Trunk Main project since both will require deep excavations (greater than 20 ft depth), bypass pumping, and could be more efficiently constructed as one project.

**Table 10-2 Conceptual Cost Opinions for Process Improvements [8]**

| Component | 2008 Project Cost | Year to be Completed | Escalated Project Cost to Midpoint of Construction |
|---|---|---|---|
| Phase I Biolac System (Capacity = 1.4 MGD MMF, or 75% of 2030 Demands) | $5,734,000 | 2011 | $6,204,000 |
| Phase I Drying Bed Improvements | $1,716,000 | 2011 | $1,857,000 |
| Phase II Biolac System (Capacity = 1.8 MGD MMF, or 100% of 2030 Demands) | $280,000 | 2017 | $308,000 |
| Phase II Drying Beds (2 New) | $1,540,000 | 2017 | $2,108,000 |
| Percolation Ponds | $1,363,000 | 2017 | $1,865,000 |
| Tertiary Filtration | $2,016,000 | TBD | -- |
| Chlorination System | $1,748,000 | TBD | -- |
| Solar array for alternative energy (see proposal App E) | $4,010,000 | TBD | -- |

Table 10-2 includes construction of the wave oxidation system and integral clarifiers in the existing secondary ponds in phases. The project cost summaries in Section 8.0 include a cost of $4,473,000 for a complete wave oxidation system with adequate capacity through 2030. Phase I would involve liner replacement, installation of aeration lines, and construction of new clarifiers in each of the secondary ponds. This improvement should be accomplished within the same timeline as the headworks improvements (recommended as part of the same project) since the plant currently treats 0.64 MGD on a maximum month basis, with a permitted MMF capacity of 0.90 MGD. Diffusers would be installed to meet a capacity of 75% of 2030 Demands (approximate to projected 2020 Demands). Phase II would include installation of additional diffusers and an additional blower to meet 2030 Demands.

Blowers/Aeration: Although blower condition was not assessed in detail in this study, the existing blowers may be capable of supporting aeration demand for the first few years of operation. This should be explored during preliminary facility design. However, cost for new blowers was included in the project cost opinions for planning purposes.

Solids Handling Facilities: At the same time the Phase I Biolac project is constructed, we recommend converting the existing primary treatment ponds to aerated sludge holding lagoons, lining the District's existing drying beds,

[8] Phase I improvements meet Scenario 1 demands from the Water and Sewer Master Plan Update (Canon Associates, December 2007).

and constructing a decanting pump station.  Two additional drying beds would be installed if needed prior to 2015, or in conjunction with the Phase II Biolac expansion in 2015.

If odors become a concern near the plant site, additional solids handling facilities (such as a centrifuge or belt press) may be required to process sludge before storing or drying it onsite.

Disposal or Reuse Option:  Evaluating potential discharge, percolation, or reuse opportunities will require further investigation by the District.  Currently, the District is investigating potential recharge and reuse opportunities through the Revised Draft Preliminary Screening Evaluation of Southland WWTF Disposal Alternatives (ibid).

# 11.0 BASIS FOR ASSESSMENT OF RATES & FEES

The objectives of this section are 1) to establish guidelines for determining the value of the existing facilities at Southland WWTF that would remain in service for future demands, and 2) to recommend a cost allocation strategy for existing ratepayers and new development to assist in funding Phase I WWTF improvements. [9]

<u>Use of Existing Facilities</u>

The process schematic of the existing WWTF is included as Figure 4-1. If the Biolac® System is installed, all the basins, drying beds, and percolation ponds will remain in service with the recommended upgrades. However, the influent trunk main, flow meter, and pump station will be replaced. The grinder will be replaced with screening and grit removal systems in order to reduce the amount of solids in the influent and resulting wear on equipment.

The recommended process improvement, a Biolac® system, utilizes two of the four existing aerated ponds as basins (the two larger, secondary aeration ponds). The flow diagram and site plan (with the existing facilities in gray) for the Biolac® retrofit are included as Figures 8-3 and 8-4. With this alternative, the mechanical aerators will be replaced with a Wave Oxidation™ system and clarifiers. Existing aeration piping will be abandoned or removed. The District will be able to use the blower building and three existing blowers, but may need to add or replace some in the future as demand increases. The Biolac® upgrade is recommended in phases as discussed in Section 10.

With increased biological treatment of any extended aeration processes, a greater amount of sludge will be produced than is currently generated. The two existing primary aerated ponds would be operated as sludge holding lagoons to provide treatment and storage. The aeration system will need to be removed and brush aerators will be added to maintain an aerated layer of water over the sludge.

The two existing sludge drying beds will continue in service. In order to meet increased demands, we recommend adding concrete liners and a decanting pump station for dewatering the beds and conveying the supernatant back to the plant's headworks for treatment. This retrofit is recommended to coincide with the Phase I Biolac improvements (see Table 11-2). During the second phase of construction, two new drying beds should be installed to ensure storage and dewatering capacity for buildout demands.

The WWTF currently uses onsite infiltration basins for final treatment and disposal of the effluent. Continued onsite percolation is assumed in this report, but pending studies and future policy direction regarding wastewater

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[9] Though Phase II improvements are discussed in this report, the cost-sharing strategy was developed only for Phase I at this time based on direction from the Board during the April 11, 2007 NCSD Board Meeting.

reuse and disposal may require additional plant improvements. An analysis is currently underway to investigate the potential impacts to groundwater and the District is exploring sites for groundwater recharge. A survey to identify prospective users of reclaimed wastewater is recommended, as well.

Cost-Sharing Strategy

Nearly all the recommended improvements have two objectives: meet existing demands, and handle anticipated demands from future development. To assist the District in developing a cost-sharing strategy for the Phase I WWTF improvements, each project cost is separated into two funding categories: existing customers and future development, as shown in Table 11-1.

Table 11-1 Recommended Funding Allocation

| Demands | AAF (mgd) | Percentage |
|---|---|---|
| Existing | 0.59 | 47 % |
| Future Development | 0.66 | 53 % |
| Total Phase I Capacity | 1.25 | 100 % |

The project costs are then divided between existing ratepayers and future development based on relative capacity.

Table 11-2 Proposed Cost-Sharing for Recommended Phase I WWTF Improvements

| Component | 2008 Project Cost | Year to be Complete | Escalated Project Cost to Midpoint of Construction [1] | % Capacity for Existing Users | Cost for Existing Users[2] (midpoint of construction) | % Capacity for Future Development | Cost for Future Development (midpoint of construction) |
|---|---|---|---|---|---|---|---|
| FACILITY IMPROVEMENTS | | | | | | | |
| Frontage Rd. Trunk Main 21" Upgrade | $2,182,000 | 2011 | $2,361,000 | 47 | $1,115,000 | 53 | $1,247,000 |
| Influent Pump Station and Flowmeter Improvements | $967,000 | 2011 | $1,046,000 | 47 | $494,000 | 53 | $553,000 |
| Spiral Screening System | $512,000 | 2011 | $554,000 | 47 | $262,000 | 53 | $293,000 |
| Grit Removal System | $629,000 | 2011 | $681,000 | 47 | $322,000 | 53 | $360,000 |
| PROCESS IMPROVEMENTS | | | | | | | |
| Phase I Biolac System (Capacity = 1.4 MGD MMF, or 75% of 2030 Demands) | $6,014,000 | 2011 | $8,229,000 | 47 | $3,885,000 | 53 | $4,345,000 |
| Phase I Drying Bed Improvements | $1,716,000 | 2011 | $1,857,000 | 47 | $877,000 | 53 | $981,000 |
| Percolation Ponds | $1,363,000 | 2017 | $1,865,000 | 47 | $877,000 | 53 | $988,000 |

[1] Cost is escalated using a 4% annual cost escalation.
[2] Percent capacity is determined by ratio of flow demands for existing users to total future demand.