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November 1, 2007

Nipomo Community Services District
148 Wilson Street
P.O. Box 326
Nipomo, CA 93444

(805) 929-1133 Phone
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Dear Bruce Buel:

I am requesting a copy of the following study:

Southland Wastewater Treatment Facility Master Plan (Boyle Engineering, Draft February 2007) or later version.

I would prefer a CD Disk over a paper copy.

Thank You

Harold Snyder

Email Delivered.

NIPOMO COMMUNITY SERVICES DISTRICT

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November 9, 2007

Mr. Harold Snyder
P. O. Box 926
Nipomo, CA 93444

SUBJECT: NOVEMBER 1, 2007 PUBLIC RECORDS REQUEST RE SOUTHLAND WWTF

Dear Mr. Snyder,

Attached is a diskette containing the full text of Boyle Engineering's February 2007 Southland WWTF Master Plan per your request.

If you have any questions, please don't hesitate to call me.

Sincerely,

NIPOMO COMMUNITY SERVICES DISTRICT

Bruce Buel
General Manager

CC: Public Records Request File
Chronological File

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Southland Wastewater Treatment Facility Master Plan - DRAFT

Nipomo Community Services District

District General Manager	Bruce Buel

Boyle Engineering Corporation

Project Manager	Mike Nunley, PE
Project Engineers	Ron Abraham, PE
	Eileen Mick, EIT

19996.17

Revised February 19, 2007

BOYLE 1194 Pacific Street, Suite #204 San Luis Obispo, CA 93401

Executive Summary

Introduction

The Nipomo Community Services District (District) owns and operates the Southland Wastewater Treatment Facility (WWTF), which treats a combination of domestic and industrial wastewater from the community of Nipomo, California. The WWTF has a permitted capacity of 900,000 gallons per day (gpd) based on the maximum monthly demand. Wastewater is treated by four aerated ponds and discharged to onsite infiltration basins.

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. This is the third of a series of reports Boyle performed in response to the NOV (following the Action Plan, May 2006, and Technical Memorandum, July 2006). This report comprises the WWTF Master Plan, which was prepared to assist in the strategy for future capital improvements.

The purpose of the Master Plan is to evaluate existing and future demands of the WWTF, identify the needed improvements to meet these demands, and develop a capital improvements program to assist the District in planning.

Existing Loads

Monitoring data from the previous two years (September 2004 to August 2006) were analyzed to determine flow demands, peaking factors, loading rates, and solids production. Several flow rates were analyzed and loading rates were determined. Inflow and infiltration was investigated, but did not appear to significantly contribute to plant flows. Table ES-1 summarizes the peaking factors established.

Table ES-1 Summary of Peaking Factors

Flow Condition	Existing Flow (mgd)	Peaking Factor
Average Annual Flow (AAF)	0.59	--
Maximum Monthly Flow (MMF)	0.79	1.34
Peak Daily Flow (PDF)	2.02*	2.00
Peak Hourly Flow (PHF)	1.77	3.00

* Measured value suspected to be erred due to meter problems and was not used to calculate peaking factor

The loading of organic materials and solids in domestic wastewater are important to establish the process capacity of the WWTF. Influent BOD_5 measurements began in December 2005. The data from December 2005 through August 2006 were used to establish the following:

- Average Daily BOD_5 loading = 1311 lb/day, and
- Maximum Daily BOD_5 loading = 1514 lbs/day.

Projected Loads

Plant records from September 2004 to August 2006 indicate an AAF of 0.59 mgd. Under direction of NCSD staff, this study used the projected 2030 AAF from the Draft Water and Sewer Master Plan (Cannon Associates) and derived intermediate future AAFs assuming linearized growth between existing and 2030 flow rates. Peaking factors were used to project other relevant flows. Table ES-2 summarizes current and projected future flow rates. According to this conservative growth projection, the permitted capacity (MMF = 0.9 mgd) could be reached by December 2007. The District should begin planning and designing a plant expansion by Spring 2007.

Table ES-2 Projected Flow Rates

Flow Condition	Peaking Factor	Existing Flow (mgd)	Projected Flow (mgd)**				
			2010	2015	2020	2025	2030
Average Annual Flow (AAF)	--	0.591	0.838	1.05	1.25	1.45	1.67
Maximum Monthly Flow (MMF)	1.34	0.791	1.12	1.41	1.68	1.94	2.34
Peak Daily Flow (PDF)	2.00	2.024*	1.68	2.10	2.50	2.90	3.34
Peak Hourly Flow (PHF)	3.00	1.77	2.51	3.15	3.75	4.35	5.01

* Measured value suspected to be erred due to meter submergence
** Projected AAF based on Draft Water and Sewer Master Plan (GTA & Cannon Assoc.)

Projected BOD loads were determined by dividing the existing average annual and maximum monthly BOD_5 concentrations by the AAF and MMF, respectively. This provides the loadings in terms of pounds of BOD_5 per million gallons. These were multiplied by projected flow rates to find projected BOD_5 loadings, shown in Table ES-3.

Table ES-3 Projected BOD_5 Loading Rates

Year	2006	2010	2015	2020	2025	2030
AAF (mgd)	0.591	0.838	1.05	1.25	1.45	1.67
Average Annual BOD_5 Loading (lb/day)	1,311	1,860	2,330	2,770	3,220	3,700
MMF (mgd)	0.791	1.120	1.41	1.68	1.94	2.34
Maximum Monthly BOD_5 Loading (lb/day)	1,514	2,140	2,700	3,220	3,710	4,480

A frequency diagram was created using monitoring results for influent BOD_5 for December 2005 through August 2006. This revealed a 90% frequency value of 350 mg/L. This value is recommended for use in planning and design purposes.

Treatment Capacity

Evaluation of the treatment capacity of the WWTF showed the ability to treat existing influent wastewater under various flow rates and temperature conditions (Table ES-4). However, when projected 2030 flow rates were applied, the plant model did not meet current effluent limits (Table ES-5). If the ponds are operated in two parallel trains of two, the permitted BOD_5 effluent limit is expected to be reached by 2008 during high temperature, high flow conditions according to the conservative growth projections (plant flow limit would be reached prior to that time, according to flow projections). If the ponds are run in series, the permitted BOD_5 limit will be reached in 2010. However, there are potential conditions that may attribute to increased effluent BOD concentrations when running the ponds in series. We recommend referring to the parallel configuration when estimating plant capacity.

Table ES-4 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)	41	36	45
2 Parallel Trains of 2 Ponds [BOD_5] (mg/L)	59	55	64
WDR Effluent BOD_5 limit = 100 mg/L			

Table ES-5 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] with baffle (mg/L)	151	180	135
[BOD_5] without baffle (mg/L)	121	150	105
2 Parallel Trains of 2 Ponds			
[BOD_5] with baffle (mg/L)	162	189	148
[BOD_5] without baffle (mg/L)	135	162	121
Existing WDR Effluent BOD_5 Limit = 100 mg/L			

System Improvements

Several system improvements are identified in the Master Plan to meet hydraulic demands and improve operability of the plant.

- *Frontage Road trunk main replacement*: A hydraulic analysis was performed on Frontage Road trunk main from Division Street to the WWTF. The entire stretch of 12-inch pipeline was found to be undersized for projected future demands, both AAF and PHF, except one section immediately above Story Street where the slope is nearly 3.5 times that of the next greatest slope in the study reach. 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 was examined for hydraulic capacity. Two Fairbanks-Morse submersible pumps were installed in 2000, rated at approximately 2300 gpm each. System and pump curves reveal sufficient pump capacity to handle the current peak hour flow with one pump as a backup. However, an upgrade will be required to maintain 100% redundancy in the future. The current pumps will meet projected demands up to 2015. Analysis indicates that although 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*: The WWTF currently lacks screening or grit removal, with just two grinders to grind large objects ahead of the pump station. 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 types of screens and two types of grit removal systems were compared for the WWTF improvement. Two parallel shaftless screw screens (such as Parkson Helisieve® or equal) are 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.

Treatment Process Upgrade

The WWTF is operating close to its permitted capacity. Plant demands could reach the flow limit (MMF = 0.9 mgd) as early as December 2007 and the effluent BOD_5 limit of 100 mg/L in 2008 during high flow conditions. An upgrade is required. Considering how rapidly demands may meet these limits, the District should begin planning and designing a WWTF upgrade by Spring of 2007 and work with the RWQCB to develop a phased approach for permitting and upgrading the plant.

Water quality goals play a large role in determination of treatment alternatives. Discharge options discussed in this Master Plan include: reuse as irrigation of parks, reuse as groundwater recharge, and onsite infiltration (currently practiced). Both reuse options require tertiary treatment (coagulation, filtration, and disinfection). Infiltration requires the discharger demonstrate no impact to groundwater. Based on conversations with RWQCB staff and review of the Basin Plan, more stringent discharge requirements are inevitable. The existing process will not meet water quality goals that are more stringent than the existing requirements, or act as pretreatment for a tertiary process. Therefore, we recommend the following:

- Sample wastewater effluent for constituents that may effect reuse as irrigation for parks and for groundwater recharge;
- Perform a user survey to determine the potential market for reclaimed wastewater;

- Evaluate the percolation capacity of the existing infiltration basins and potential future infiltration locations on site; and
- Select a treatment plant process that will provide adequate pretreatment for tertiary filtration to protect the District's option for reuse in the future.

Four treatment alternatives were evaluated for the WWTF upgrade: additional aerated ponds, Biolac® wave oxidation system, oxidation ditch, and conventional activated sludge. We recommend the Biolac system because it provides a high quality effluent (sufficient for a tertiary process pretreatment) at a lower cost than any of the other three alternatives examined. Comprehensive life cycle costs are approximately half that of a pond system. It requires a Class II operator to manage, with a higher degree of operator involvement than a pond system, but routine operations and maintenance are less complex than the other, more expensive treatment technologies reviewed (oxidation ditch and activated sludge). We recommend retrofitting Pond 3 and 4 with Biolac® wave oxidation systems and integral clarifiers. Primary ponds 1 and 2 would be converted to aerated sludge holding lagoons. The upgrade could be phased by installing 75% of the aeration equipment required to meet the projected 2030 demands. This is estimated to be sufficient until 2020. Phase II would include installation of additional diffusers and an additional blower.

Solids Handling

We recommend lining the two existing drying beds and installing a decant pumping station concurrently with the Phase I Biolac project. Two additional beds would be constructed with the Phase II Biolac expansion.

Short-Term Performance Improvements and Monitoring

In order to meet the District's wastewater demand while a plant expansion is being planned and designed, we recommend the following steps:

1. Remove the baffles in both Ponds 3 and 4 to provide the maximum volume of treatment capacity within the ponds.

2. Spread the aerators to optimize mixing and aeration within Ponds 3 and 4. However, the outlet should be located outside of the manufacturer's recommended zone of influence around the aerators.
3. Replace the existing floating outlets with flexible outlet pipes that are mounted to a fixed pole or walkway. The outlet could be mounted to the pole by a chain and an adjustable hook.
4. Begin sampling BOD_5, TSS, carbonaceous BOD ($CBOD_5$), soluble BOD ($SBOD_5$), total Kjeldahl nitrogen (TKN), total ammonia, nitrate, temperature, and nitrate in the plant influent and in the effluent from each pond. Samples should be taken on a monthly basis to allow the District to evaluate whether an interim increase in their permitted capacity, or an interim increase in their permitted effluent limits, could be requested from Regional Water Quality Control Board. This would allow more time for the District to expand the treatment facility.

Capital Improvements Plan

A Capital Improvement Plan was developed to assist the District in planning and budgeting for WWTF improvements. Major capital improvements can be separated into two categories:

- Facility Improvements: Those projects which would improve plant operability without requiring major process improvements. Projects under construction by District staff are not included in this list, but are discussed in Section 7.0.
- 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. The cost for constructing three additional percolation ponds is included in these tables, since this would likely be desirable as a secondary or "wet-weather" disposal option even if other reuse opportunities arise. However, the capacity of these additional percolation ponds is unknown and should be evaluated as discussed herein.

A 4% annual cost escalation factor was applied to the 2007 project costs summarized below.

Table ES-6 Conceptual Cost Opinions for Facility Improvements

Component	2007 Project Cost	Year to be Completed	Escalated Project Cost to Midpoint of Construction
Frontage Rd. Trunk Main 21" Upgrade	$2,182,000	2009	$2,361,000
Influent Pump Station and Flowmeter Improvements	$967,000	2009	$1,046,000
Spiral Screening System	$468,000	2009	$507,000
Grit Removal System	$560,000	2009	$606,000

Feb 2007 ENR(CCI) =7880 in all Cost Opinions

Table ES-7 Conceptual Cost Opinions for Process Improvements

Component	2007 Project Cost	Year to be Completed	Escalated Project Cost to Midpoint of Construction
Phase I Biolac System (Capacity = 1.7 MGD MMF, or 75% of 2030 Demands)	$4,060,000	2009	$4,392,000
Phase I Drying Bed Improvements	$1,716,000	2009	2,348,000
Phase II Biolac System (Capacity = 2.4 MGD MMF, or 100% of 2030 Demands)	$198,000	2015	$217,000
Phase II Drying Beds (2 New)	$1,540,000	2015	$2,108,000
Percolation Ponds	$1,363,000	2015	1,865,000
Tertiary Filtration	$1,898,000	TBD	--
Chlorination System	$1,546,000	TBD	--

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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 just 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 2004 to August 2006 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 Draft 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.

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.

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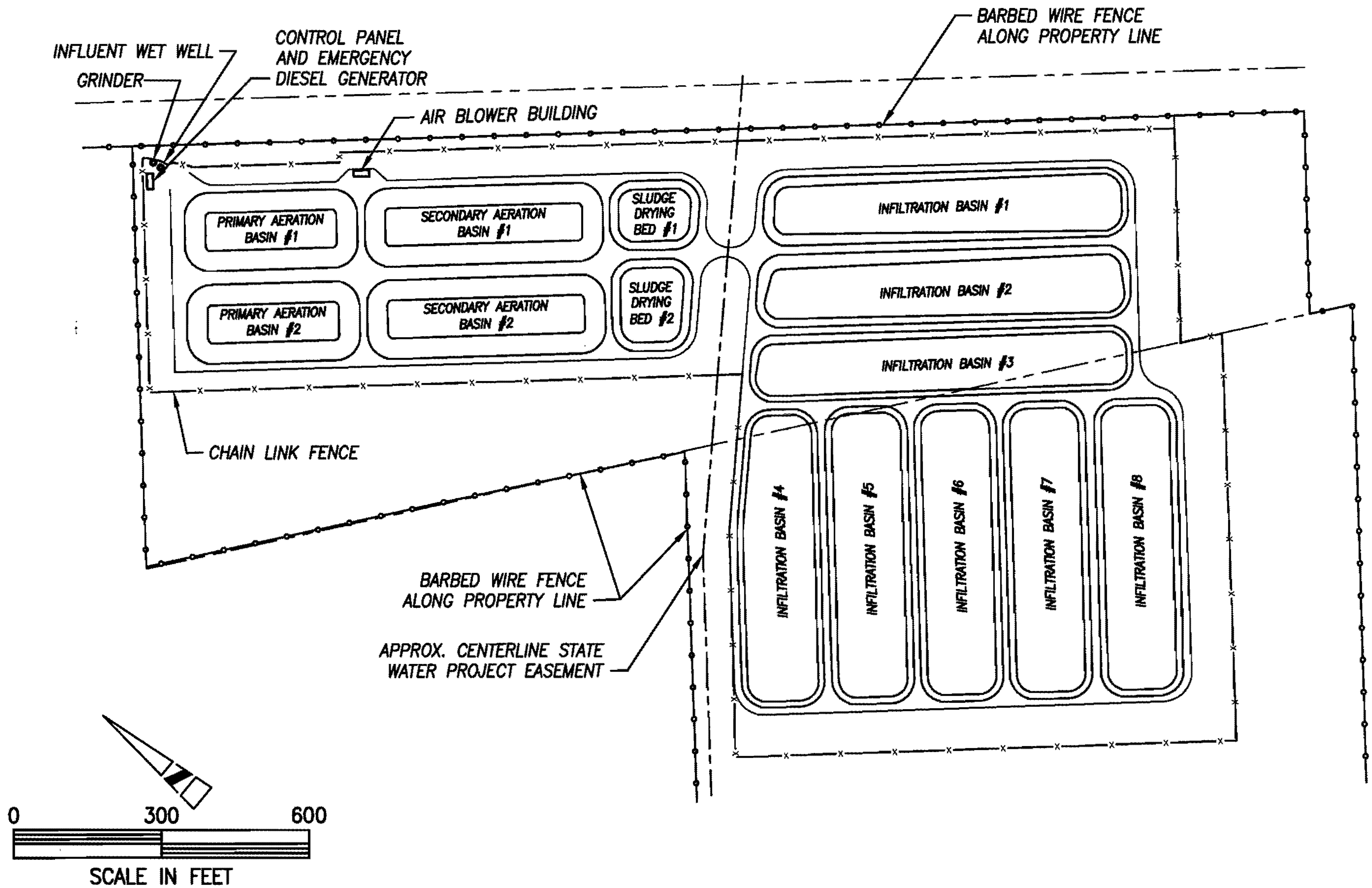


FIGURE 1-1	NCSD SOUTHLAND WWTF MASTER PLAN SITE PLAN	BEC PROJECT NO. 19996.17	BOYLE ENGINEERING CORPORATION

1194 Pacific St., Suite 204 San Luis Obispo, CA 93401
Tel. 805-542-9840 Fax 805-542-9990
WWW.BOYLEENGINEERING.COM

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 2004 through August 2006) 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 0.58 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 0.79 mgd over the past two years (July 2005).

Peak Day Flow (PDF) is the maximum daily flow rate experienced at the WWTF. Flow records show the PDF to be 2.024 mgd (October 3, 2005). The value is questionable because of metering problems. Surges in power supply at the WWTF have caused temporary pump failure on occasion, causing submerged conditions at the meter and resulting in false flow readings. While the water level reading may be accurate, velocity is much lower than under free-flow conditions and, as a result, the meter reading is not representative of the influent flow. For this reason the recorded peak daily flow was not used to determine the design peaking factor. Instead, based on review of similar, primarily domestic-use wastewater facilities, a peaking factor of 2.0 was determined to be conservative for PDF projections. It should be noted that peak day values for July 2005 and January 2006 are also suspected erred readings.

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.

The AAF, AWWF, ADWF, and MMF are based on WWTF flow records; however, the flow meter is not considered reliable for short-term peak flows, so an alternative method must be used to establish the PHF and peaking factor. One common way to determine the peaking factor for peak hourly flow is through an empirical equation based on the plant's service population.

$$P.F. = \frac{18 + P^{0.5}}{4 + P^{0.5}};$$ where P is population in thousands[1].

District staff estimated a 2006 service population of 9,900, which gives a calculated peaking factor of 3.0. Using this peaking factor to calculate, the existing PHF rate is expected to be three-times the AAF, or 1.77 mgd.

It was assumed the flow meter problems (flooding of Parshall flume) would affect short-term peaking measurements (hour or day flows) but would have less impact on long-term averages, since the pump station functions properly most of the time except during power surges. Therefore, the flow meter data was assumed to be reliable for maximum month and average wet weather, dry weather, and annual flows.

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 2.024 mgd (October 3, 2005). As stated earlier, this measurement is questionable.

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 1.899 mgd (January 16, 2006). As stated earlier, accuracy of this measurement is questionable.

[1] Fair, G. M. and Geyer, J. C., *Water Supply and Waste-Water Disposal.* 1st Ed., (1954) Via: Design of Municipal Wastewater Treatment Plants WEF Manual of Practice No. 8, Fourth Edition, Volume 1; Planning and Configuration of Wastewater Treatment Plants, Water Environment Federation, (1998).

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-04	0.497	0.738	0.00
Oct-04	0.443	0.616	2.33
Nov-04	0.456	0.652	2.53
Dec-04	0.473	0.703	5.27
Jan-05	0.582	0.897	2.67
Feb-05	0.611	0.834	5.74
Mar-05	0.625	0.812	4.05
Apr-05	0.622	0.885	1.76
May-05	0.729	1.156	1.95
Jun-05	0.761	1.047	0.08
Jul-05	0.791	1.714*	0.00
Aug-05	0.556	1.400	0.00
Sep-05	0.577	0.999	0.00
Oct-05	0.641	2.024*	0.00
Nov-05	0.533	0.679	0.00
Dec-05	0.547	0.888	0.00
Jan-06	0.654	1.899*	0.90
Feb-06	0.551	0.736	1.48
Mar-06	0.570	0.870	5.15
Apr-06	0.610	0.909	2.40
May-06	0.639	0.798	1.57
Jun-06	0.567	0.952	0.00
Jul-06	0.557	0.752	0.03
Aug-06	0.595	1.202	0.00
	AAF = 0.591	**PDF = 2.024***	**MMF = 0.791**
	AWWF = 0.582	Mean PDWF = 1.091	Max PDWF = 2.024
	ADWF = 0.593	Mean PWWF = 0.905	Max PWWF = 1.899

* Suspected to be erred meter reading due to backflow interference.
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)	0.591	--
Maximum Monthly Flow (MMF)	0.791	1.34
Peak Daily Flow (PDF)	2.024*	2.00
Peak Hourly Flow (PHF)	1.773	3.00
* Measured value suspected to be erred due to meter submergence and was not used to calculate peaking factor		

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), the average BOD_5 concentrations were multiplied by the daily flow rate 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

Date	Influent BOD_5 (mg/L)	Monthly Average Influent BOD_5 (mg/L)	Average Daily Flow (mgd)	Average Daily BOD_5 loading (lb/day)
12/07/05	330	285	0.547	1300
12/21/05	240			
01/04/06	35	215	0.654	1173
01/18/06	340			
01/25/06	270			
02/01/06	310	268	0.551	1230
02/08/06	101			
02/15/06	380			
02/22/06	280			
04/05/06	230	298	0.610	1514
04/12/06	320			
04/19/06	360			
04/26/06	280			
05/03/06	130	258	0.639	1372
05/10/06	350			
05/17/06	250			
05/24/06	300			
06/07/06	233	256	0.567	1209
06/14/06	220			
06/21/06	270			
06/28/06	300			
08/02/06	290	278	0.595	1380
08/09/06	260			
08/16/06	282			
08/30/06	280			
AVERAGE		265	0.595	**1311**
MAXIMUM				**1514**

As the solids layer, including grit, sludge, and screenings, builds up on the bottom of the ponds, the retention time decreases and with it, the effluent water quality. Pond 4 was recently drained and cleaned. However, the other ponds (1, 2 and 3) have not been thuroughly cleaned. Assuming an influent TSS concentration of 265 mg/L, the net volume of solids generated over the past 5 years was estimated to be approximately 960,000 gallons at 15% solids (or 639 dry tons), about 7 % of the total available pond volume. If a higher dilution factor is assumed (7-8% solids), which is typical in poorly consolidated sludge, up to 15% of pond volume could be occupied. Calculations are included in Appendix B.

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 2004 and August 2006. 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.

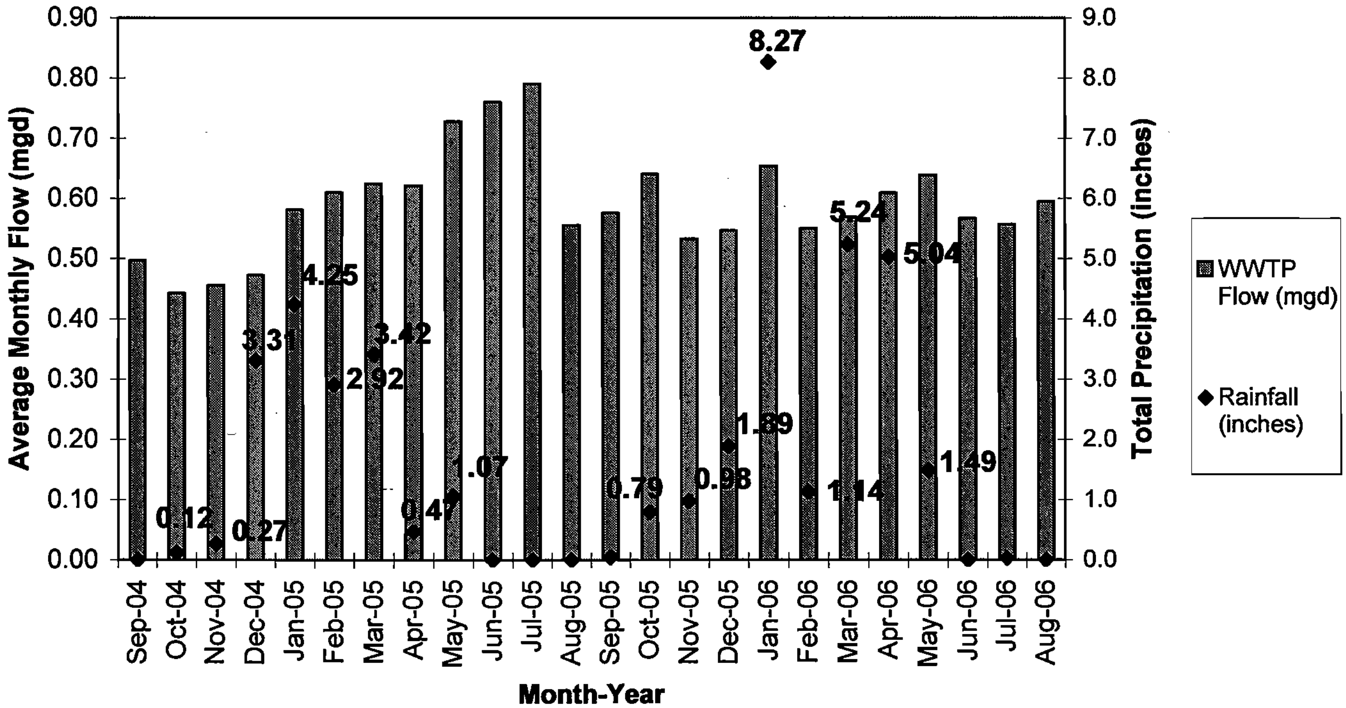


Figure 2-1 Southland Monthly Average Daily Flows and Total Precipitation (Sept 2004 – Aug 2006)

3.0 PROJECTED LOADS

3.1 Projected Future Flow Demands

Plant records from the past 2 years revealed an AAF of 0.591 mgd. This number is comparable to the AAF, 0.63 mgd, found in a conjunctive study underway to complete the Draft NCSD Water and Sewer Master Plan (currently being performed by Cannon Associates and Garing Taylor & 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 the Draft 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 2007 according to this conservatively high growth projection. However, it may not be reached until 2008 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 by spring of 2007.

Table 3-1 Projected Flow Rates

Flow Condition	Peaking Factor	Existing Flow (mgd)	Projected Flow (mgd)**				
			2010	2015	2020	2025	2030
Average Annual Flow (AAF)	--	0.591	0.838	1.05	1.25	1.45	1.67
Maximum Monthly Flow (MMF)	1.34	0.791	1.12	1.41	1.68	1.94	2.34
Peak Daily Flow (PDF)	2.00	2.024*	1.68	2.10	2.50	2.90	3.34
Peak Hourly Flow (PHF)	3.00	1.77	2.51	3.15	3.75	4.35	5.01

* Measured value suspected to be erred due to meter submergence
** Projected AAF based on Draft Water and Sewer Master Plan (GTA & Cannon Assoc.)

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)	0.591	0.838	1.05	1.25	1.45	1.67
Average Annual BOD_5 Loading (lb/day)	1,311	1,860	2,330	2,770	3,220	3,700
MMF (mgd)	0.791	1.120	1.41	1.68	1.94	2.34
Maximum Monthly BOD_5 Loading (lb/day)	1,514	2,140	2,700	3,220	3,710	4,480

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 December 2005 through August 2006. The frequency diagram reveals that 90% of the time the influent BOD_5 concentration is less than 350 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.

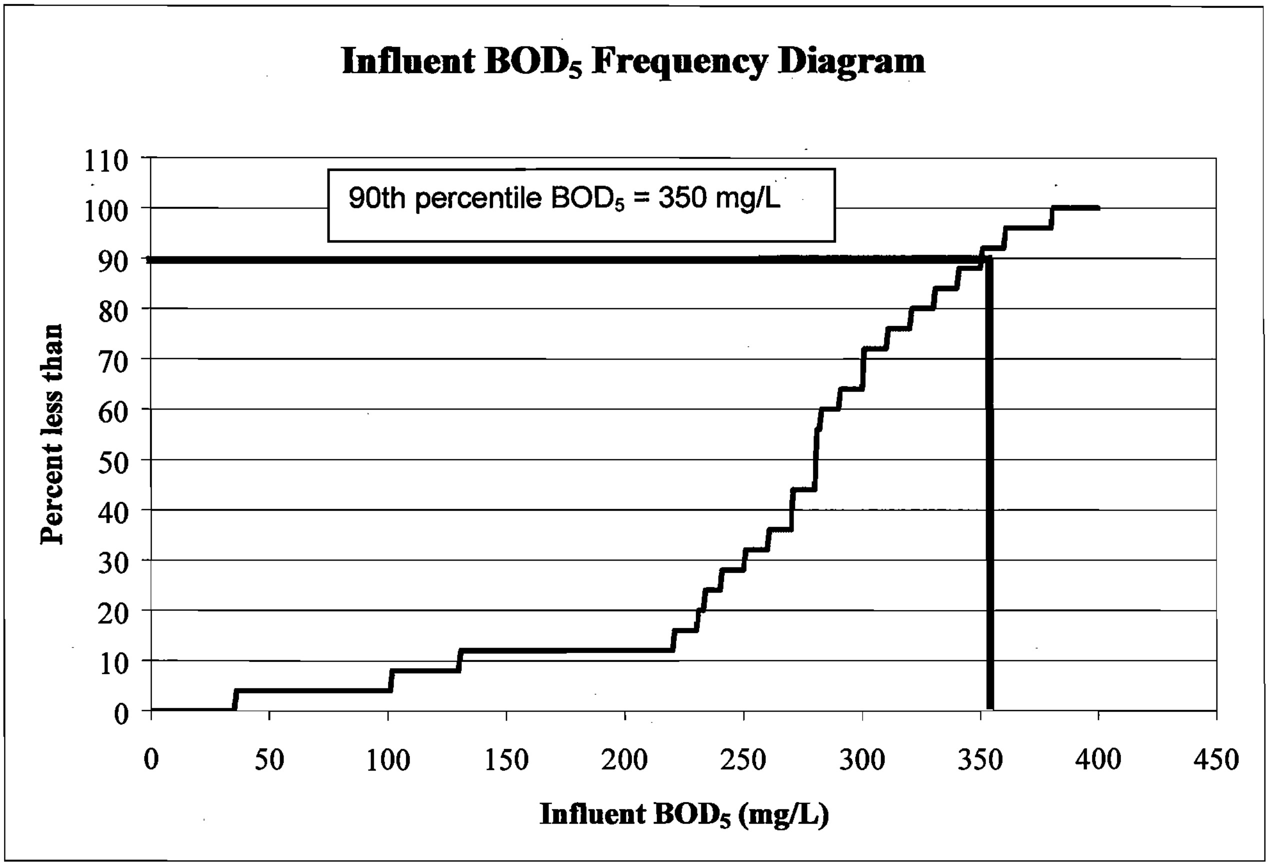


Figure 3-1 Influent BOD_5 Frequency Diagram

Future sludge production was estimated for a 5-year period at the projected 2030 AAF. The average influent TSS was assumed to be 265 mg/L, based on historical BOD data. Based on 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 fit 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, though much of the subsurface equipment remains. All subsurface equipment has been removed from Pond 4 and some from Pond 1. The District has plans to remove the remaining pieces of the subsurface aeration systems.

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[2], total hp (# of units)	110 (14)
Ponds 1 & 2, each	(4) @ 10 + (1) @ 5
Ponds 3 & 4, each	(2) @ 5

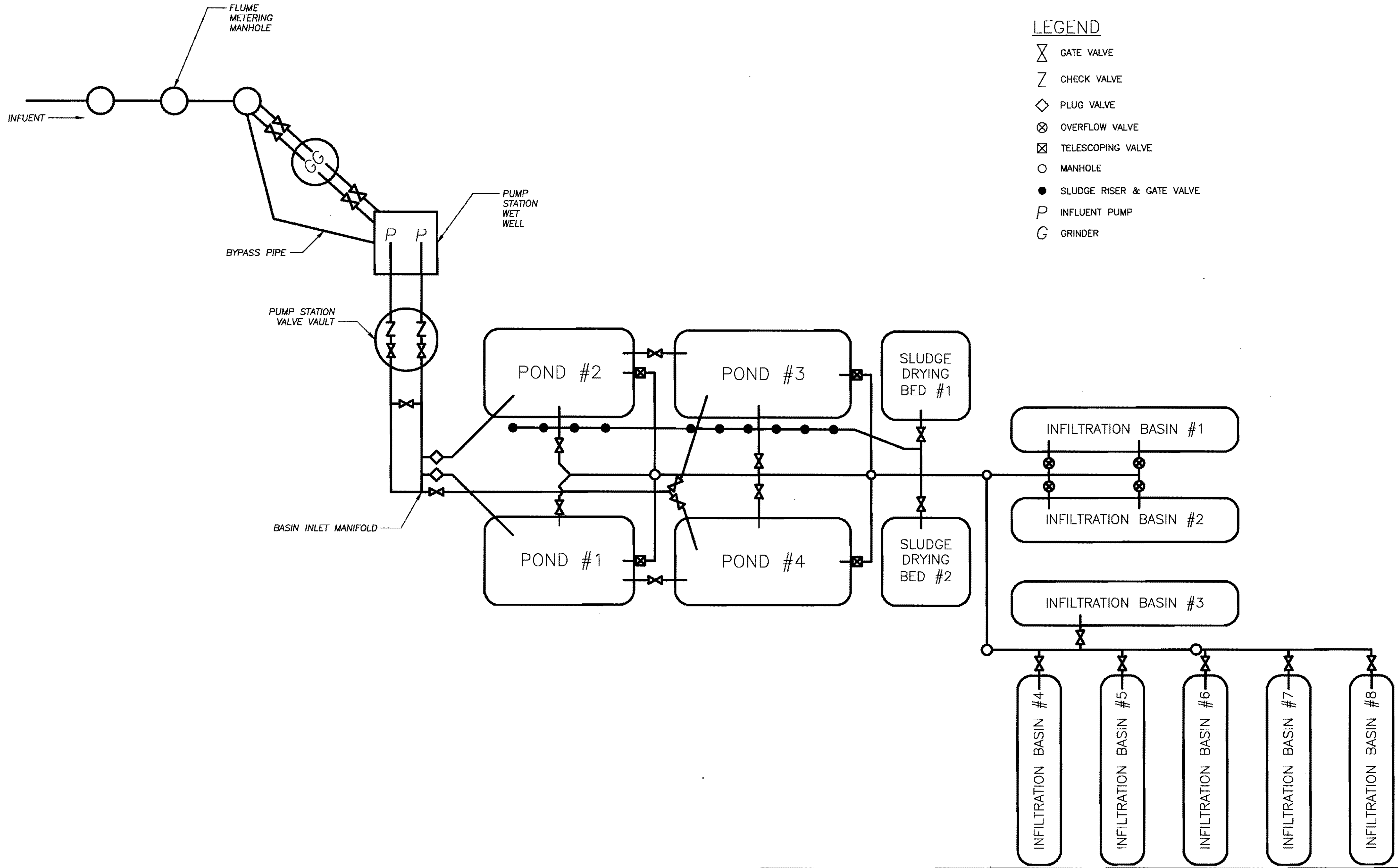
[2] Anticipated aerator distribution after Pond 4 is back online. Pond 4 was taken offline in February of 2006 for maintenance.

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.

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. 19996.17

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. Potential causes for violations were identified in the Southland WWTF Action Plan (Boyle, May 2006). Laboratory error may have been a factor in wastewater violations. Duplicate analyses of BOD_5 began in September 2005 and several significant discrepancies were found in the results from the two laboratories (Fruit Growers Laboratories (FGL) and Creek Environmental Laboratories (CEL)). Differences ranged from 30 to 90 mg/L in the first three months of duplicate analyses.

Evaluation of the existing Ramco subsurface aeration system revealed limitations that could result in poor BOD removal. Oxygen transfer and mixing was limited due to clogging and binding of the impellers, which are designed to break up coarse bubbles delivered by the diffusers. The lack of influent screening facilities may have contributed to clogging for this aeration system. Air delivery is further limited by the capacity of the diffusers. The blowers were sized to deliver approximately 14 cfm per diffuser, but each diffuser is expected to deliver only 4 cfm.

Phased replacement of the subsurface aeration system began in spring of 2004. The subsurface diffusers have been replaced with mechanical aerators in Ponds 1 and 2, though much of the subsurface aeration system remains. Analysis of BOD test results in December 2005 and January 2006 indicated a significant increase in nitrogenous BOD throughout the treatment process. This increase could be attributed to the lack of adequate aeration in Ponds 3 and 4.

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 results in solids being decanted directly to the downstream ponds. The District is proceeding with plans to resolve the problem.

Sludge accumulation in the ponds may contribute to effluent violations. In mid-December 2005 the District measured sludge levels in the ponds and found the level to be near the fixed height of the outlet from Pond 2. Levels had also accumulated to 4 to 5 feet near the curtain between the stabilization and aeration cells in Ponds 3 and 4.

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-04	0.299	0.738	0.497	21.9	64.6	41.2	40	50	42	4.2	5.7	4.7	<0.05
Oct-04	0.124	0.616	0.443	3.3	71.0	37.1	30	60	45	4.4	7.3	5.9	<0.05
Nov-04	0.147	0.652	0.456	30.2	49.0	39.3	40	110	73	4.2	7.3	6.0	<0.05
Dec-04	0.222	0.703	0.473	34.0	122.0	67.6	40	70	58	4.6	7.8	6.6	<0.05
Jan-05	0.220	0.897	0.582	69.0	115.0	89.3	50	70	60	4.7	7.8	5.9	<0.05
Feb-05	0.303	0.834	0.611	37.0	101.0	72.8	40	70	55	4.3	6.7	5.2	<0.05
Mar-05	0.458	0.812	0.625	44.0	56.1	49.8	20	120	44	2.8	4.8	4.1	<0.05
Apr-05	0.330	0.885	0.622	2.9	40	25	20	20	20	4.2	7.0	5.4	<0.05
May-05	0.481	1.156	0.729	14.8	33.2	21	20	50	30	4.8	5.2	5.0	<0.05
Jun-05	0.484	1.047	0.761	3.8	43	31.7	40	50	42	5.3	5.9	5.5	<0.05
Jul-05	0.435	1.714	0.791	8	91	46.5	30	80	48	4.6	5.6	5.3	<0.05
Aug-05	0.381	1.400	0.556	43	237	150.8	20	40	28	5.4	5.9	5.7	<0.05
Annual Avg			**0.596**			**56**			**45**			**5.0**	**<0.05**
Annual Max		**1.714**			**237.0**	**150.8**		**120**	**73**		**7.8**		
Annual Min	**0.124**			**2.9**			**20**			**2.8**			
Sep-05	0.304	0.999	0.577	23.4	218	116.6	5	30	19	4.9	7.5	6.2	<0.05
Oct-05	0.359	2.024	0.641	33.3	177	111.8	30	50	40	3.9	5.8	5.1	<0.05
Nov-05	0.336	0.679	0.533	24.8	176	91.4	20	50	33	4.8	6.7	5.6	<0.05
Dec-05	0.362	0.888	0.547	29	149	76.3	10	40	28	6.2	6.9	6.7	<0.05
Jan-06	0.371	1.899	0.654	31.3	48	41.8	10	20	18	2.0	6.6	4.7	<0.05
Feb-06	0.305	0.736	0.551	23.7	50	34.8	20	20	20	2.5	5.9	3.7	<0.05
Mar-06	0.341	0.870	0.570	24.9	63	43.4	20	50	30	2.1	5.0	4.2	<0.05
Apr-06	0.309	0.909	0.610	28.8	42	34.2	10	20	15	4.4	5.6	4.9	<0.05
May-06	0.376	0.798	0.639	26	44	35.6	10	60	27.5	3.8	4.3	4.0	<0.05
Jun-06	0.436	0.952	0.567	25	45	33.8	20	40	35	2.6	3.7	3.3	<0.05
Jul-06	0.318	0.752	0.557	33	96	54.25	20	50	37.5	3.1	4.3	3.8	<0.05
Aug-06	0.37	1.202	0.595	23	49	32	20	60	30	3.4	4.4	4.1	<0.05
Annual Avg			**0.587**			**59**			**28**			**4.7**	**<0.05**
Annual Max		**2.024**			**218.0**	**116.6**		**60**	**40**		**7.5**		
Annual Min	**0.304**			**23.0**			**5**			**2.0**			

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

FIGURE 5-1

NCSD SOUTHLAND WWTF MASTER PLAN

FRONTAGE ROAD TRUNK MAIN STUDY AREA

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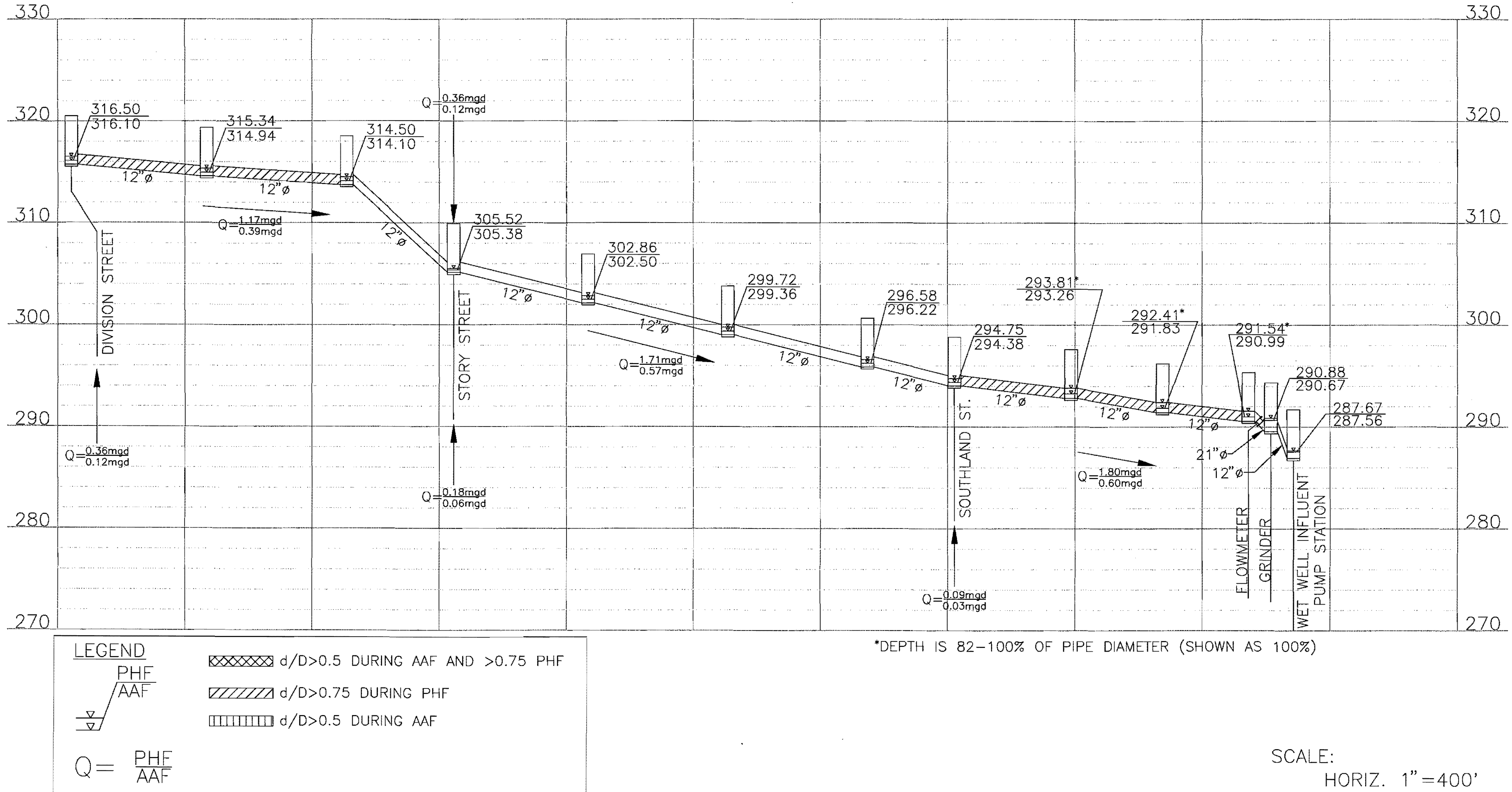

*DEPTH IS 82–100% OF PIPE DIAMETER (SHOWN AS 100%)

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

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NCSD SOUTHLAND WWTF MASTER PLAN

FRONTAGE ROAD TRUNK MAIN
HYDRAULIC PROFILE FOR EXISTING DEMANDS

BEC PROJECT NO. 19996.17

FIGURE 5-2

DWG: W:\Nipomo CSD (19996)\Southland Master Plan 19996.17\CADD\Figures\FIGURE 5-2.dwg USER: emick
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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).

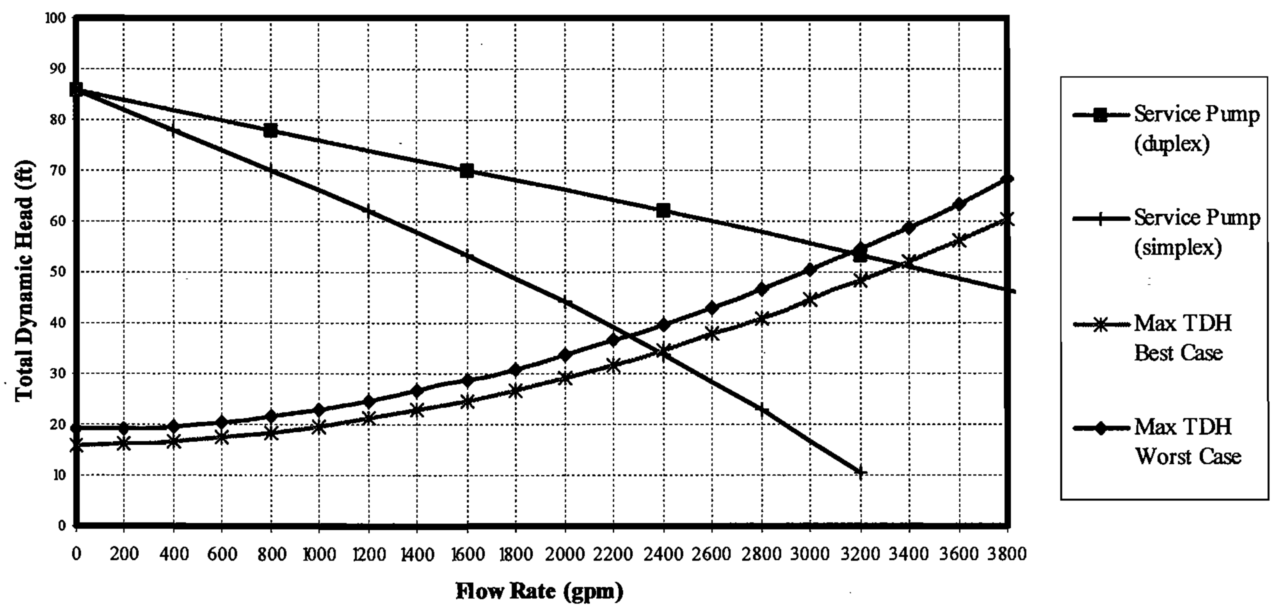

Figure 5-3 Composite Service Pump Curve and System Curve

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[3]:

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

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 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. 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 90th percentile BOD_5 (350 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 an 87 – 90% reduction in BOD_5

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[3] Sanks, Robert L. *Pumping Station Design,* 2nd Edition. Butterworth-Heinemann: (1998), 370.

concentration (from 350 mg/L to 36 – 45 mg/L). The two ponds in series configuration also shows the ability for adequate levels of treatment, providing effluent BOD_5 concentrations between 55 and 64 mg/L, or an 82 – 84% reduction of BOD_5. However, other factors can hinder the ponds' capability to reduce BOD when operating in series. 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 as the predicted capacity of the plant as opposed to the ponds in series. Table 5-2 summarizes the results of the analysis and calculations are included in Appendix B.

Table 5-2 Modeled Effluent Quality Under Existing Flow Conditions

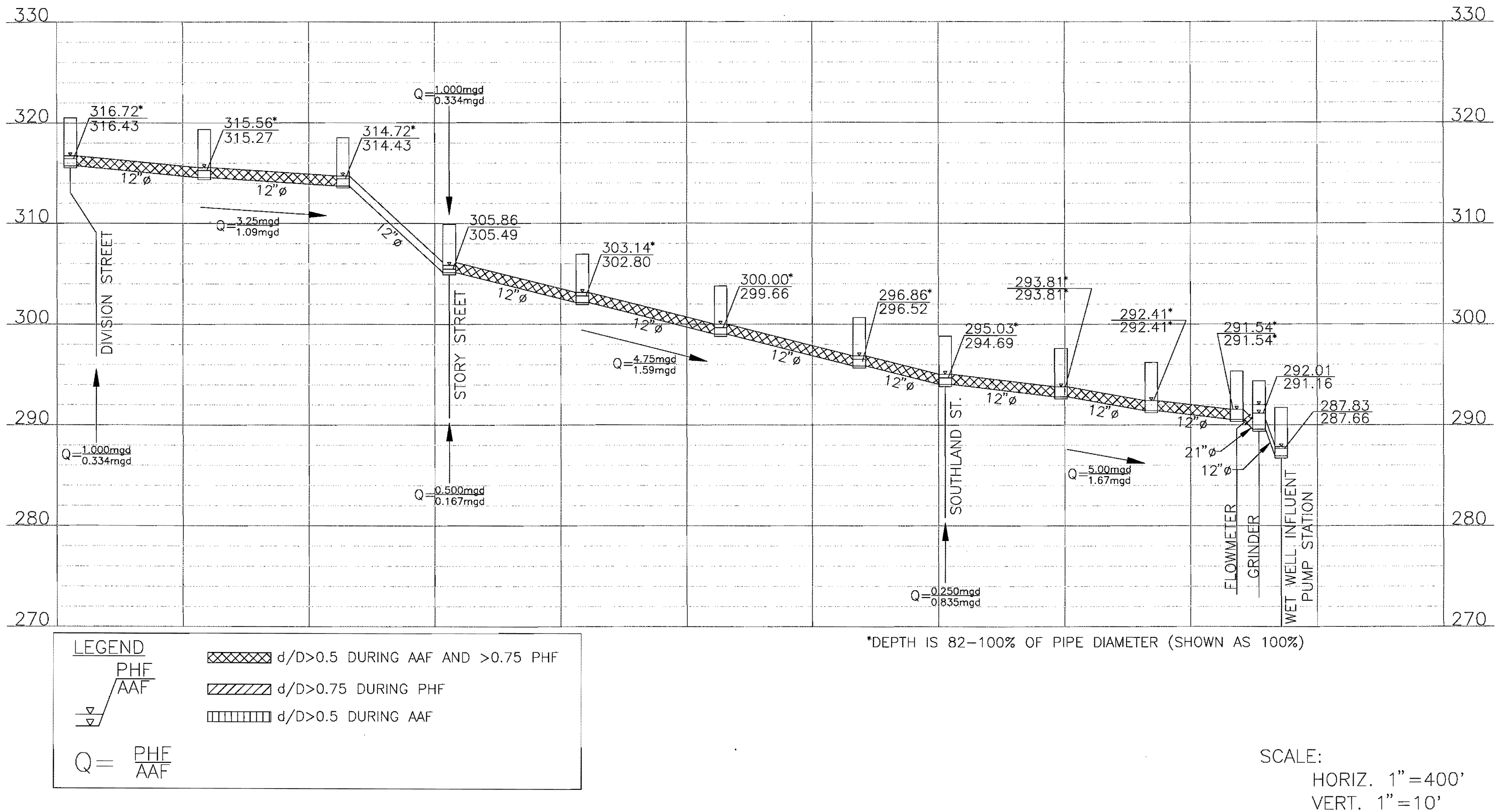
	Temperature (T) and Flow (Q) Conditions		
	Low T, Low Q	High T, High Q	High T, MMF
4 Ponds in Series [BOD_5] (mg/L)	41	36	45
2 Parallel Trains of 2 Ponds [BOD_5] (mg/L)	59	55	64
WDR Effluent BOD_5 limit = 100 mg/L			

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.



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FRONTAGE ROAD TRUNK MAIN
HYDRAULIC PROFILE FOR 2030 PROJECTED DEMANDS

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FIGURE 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 90th percentile BOD_5 concentration (350 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.

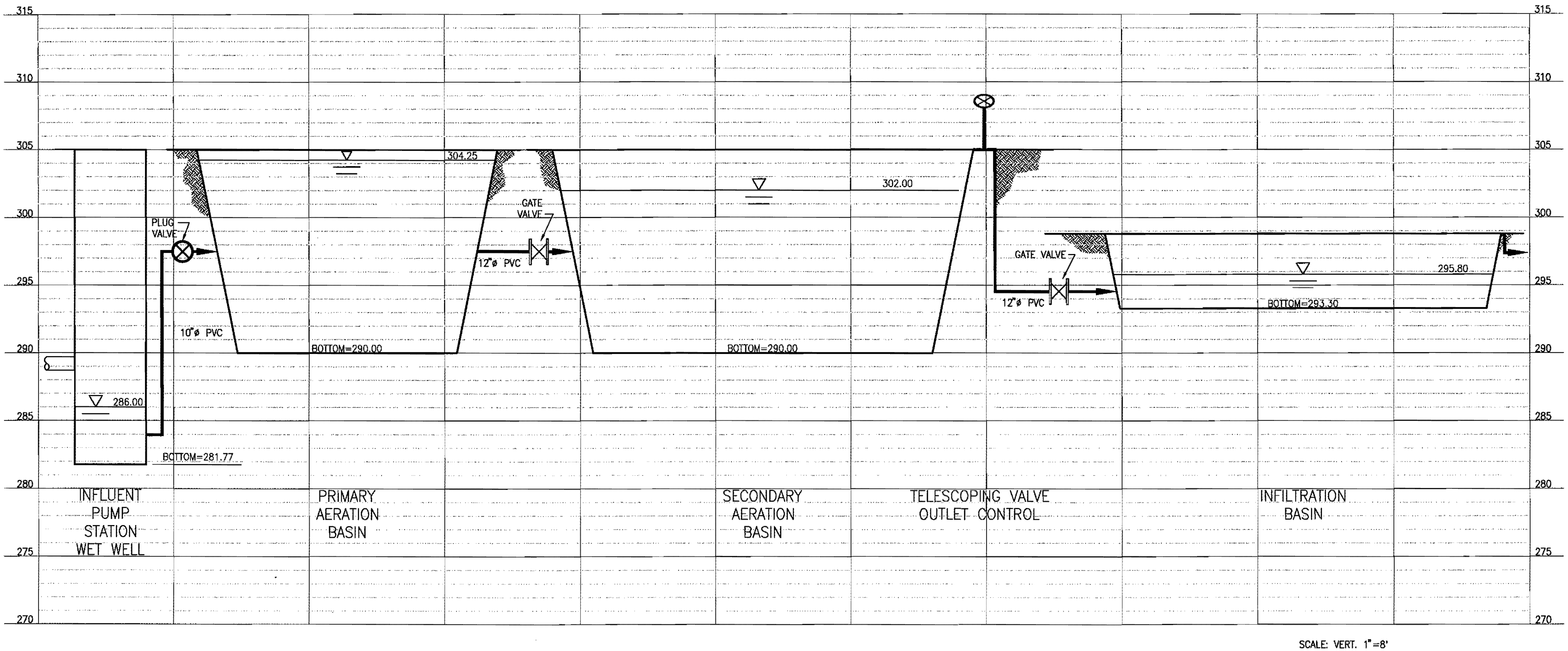
Table 5-3 summarizes the results of the analysis. Neither configuration provides sufficient treatment under any boundary flow condition. Full calculations are included in Appendix B.

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] with baffle (mg/L)	151	180	135
[BOD_5] without baffle (mg/L)	121	150	105
2 Parallel Trains of 2 Ponds			
[BOD_5] with baffle (mg/L)	162	189	148
[BOD_5] without baffle (mg/L)	135	162	121
Existing WDR Effluent BOD_5 Limit = 100 mg/L			

If the ponds are operated in two parallel trains of two, the permitted BOD_5 effluent limit is expected to be reached by 2008 during high temperature, high flow conditions according to the conservative growth projections presented in Section 3.0. If the ponds are run in series, the permitted BOD_5 limit will be reached in 2010 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 ponds are operating close to their permitted capacity (see Section 3.0).



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NCSD SOUTHLAND WWTF MASTER PLAN

TREATMENT PLANT HYDRAULIC PROFILE FOR 2030 PEAK DAY FLOW

BEC PROJECT NO. 19996.17

FIGURE 5-5

DWG: W:\Nipomo CSD (19996)\Southland Master Plan 19996.17\CADD\Figures\FIGURE 5-5.dwg USER: emick
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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 be upgraded 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[4], 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

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

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[5] (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.

[5] Effluent is currently secondary

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.

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

Problem and Related Constituent	References	Water Quality Guidelines: No Problem	Increasing Problems	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

	Constituent Limits				
	Salt tolerance		Chloride tolerance (Cl^-)		Boron tolerance
	In Sat. Soil Extracts EC_e[1]	In Irrigation Water EC_w[2]	In Sat. Soil Extracts[3]	In Sat. Soil Extracts[4]	In Soil Water[5]
Crop	(dS/m) or (mmhos/cm)	(dS/m) or (mmhos/cm)	(mol/m^3)	(mg/l)	(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 - Recycled water or recharge water in or above mound, for ammonia, org-N, nitrate, nitrite, and DO in excess of the BOD as required - 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 - 10 mg/L total N at a max frequency	Recycled Water: 10 mg/L total N As established by engineering report for: - Total N at some level <10 mg/L when used as part of a comprehensive nitrogen control scheme - Ammonia, nitrite, and/or org-N - Minimum DO in excess of BOD Groundwater: - 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 - Baseline monitoring and operations plan - 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 - Identify criteria for suspending recharge - Baseline monitoring and operations plan - 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 - Monitoring plan - 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
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: 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 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: 1. A demonstration that historic recharge with water containing comparable levels of nitrogen has not caused a problem, 2. Evidence that recharge water can be tracked and monitored throughout the flow path, and 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
[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. The WWTF is meeting the current Waste Discharge Requirement (WDR) and has not had a violation since December 2005. Thus, another option is to continue current discharge practices. The obvious advantage is cost. However, the District is not currently taking advantage of their treated water as a resource. There is the potential to improve reliability of groundwater resources, and conserve these supplies, through the use of reclaimed water for the uses discussed herein. However, the likelihood of the current disposal practice to remain an option is in question with the expectation that future water quality regulations will tighten. It may become necessary to improve treatment and/or better demonstrate no impact to groundwater as a result of the infiltration (a condition of the WDR), particularly if regulatory agencies become concerned with nitrates or other constituents in the area.

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

The District currently monitors these constituents monthly with three onsite monitoring wells. RWQCB staff have expressed concern with the potential impacts of the treatment plant's effluent on groundwater, noting that the wells may be receiving input from a shallow perched zone, making it difficult to evaluate the potential for impacts attributed to effluent percolation.

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.

In addition, percolation tests should be conducted adjacent to the existing percolation ponds in order to evaluate potential onsite capacity for effluent disposal, if available area was converted to percolation ponds. This should include an assessment of "baseline" groundwater conditions beneath the site, in order to evaluate potential impacts on groundwater in the future.

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;
- Evaluate percolation capacity of the existing infiltration basins and potential future infiltration locations on the treatment plant site; and
- Select a future treatment plant process which will provide adequate pretreatment for filtration. If uses such as park/school irrigation, groundwater recharge, or continued onsite percolation (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.

Wetwell and Pumping Capacity

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. Our 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. This discrepancy may be due to differences in estimated and actual peak flows (See Section 3.0), but 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 2015. Therefore, it is recommended that new pumps be installed by 2012 (at the latest – constructing a new pump station in 2009 could be accomplished 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)

Recommended Influent Lift Station Improvements

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.

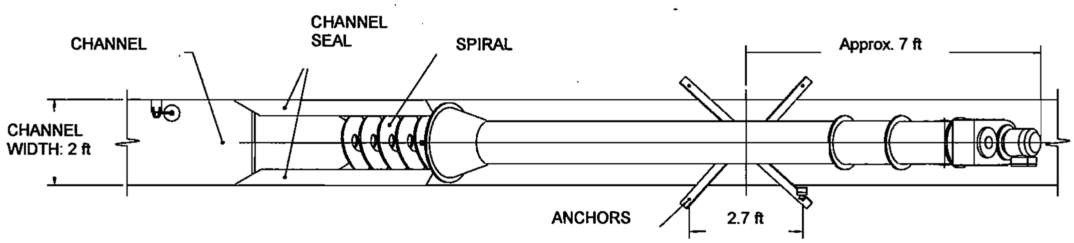


Figure 7-1 Top view Hycor® Helisieve®

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.

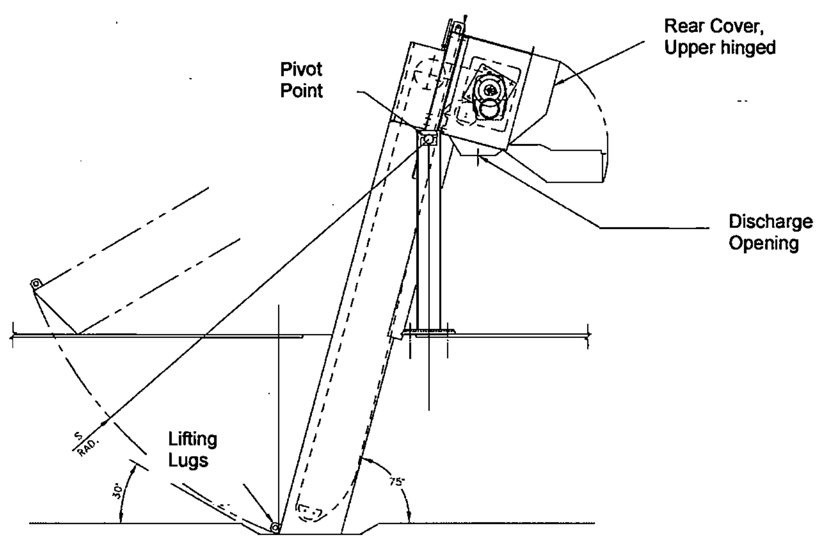


Figure 7-2 Profile view AquaGuard®

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.

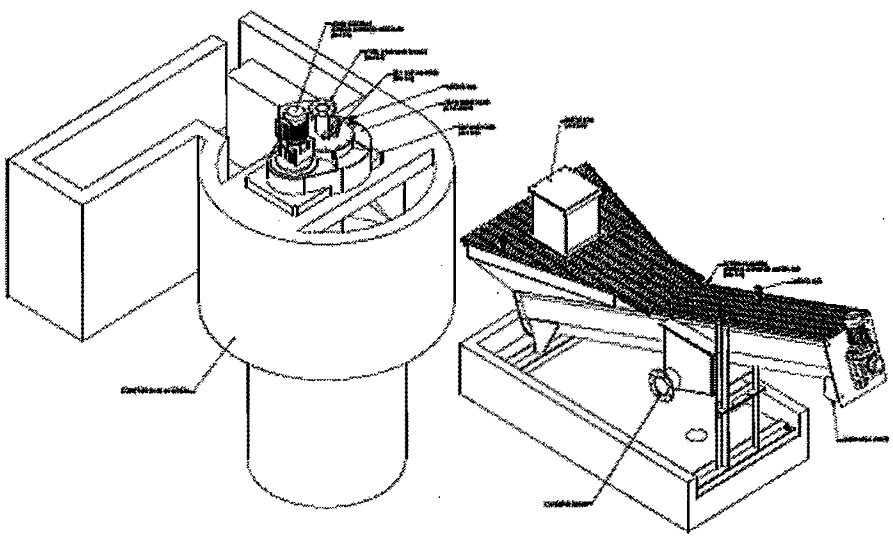


Figure 7-3 Jones & Attwood JetAir® and Screw Classifier

(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.

<u>Drum Screens</u>

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®	$468,000
(2) Parkson Aqua Guard® AG-MN-A	$783,000
Grit Removal	
(2) Jones & Attwood JetAir 100 Grit Trap + Model 100 Screw Classifier	$560,000
(2) Aerated Grit Chambers[6]	$539,000

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

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

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 $200,000 - $250,000 per pond.

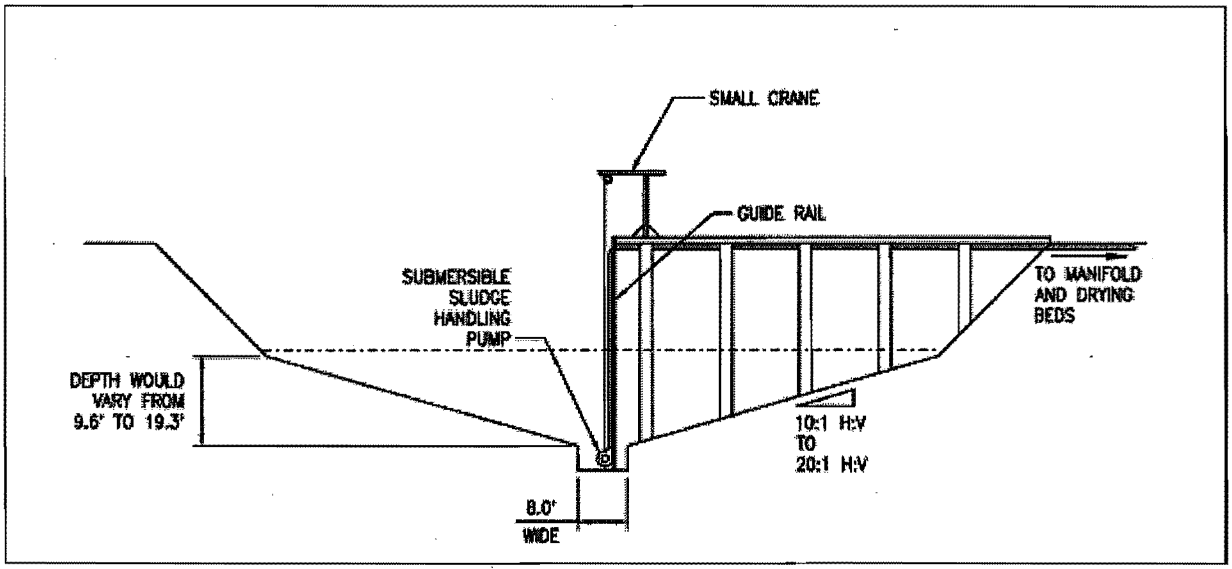


Figure 7-4 Conceptual schematic of pond with sump

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

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.

Monitoring/Analytical Capabilities

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 adjacent to the blower building, 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.

Improved Pond Access

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 $15,000. 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 $18,000, plus costs for an anchoring system.

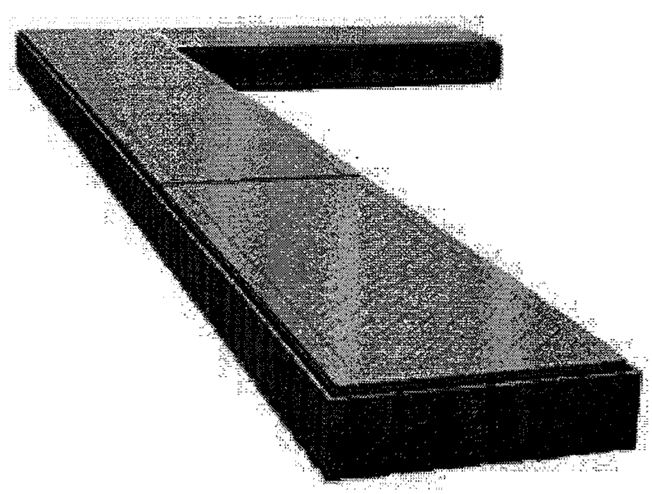


Figure 7-6 ShoreMaster's Polydock®

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.

7.7 Short-Term Performance Improvements and Monitoring

As discussed in Section 5.0, the plant is operating close to its permitted capacity. In order to meet the District's wastewater demand while a plant expansion is being planned and designed, we recommend the following steps:

1. Remove the baffles in both Ponds 3 and 4 to provide the maximum volume of treatment capacity within the ponds.
2. Spread the aerators to optimize mixing and aeration within Ponds 3 and 4. However, the outlet should be located outside of the manufacturer's recommended zone of influence around the aerators.
3. Replace the existing floating outlets with flexible outlet pipes that are mounted to a fixed pole or walkway. The outlet could be mounted to the pole by a chain and an adjustable hook.
4. Begin sampling BOD_5, TSS, carbonaceous BOD ($CBOD_5$), soluble BOD ($SBOD_5$), total Kjeldahl nitrogen (TKN), total ammonia, nitrate, temperature, and nitrate in the plant influent and in the effluent from each pond. Samples should be taken on a monthly basis to allow the District to evaluate whether an interim increase in their permitted capacity, or an interim increase in their permitted effluent limits, could be requested from Regional Water Quality Control Board. This would allow more time for the District to expand the treatment facility.

8.0 FUTURE PROCESS ALTERNATIVES

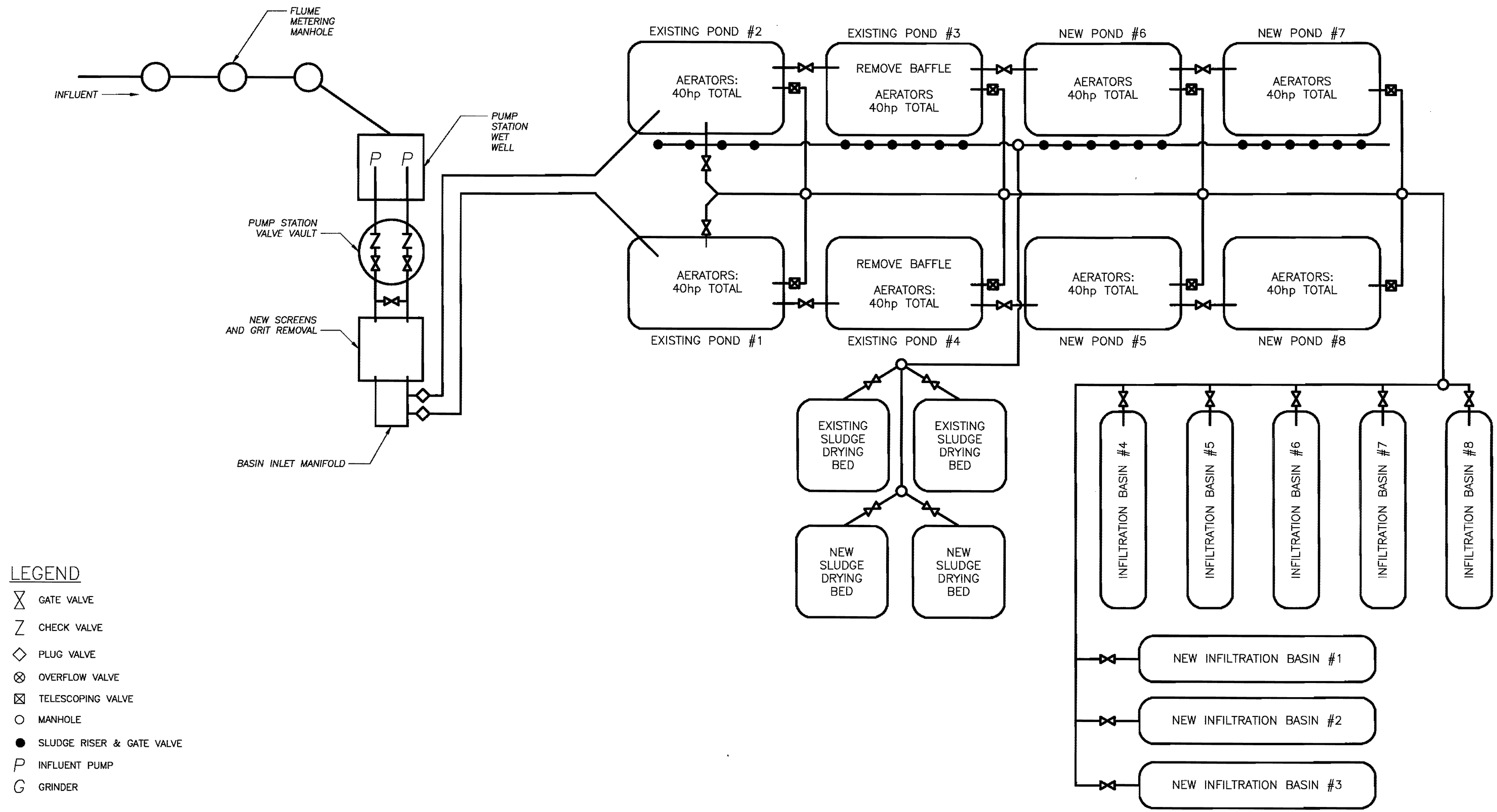
The anticipated effluent requirements for permitting and future flow increases necessitate investigation of treatment process alternatives. Four alternatives were reviewed and are discussed below: expansion of the current treatment process with additional aerated ponds, a conversion to Biolac® Wave Oxidation 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.

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 two primary ponds where the water is fully aerated. Pond 4 was drained for maintenance in February 2006. Once all four ponds are online, there will be four 10-hp mechanical surface aerators and one 5-hp mixer in each primary pond. From the primary ponds, wastewater flows into secondary ponds. The inlet and outlet ends of the secondary ponds are split with a baffle curtain to minimize short-circuiting and provide a quiescent zone. The front 40% of each pond is aerated with two 5-hp mechanical surface aerators, and the back 60% acts as a stabilization basin, providing settling time. 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 205 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 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, 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.	FIGURE
19996.17	8-1

DWG: W:\Nipomo CSD (19996)\Southland Master Plan 19996.17\CADD\Figures\FIGURE 8-1.dwg USER: emick
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DWG: W:\Nipomo CSD (19996)\Southland Master Plan 19996.17\CADD\Figures\FIGURE 8-2.dwg USER: emick
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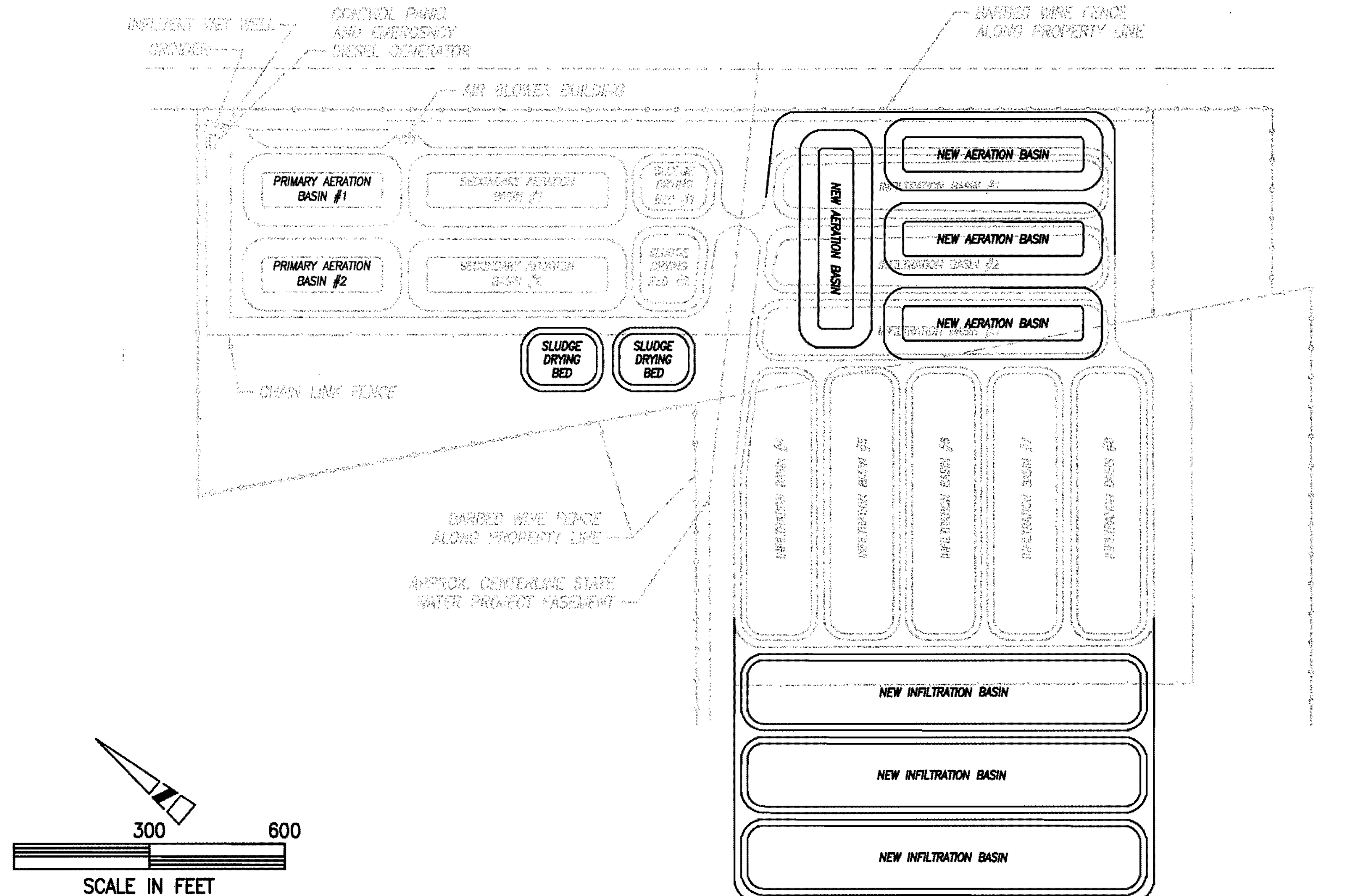


FIGURE 8-2	NCSD SOUTHLAND WWTF MASTER PLAN ALTERNATE 1: SITE PLAN FOR AERATED PONDS	BEC PROJECT NO. 19996.17	BOYLE ENGINEERING CORPORATION 1194 Pacific St., Suite 204 San Luis Obispo, CA 93401 Tel. 805-542-9840 Fax 805-542-9990 WWW.BOYLEENGINEERING.COM

8.2 Biolac® Conversion

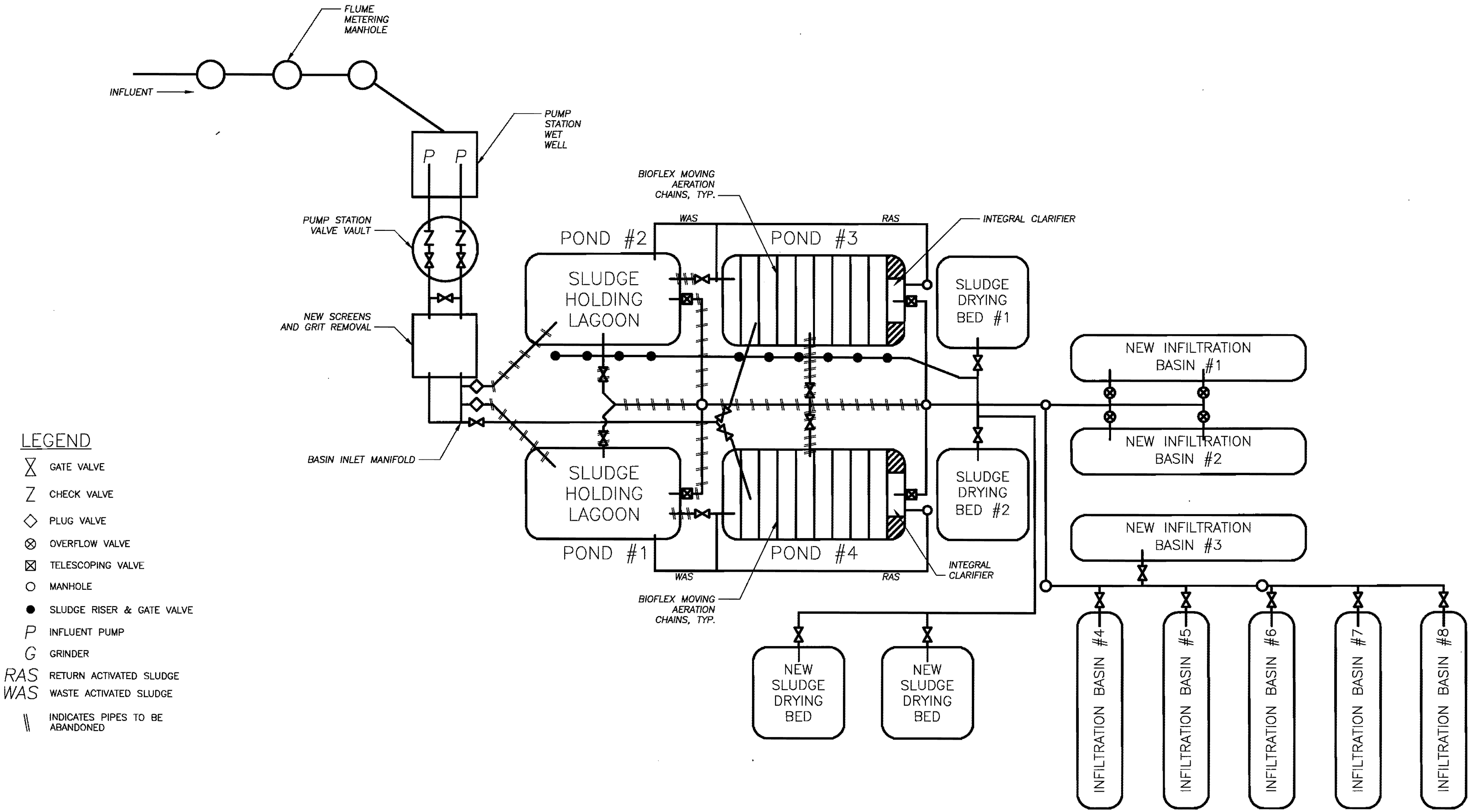
The Parkson® Biolac® Wave Oxidation System is an extended aeration process that utilizes a longer solids retention time (SRT) and moving aeration chains to reduce BOD and TSS concentrations to below 15 mg/L and total nitrogen to less than 10 mg/L. The extended SRT increases the stability of the system, 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 Biolac 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 and integral clarifiers, which will each fit within the footprint of a pond. A Biolac system in one pond will provide adequate treatment until the MMF reaches approximately 1.7 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 $7,000,000, while a pond system would cost approximately $13,700,000. 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.



BOYLE ENGINEERING CORPORATION 1194 Pacific St., Suite 204 San Luis Obispo, CA 93401 Tel. 805-542-9840 Fax 805-542-9990 WWW.BOYLEENGINEERING.COM	NCSD SOUTHLAND WWTF MASTER PLAN BIOLAC® CONVERSION PROCESS FLOW DIAGRAM	BEC PROJECT NO. 19996.17	FIGURE 8-3

DWG: W:\Nipomo CSD (19996)\Southland Master Plan 19996.17\CADD\Figures\FIGURE 8-3.dwg USER: emick
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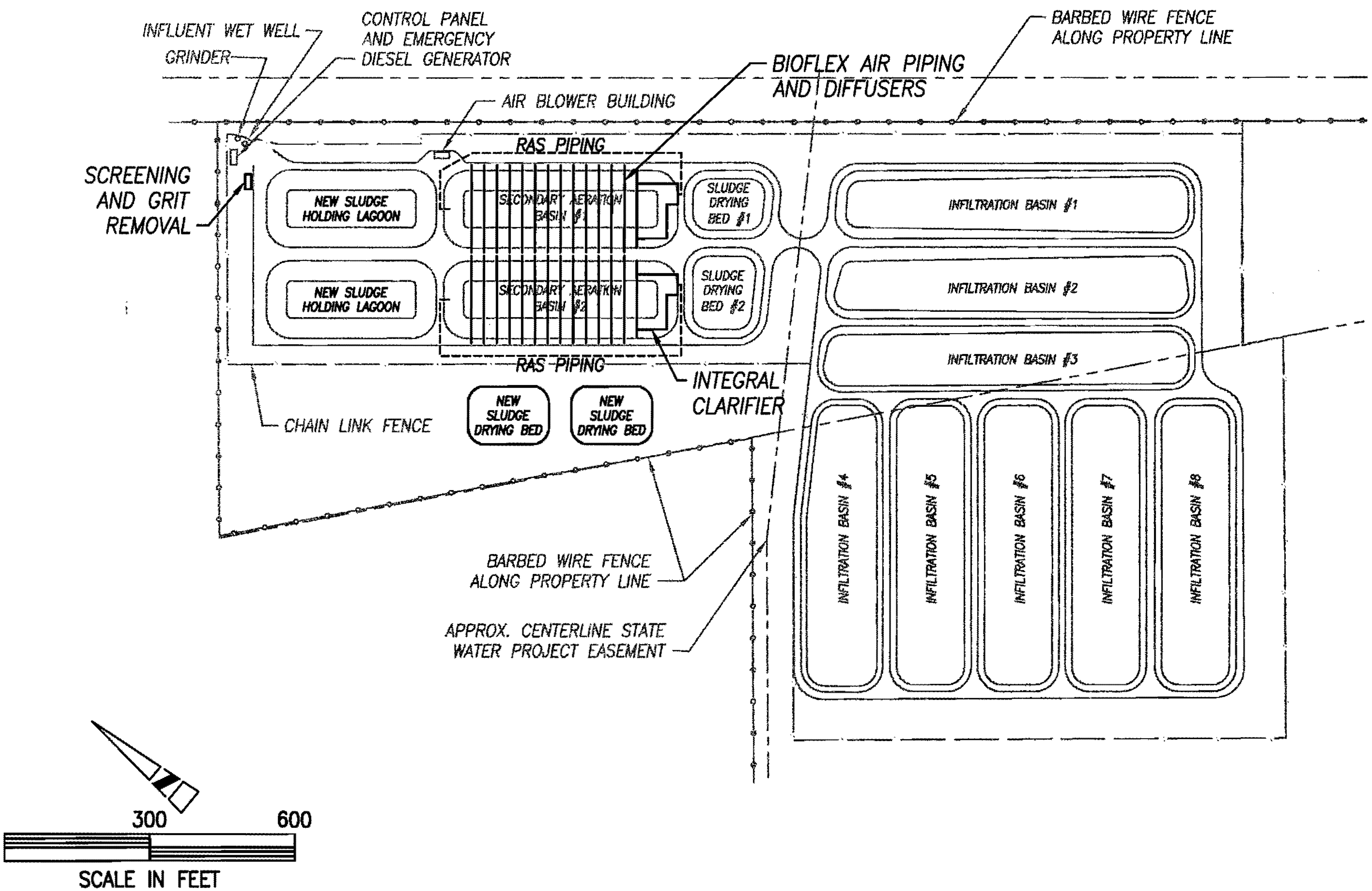

FIGURE 8-4	NCSD SOUTHLAND WWTF MASTER PLAN ALTERNATE 2: SITE PLAN FOR BIOLAC	BEC PROJECT NO. 19996.17	BOYLE ENGINEERING CORPORATION 1194 Pacific St., Suite 204 Tel. 805-542-9840 San Luis Obispo, CA 93401 Fax 805-542-9990 WWW.BOYLEENGINEERING.COM

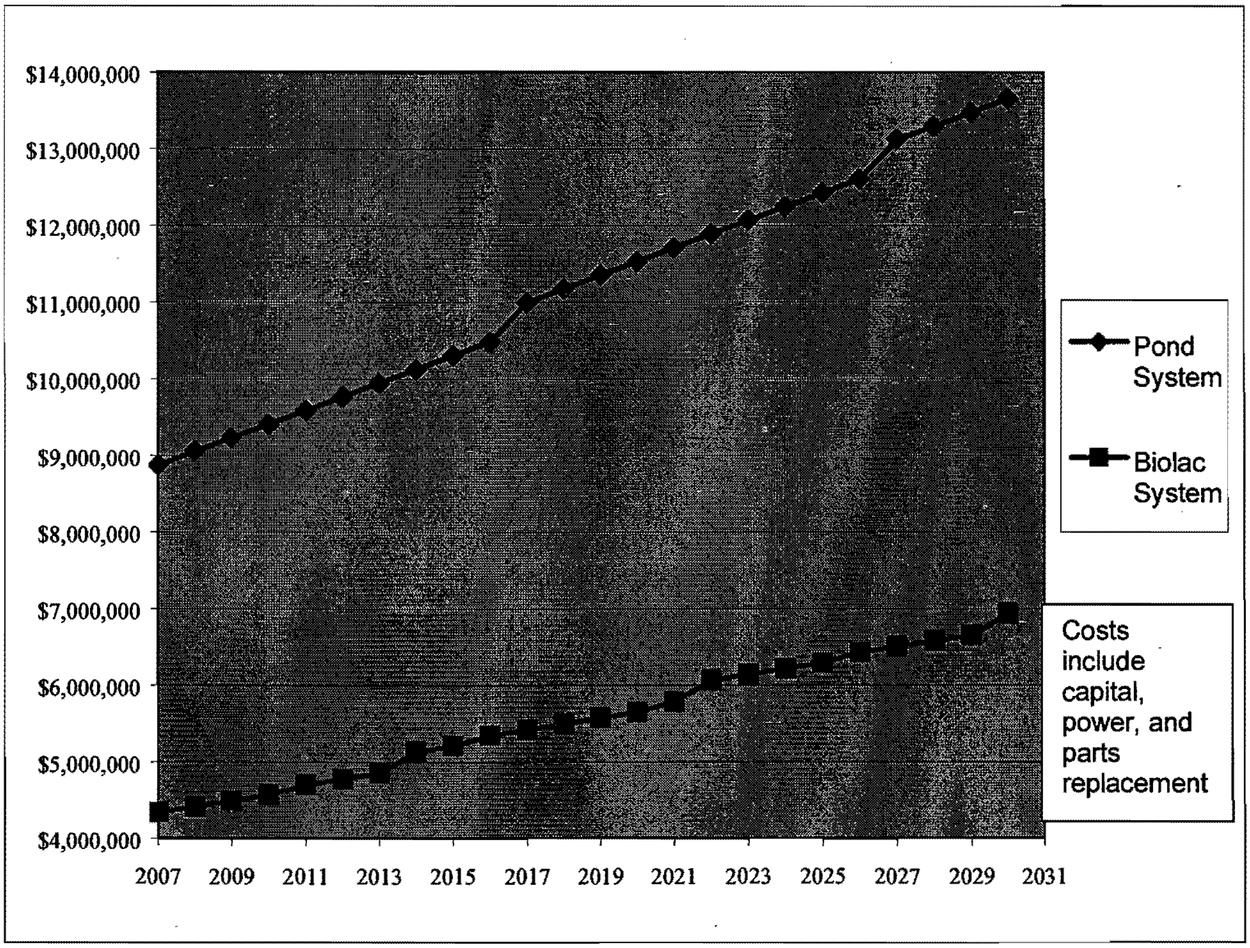

Figure 8-5 Comparative Life-Cycle Costs of an Aerated Pond System and a Biolac® System

8.3 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[7]. The basic process flow diagram is shown as Figure 8-6.

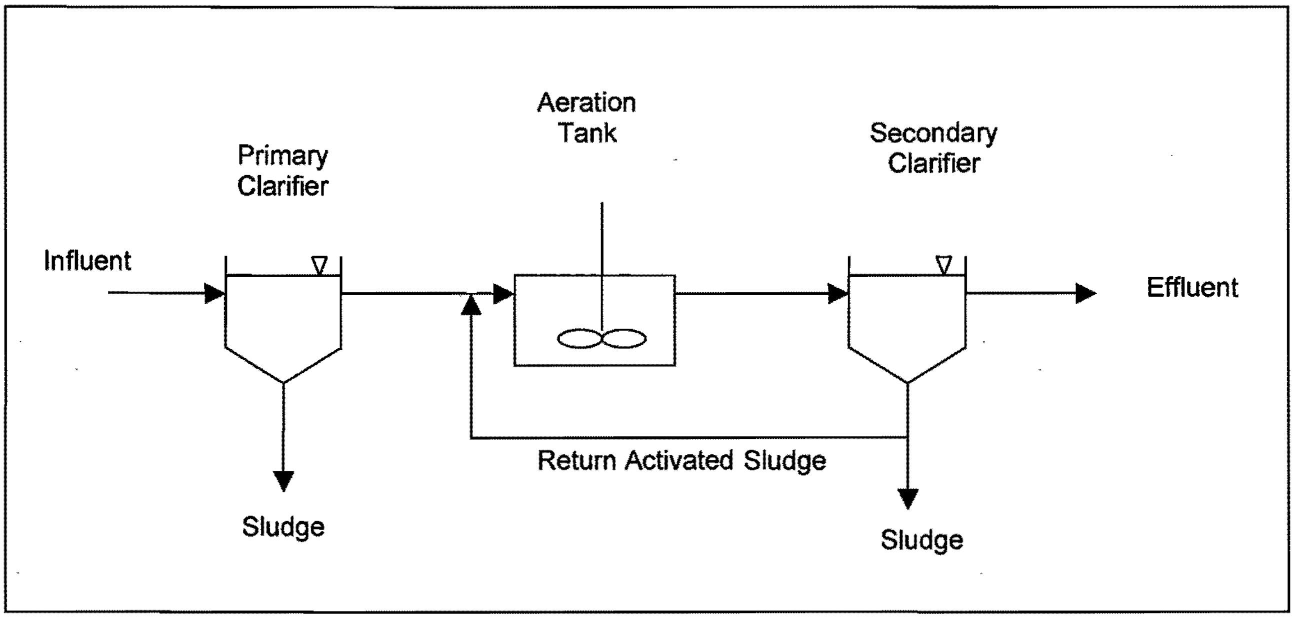

Figure 8-6 Completely mixed activated sludge process flow diagram

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

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

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 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.

8.4 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 (2006 US $)	Total Estimated Footprint (acre)
Treatment Processes		
Additional Aeration Ponds (4)	$8,697,000	7.8 +
Biolac® Wave Oxidation System	$4,258,000	Within 2 existing secondary ponds
Eimco Carrousel 3000 + 2 secondary clarifiers	$7,549,000	0.45
Activated Sludge + primary & secondary clarifiers	$8,794,000	0.23

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300 600

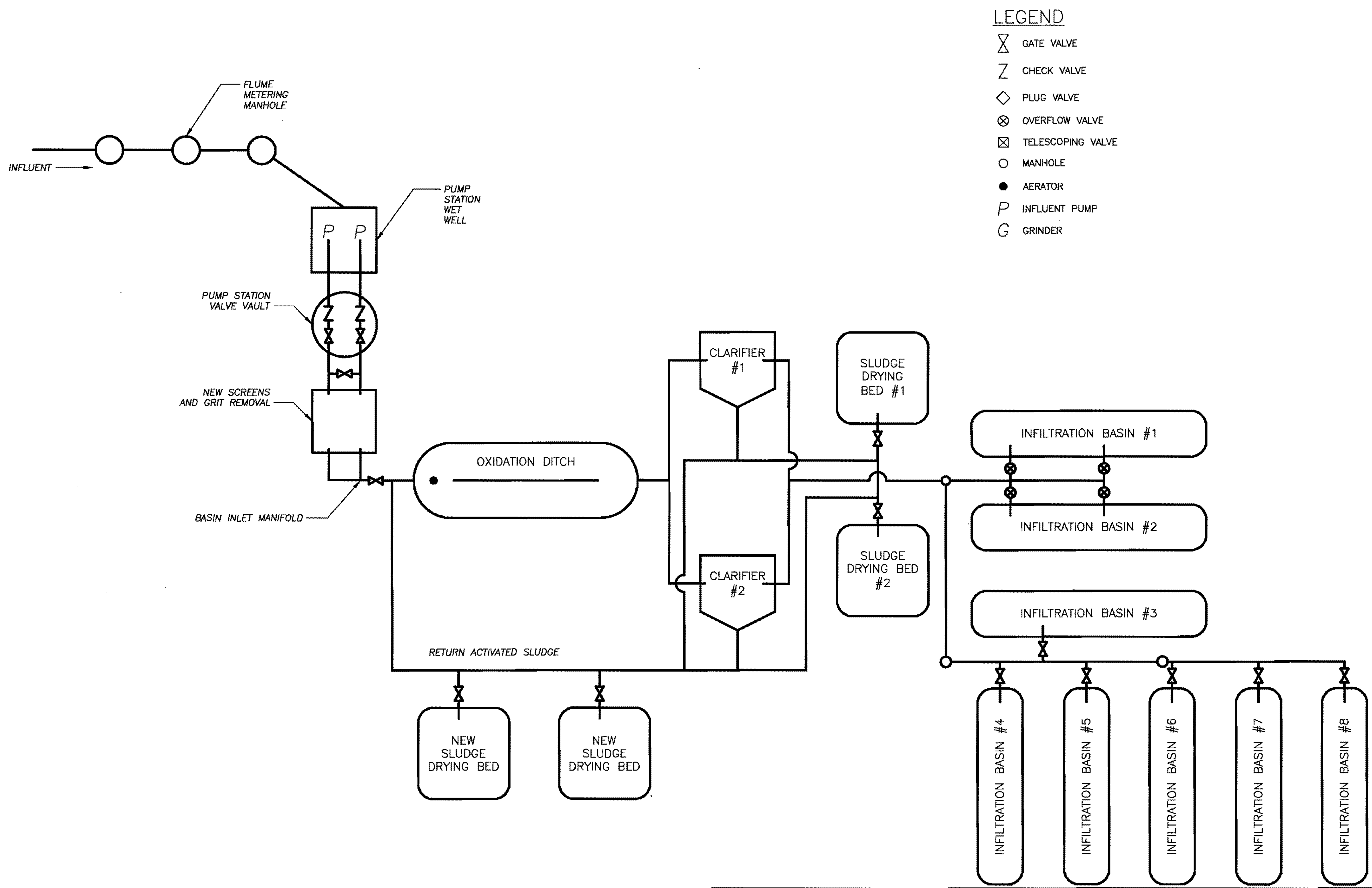
SCALE IN FEET

LEGEND

OXIDATION DITCH SYSTEM

ACTIVATED SLUDGE SYSTEM

FIGURE 8-7	NCSD SOUTHLAND WWTF MASTER PLAN	BEC PROJECT NO.	BOYLE ENGINEERING CORPORATION
	ALTERNATES 3 & 4: 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	BEC PROJECT NO.	FIGURE
OXIDATION DITCH PROCESS FLOW DIAGRAM	19996.17	8-8

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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 $2,560,000. 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. The total approximate footprint is 24 feet long by 14 feet wide. Each unit is approximately 10 feet wide, 20 feet long and 10 feet high. The estimated capital cost for the system is approximately $1,900,000.

<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 $1,550,000.

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 $4,000,000.

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.

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 6550 pounds of sludge would be produced per day during average flow conditions. Assuming 2% solids, the volume of sludge produced would be approximately 5140 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 80 days of storage each (approximately 4 months years). If solids reach 6% within the first year of storage, the ponds may store approximately 1 year 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 2015 (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 the current practice of onsite percolation. If the District chooses to continue onsite percolation as primary means of effluent disposal, or 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 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. Ponds 3 and 4 should be relined and retrofit with Biolac wave oxidation systems and integral clarifiers. 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 the preferred disposal method in the future, or as secondary or "wet-weather" disposal if other reuse options are pursued.

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 December 2007 and the WWTF is expected to exceed effluent quality limits (BOD_5 = 100 mg/L) in 2008 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 maintaining existing discharge practices. 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 operating close to its rated capacity, and could exceed permitted flow limits by the end of December, 2007, 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 is available;
- 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.
- 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 and integral clarifiers. The system should be constructed in two phases – Phase I would provide 75% of the 2030 capacity, 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; and
- 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.

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. Projects which are currently being constructed by the District are not included in this list, but are discussed in Section 7.0.
- 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 2007 project costs summarized below.

Table 10-1 Conceptual Cost Opinions for Facility Improvements

Component	2007 Project Cost	Year to be Completed	Escalated Project Cost to Midpoint of Construction
Frontage Rd. Trunk Main 21" Upgrade	$2,182,000	2009	$2,361,000
Influent Pump Station and Flowmeter Improvements	$967,000	2009	$1,046,000
Spiral Screening System	$468,000	2009	$507,000
Grit Removal System	$560,000	2009	$606,000

Feb 2007 ENR(CCI) =7880 in all Cost Opinions

Table 9-1 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

Component	2007 Project Cost	Year to be Completed	Escalated Project Cost to Midpoint of Construction
Phase I Biolac System (Capacity = 1.7 MGD MMF, or 75% of 2030 Demands)	$4,060,000	2009	$4,392,000
Phase I Drying Bed Improvements	$1,716,000	2009	$2,348,000
Phase II Biolac System (Capacity = 2.4 MGD MMF, or 100% of 2030 Demands)	$198,000	2015	$217,000
Phase II Drying Beds (2 New)	$1,540,000	2015	$2,108,000
Percolation Ponds	$1,363,000	2015	$1,865,000
Tertiary Filtration	$1,898,000	TBD	--
Chlorination System	$1,546,000	TBD	--

Table 9-2 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 $4,258,000 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 0.79 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, 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 Draft Water and Sewer Master Plan. At a minimum, the District should evaluate the percolation capacity of the existing WWTF property to handle flows beyond rated limits. The cost opinions above assume the maximum number and size of percolation pond facilities are constructed that will fit within the treatment plant site.

APPENDIX A

WASTE DISCHARGE ORDER
MONITORING & REPORTING PROGRAM

WDID No. 400104001

CALIFORNIA REGIONAL WATER QUALITY CONTROL BOARD
CENTRAL COAST REGION
81 Higuera Street Suite #200
San Luis Obispo, California 93401

ORDER NO. 97-75

WASTE DISCHARGE REQUIREMENTS FOR NIPOMO COMMUNITY SERVICES DISTRICT, SOUTHLAND WASTEWATER WORKS, SAN LUIS OBISPO COUNTY

The California Regional Water Quality Control Board, Central Coast Region (Board), finds;

1. Nipomo Community Services District (Discharger) owns and operates a municipal wastewater treatment facility which serves the town of Nipomo.

2. The Discharger filed a Report of Waste Discharge, in accordance with Section 13260 of the California Water Code, for authorization to increase discharges to the wastewater facility on January 24, 1996, and supplemented the Report of Waste Discharge with additional information on July 31, and September 30, 1996, and July 9, 1997. The discharge is currently regulated by Waste Discharge Requirements Order No. 84-56 adopted by the Board on July 13, 1984.

3. The treatment facility consists of influent grinding and aerated lagoons. Treated wastewater is discharged to 5.3 acres of percolation beds. Current design capacity is 360,000 gallons per day (1360 m^3/day), and design capacity of the expanded facilities is 900,000 gallons per day (3406 m^3/day), for which 14.5 acres total percolation basin area will be needed.

4. The percolation beds are located on level topography consisting of sandy soils. Perched ground water occurs at approximately 30 to 40 feet below ground surface, however the quality and direction of flow of this perched water is not clearly determined. A deeper ground water supply occurs at approximately 180 to 200 feet below ground surface and flows toward the southwest. Ground water constituent concentrations in the vicinity of the discharge are reportedly:

Total Dissolved Solids	260 mg/l
Sodium	36 mg/l
Chloride	36 mg/l
Nitrate (as N)	11 mg/l
Sulfate	22 mg/l
Boron	<0.1 mg/l

5. Nipomo Creek, tributary to the Santa Maria River, is located approximately 1/4 mile northeast of the discharge facilities and flows in a southeasterly direction. The wastewater facilities are not within the 100-year flood plain of Nipomo Creek.

6. The Water Quality Control Plan, Central Coast Basin (Basin Plan) was adopted by the Board on September 8, 1994. The Basin Plan incorporates statewide plans and policies by reference and contains a strategy for protecting beneficial uses of State waters.

7. Present and anticipated beneficial uses of ground water in the vicinity of the discharge include: Domestic, Municipal, Agricultural and Industrial Supply.

B. DISCHARGE LIMITATIONS

1. Effluent flow averaged over each month shall not exceed 360,000 gpd. After completion of the facility expansion, monthly flow shall not exceed 900,000 gpd. Incremental flow increases (600,000 gpd Phase I and 900,000 gpd Phase II) shall be allowed with written approval of the Executive Officer, after the Discharger demonstrates that expansion of the facilities is completed.

2. Effluent discharged to the disposal facilities shall not exceed the following parameters:

Parameter	Units	Month. Mean	Daily Maximum
BOD_5	mg/l	60	100
Suspended Solids	mg/l	60	100
Settleable Solids	ml/l	0.2	0.5
pH[A]	Within the range 6.5 to 8.4		
Dissolved Oxygen	mg/l	Minimum 1.0	

3. Wastewater treatment and disposal facilities shall be managed to exclude the public and posted to warn the public of the presence of wastewater.

4. Freeboard in all ponds shall exceed two feet at all times, unless the ponds are specifically designed for a different freeboard.

C. GROUND WATER LIMITATIONS

1. The treatment or discharge shall not cause nitrate concentrations in the ground water downgradient of the disposal facilities to exceed 10.0 mg/l (as N).

2. The discharge shall not cause a significant increase of mineral constituent concentrations in underlying ground waters, as determined by comparison of representative samples of groundwater collected from wells located upgradient and downgradient of the disposal area.

3. The discharge shall not cause concentrations of chemicals and radionuclides in groundwater to exceed limits set forth in Title 22, Chapter 15, Articles 4, 4.5, 5 and 5.5 of the California Code of Regulations.[A]

D. PROVISIONS

1. The requirements prescribed by this Order supersede requirements prescribed by Order No. 84-56 adopted by the Board on July 13, 1984. Order No. 84-56 "Waste Discharge Requirements for Nipomo Community Services District and Local Sewering Entity of San Luis Obispo County Service Area No. 1" is hereby rescinded.

2. Discharger shall comply with "Monitoring and Reporting Program No. 97-75", as specified by the Executive Officer.

3. Discharger shall comply with the attached "Standard Provisions and Reporting Requirements for Waste Discharge Requirements" dated January, 1984.

4. Discharger shall implement salts best management practices within the sewer service area to minimize salts contributions to the sewer system and subsequent discharge to the disposal facilities.

5. Discharger shall submit results and conclusions of the ground water investigation described in Monitoring and Reporting Program by October 24, 1998. If the investigation indicates the discharge may be impacting ground water in the vicinity, proposed mitigation measures (additional treatment and a time schedule) shall be submitted with the summary report. Incremental flow increases shall be authorized (as described in Discharge Limitation B.1.)

CALIFORNIA REGIONAL WATER QUALITY CONTROL BOARD
CENTRAL COAST REGION

MONITORING AND REPORTING PROGRAM NO. 97-75

FOR

NIPOMO COMMUNITY SERVICES DISTRICT,
SOUTHLAND WASTEWATER WORKS,
SAN LUIS OBISPO COUNTY

Influent Monitoring

Representative samples of the treatment plant influent shall be collected and analyzed as follows:

Parameter	Units	Type of Sample	Sampling and Analyzing Frequency
Maximum Flow	MGD	Metered	Daily
Average Flow	MGD	Calculated	Monthly

Effluent Monitoring

Representative samples of the treatment plant effluent shall be collected and analyzed as follows:

Parameter	Units	Type of Sample	Sampling and Analyzing Frequency
Settleable Solids	ml/l	Grab	Daily
Biochemical Oxygen Demand	mg/l	6-hr. Composite	Weekly
Suspended Solids	mg/l	6-hr. Composite	Weekly
Dissolved Oxygen	mg/l	Grab	Weekly
pH	pH Units	Grab	Weekly
Total Dissolved Solids	mg/l	6-hr. Composite	Semi-annually (Jan/July)
Sodium	mg/l	6-hr. Composite	Semi-annually (Jan/July)
Chloride	mg/l	6-hr. Composite	Semi-annually (Jan/July)
Total Nitrogen (as N)	mg/l	6-hr. Composite	Semi-annually (Jan/July)

Ground Water Monitoring

Discharger shall install new monitoring wells upgradient and downgradient of the disposal area which facilitate representative sampling from the first available ground water. Discharger shall be responsible for determining direction of ground water flow and level to determine the appropriate location and depth of upgradient and downgradient monitoring wells. The monitoring wells shall meet or exceed well standards contained in the Department of Water Resources Bulletins 74-81 and 74-90. Discharger shall also comply with the monitoring well reporting provisions of Section 13750 through 13755 of the California Water Code.

APPENDIX B

CALCULATIONS

BOYLE ENGINEERING

ENGINEERS, SURVEYORS, PLANNERS

BY:	EKM	**DATE:**	12/1/2006	**SUBJECT:**	SOUTHLAND WWTF	**JOB NO.**	19996.17
CHKD. BY:		**DATE:**			EXISTING TREATMENT CAPACITY		

BOD Removal in Ponds

$$\frac{C_n}{C_o} = \frac{1}{1+(k/nt)^n}$$ First order for *n* equally sized lagoons in series (ref. M&E p 843)

$$C = \frac{C_o}{1+(kV/Q)}$$ First order for each lagoon with unique volume and/ or removal rate (ref. M&E p 843)

Effluent BOD_5 Goal

C = 80 mg/L* (conserv. assumption of 80% of eff. Limitation)

Influent BOD_5

C_o = 350 mg/ L (Dec05 - Aug06 90th percentile BOD_5)

Estimated Inf. BOD_u = 514.5 mg/ L (inf. BOD_5 x 1.47)

$k_T = k_{20}(1.036)^{T-20}$

k_{20} =	0.276 d^{-1} (first-order rate constant at 20°C)	
T_L =	49.4 °F (Approximate ground temp., Dec)	
=	9.7 °C	282.8 °K
T_H =	71.5 °F (Approximate ground temp., July)	
=	21.9 °C	295.1 °K
k_L =	0.19 d^{-1}	
k_H =	0.30 d^{-1}	

Flows (current 2006)

Jul-06	0.791 mgd	= Q_H (July for conservative flow)
Dec-06	0.547 mgd	= Q_L
Permitted MMF	0.900 mgd	= Q_{MMF}

Volumes

Primary	=	295,700 ft^3
	=	2,211,984 gallons

*Fraction of Secondary Ponds for clarification: 0.4

Secondary	=	250,380 ft^3 (total volume available for aeration)
	=	1,872,968 gallons

BOYLE ENGINEERING

ENGINEERS, SURVEYORS, PLANNERS

BY: EKM **DATE:** 12/1/2006 **SUBJECT:** SOUTHLAND WWTF **JOB NO.** 19996.17

CHKD. BY: ________ **DATE:** ________ EXISTING TREATMENT CAPACITY

Aeration requirement (oxygen demand):

O_2 demand (lb/ day) = Co x 1.5 x Q_{Ave} x 8.34e-6 Note: 1mg/L = 8.34e-6 lb/gal;

Calculated oxygen demands:

Cu = 525 mg/ L (1.5 x Co)

Q_L = 547,000 gpd
Q_H = 791,000 gpd
Q_{MMF} = 900,000 gpd

Oxygen demand for low flow rate:	**2,395.0 lb O_2/ day**
Oxygen demand for high flow rate:	**3,463.4 lb O_2/ day**
Oxygen demand for permit MMFflow rate:	**3,940.7 lb O_2/ day**

BOYLE ENGINEERING

ENGINEERS, SURVEYORS, PLANNERS

BY:	EKM	**DATE:**	12/1/2006	**SUBJECT:**	SOUTHLAND WWTF	**JOB NO.**	19996.17
CHKD. BY:		**DATE:**			EXISTING TREATMENT CAPACITY		

Current System Aeration Capacity

Calculate actual oxygen transfer rate for low-speed surface aerators

$$N = N_o \times \frac{B\,C_W - C_l}{C_{s\,20}} \times 1.024^{T-20} \times a$$

N_o = 2.5 lb O_2/ HP.hr (O_2 transferred under std. cond. for low-speed surface)
B = 1 (salinity-surface tension factor, typically 1)
C_{WL} = 11.0 mg/ L (oxygen saturation concentration at temp 9.7C and 300 ft, M&E)
C_{WH} = 8.5 mg/ L (oxygen saturation concentration at temp 21.9C and 300 ft, M&E)
C_l = 2.0 mg/ L (operating oxygen concentration)
Cs_{20} = 9.08 mg/ L (oxygen saturation concentration at temp 20C)
T_L = 49.4 °F (Approximate ground temp., Dec)
= 9.7 °C
T_H = 71.5 °F (Approximate ground temp., July)
= 21.9 °C
a = 0.82 oxygen transfer correction factor for municipal wastewater

N_L = **1.95 lb O_2/ HP.hr (low temp)**
N_H = **2.01 lb O_2/ HP.hr (high temp)**

Available HP = 110 HP (for surface aerators)

$AOTR_L$ = **5140.8 lb O_2/ day (low temp)**
$AOTR_H$ = **5295.8 lb O_2/ day (high temp)**

BOYLE ENGINEERING

ENGINEERS, SURVEYORS, PLANNERS

BY: EKM	**DATE:** 12/1/2006	**SUBJECT:**	SOUTHLAND WWTF	**JOB NO.**	19996.17
CHKD. BY:	**DATE:**		EXISTING TREATMENT CAPACITY		

Four Ponds in Series - Winter Season (Low temp & low flow condition)

Pond #1	V_1 =	2,211,984	gallons
	Q =	547,000	gpd
	k_L =	0.19	d^{-1}
	t =	4.04	days
	C_o =	350	mg/L
	C_1 =	197.2	mg/ L
Pond #2	V_2 =	2,211,984	gallons
	Q =	547,000	gpd
	k_L =	0.19	d^{-1}
	t =	4.04	days
	C_1 =	197.2	mg/ L
	C_2 =	111.2	mg/ L
Pond #3	V_3 =	1,872,968	gallons
	Q =	547,000	gpd
	k_L =	0.19	d^{-1}
	t =	3.42	days
	C_2 =	111.2	mg/ L
	C_3 =	67.1	mg/ L
Pond #4	V_4 =	1,872,968	gallons
	Q =	547,000	gpd
	k_L =	0.19	d^{-1}
	t =	3.42	days
	C_3 =	67.1	mg/ L
	C_4 =	**40.5**	**mg/ L**

total retention time = **14.94**

% reduction = **88%**

BOYLE ENGINEERING

ENGINEERS, SURVEYORS, PLANNERS

BY: EKM **DATE:** 12/1/2006 **SUBJECT:** SOUTHLAND WWTF **JOB NO.** 19996.17
CHKD. BY: **DATE:** EXISTING TREATMENT CAPACITY

Four Ponds in Series - Summer Season (High temp & high flow condition)

Pond #1	V_1 =	2,211,984	gallons
	Q =	791,000	gpd
	k_H =	0.30	d^{-1}
	t =	2.80	days
	C_o =	350	mg/L
	C_1 =	191.6	mg/ L
Pond #2	V_2 =	2,211,984	gallons
	Q =	791,000	gpd
	k_H =	0.30	d^{-1}
	t =	2.80	days
	C_1 =	191.6	mg/ L
	C_2 =	104.9	mg/ L
Pond #3	V_3 =	1,872,968	gallons
	Q =	791,000	gpd
	k_H =	0.30	d^{-1}
	t =	2.37	days
	C_2 =	104.9	mg/ L
	C_3 =	61.7	mg/ L
Pond #4	V_4 =	1,872,968	gallons
	Q =	791,000	gpd
	k_H =	0.30	d^{-1}
	t =	2.37	days
	C_3 =	61.7	mg/ L
	C_4 =	**36.3**	**mg/ L**

total retention time = **10.33**

% reduction = **90%**

BOYLE ENGINEERING

ENGINEERS, SURVEYORS, PLANNERS

BY: EKM **DATE:** 12/1/2006 **SUBJECT:** SOUTHLAND WWTF **JOB NO.** 19996.17

CHKD. BY: ______ **DATE:** ______ EXISTING TREATMENT CAPACITY

Four Ponds in Series - MMF Summer Season (High temp & MMF flow condition)

Pond #1	V_1 =	2,211,984	gallons
	Q =	900,000	gpd
	k_H =	0.30	d^{-1}
	t =	2.46	days
	C_o =	350	mg/L
	C_1 =	202.7	mg/ L
Pond #2	V_2 =	2,211,984	gallons
	Q =	900,000	gpd
	k_H =	0.30	d^{-1}
	t =	2.46	days
	C_1 =	202.7	mg/ L
	C_2 =	117.4	mg/ L
Pond #3	V_3 =	1,872,968	gallons
	Q =	900,000	gpd
	k_H =	0.30	d^{-1}
	t =	2.08	days
	C_2 =	117.4	mg/ L
	C_3 =	72.7	mg/ L
Pond #4	V_4 =	1,872,968	gallons
	Q =	900,000	gpd
	k_H =	0.30	d^{-1}
	t =	2.08	days
	C_3 =	72.7	mg/ L
	C_4 =	**45.0**	**mg/ L**

total retention time = **9.08**

% reduction = **87%**

BOYLE ENGINEERING

ENGINEERS, SURVEYORS, PLANNERS

BY: EKM **DATE:** 12/1/2006 **SUBJECT:** SOUTHLAND WWTF **JOB NO.** 19996.17

CHKD. BY: ______ **DATE:** ______ EXISTING TREATMENT CAPACITY

Two Ponds in Series - Two parallel flow trains - Winter Season (Low temp & low flow conditon):

Pond #1	V_1 =	2,211,984	gallons
	Q =	273,500	gpd
	k_L =	0.19	d^{-1}
	t =	8.09	days
	C_o =	350	mg/L
	C_1 =	137.3	mg/ L
Pond #4	V_3 =	1,872,968	gallons
	Q =	273,500	gpd
	k_L =	0.19	d^{-1}
	t =	6.85	days
	C_1 =	137.3	mg/ L
	C_3 =	**59.4**	**mg/ L**
Pond #2	V_2 =	2,211,984	gallons
	Q =	273,500	gpd
	k_L =	0.19	d^{-1}
	t =	8.09	days
	C_o =	350	mg/L
	C_2 =	137.3	mg/ L
Pond #3	V_4 =	1,872,968	gallons
	Q =	273,500	gpd
	k_L =	0.19	d^{-1}
	t =	6.85	days
	C_2 =	137.3	mg/ L
	C_4 =	**59.4**	**mg/ L**
	% reduction =	**83%**	

total retention time = **14.94**

BOYLE ENGINEERING

ENGINEERS, SURVEYORS, PLANNERS

BY:	EKM	**DATE:**	12/1/2006	**SUBJECT:**	SOUTHLAND WWTF	**JOB NO.**	19996.17
CHKD. BY:		**DATE:**			EXISTING TREATMENT CAPACITY		

Two Ponds in Series, Two parallel flow trains - Summer Season (High temp & high flow condition)

Pond #1	V_1 =	2,211,984	gallons
	Q =	395,500	gpd
	k_H =	0.30	d^{-1}
	t =	5.59	days
	C_o =	350	mg/L
	C_1 =	131.9	mg/ L
Pond #4	V_3 =	1,872,968	gallons
	Q =	395,500	gpd
	k_H =	0.30	d^{-1}
	t =	4.74	days
	C_1 =	131.9	mg/ L
	C_3 =	**55.0**	**mg/ L**
Pond #2	V_2 =	2,211,984	gallons
	Q =	395,500	gpd
	k_H =	0.30	d^{-1}
	t =	5.59	days
	C_o =	350	mg/L
	C_2 =	131.9	mg/ L
Pond #3	V_4 =	1,872,968	gallons
	Q =	395,500	gpd
	k_H =	0.30	d^{-1}
	t =	4.74	days
	C_2 =	131.9	mg/ L
	C_4 =	**55.0**	**mg/ L**

total retention time = **10.33**

% reduction = **84%**

BOYLE ENGINEERING

ENGINEERS, SURVEYORS, PLANNERS

BY: EKM **DATE:** 12/1/2006 **SUBJECT:** SOUTHLAND WWTF **JOB NO.** 19996.17

CHKD. BY: ______ **DATE:** ______ EXISTING TREATMENT CAPACITY

Two Ponds in Series, Two parallel flow trains - MMF Summer Season (High temp & MMF flow cond.)

Pond #1	V_1 =	2,211,984 gallons
	Q =	450,000 gpd
	k_H =	0.30 d^{-1}
	t =	4.92 days
	C_o =	350 mg/L
	C_1 =	142.7 mg/ L
Pond #4	V_3 =	1,872,968 gallons
	Q =	450,000 gpd
	k_H =	0.30 d^{-1}
	t =	4.16 days
	C_1 =	142.7 mg/ L
	C_3 =	**64.0 mg/ L**
Pond #2	V_2 =	2,211,984 gallons
	Q =	450,000 gpd
	k_H =	0.30 d^{-1}
	t =	4.92 days
	C_o =	350 mg/L
	C_2 =	142.7 mg/ L
Pond #3	V_4 =	1,872,968 gallons
	Q =	450,000 gpd
	k_H =	0.30 d^{-1}
	t =	4.16 days
	C_2 =	142.7 mg/ L
	C_4 =	**64.0 mg/ L**

total retention time = **9.08**

% reduction = **82%**

*M&E Reference: Wastewater Engineering Treatment and Reuse, 4th Edition

Boyle Engineering Corporation

BY:	EKM	**DATE:**	10/30/2006	**SUBJECT**	Southland WWTF Master Plan	**JOB NO:**	19996.17
CHKD. BY:		**DATE:**			Solids Production Calculations		

Determine: Volume of solids added to ponds in past 5 years

Assumptions:

AAF = 0.60 mgd, Average TSS_{in} = 265 mg/L, Average TSS_{out} = 40 mg/L

1) Total volume of wastewater treated in past 5 years

$V = Q \times t$

$V = 0.60 \text{ mgd} \times 5 \text{ yrs} \times 365 \text{ days/yr}$

$V = 1095$ Mgal

2) Mass of TSS removed

$\text{Mass} = (TSS_{in} - TSS_{out}) \times V \times (8.34 \text{ lb/Mgal} \times \text{mg/L})$

$\text{Mass} = (265 - 40) \times (1095) \times (8.34)$

= 2,054,768 lbs

= 410,954 lbs/yr

3) Mass of volatile and fixed solids

$\text{Mass}_{VSS} = 0.70 \times TSS$

$= 0.70 \times (2{,}054{,}768)$

= 1,438,337 lbs

= 287,667 lbs/yr

$\text{Mass}_{Fixed} = \text{Mass}_{TSS} - \text{Mass}_{VSS}$

$= 2{,}054{,}768 - 1{,}438{,}337$

= 616,430 lbs

= 123,286 lbs/yr

4) Amount of accumulation at the end of 5 years

Assume 60% VSS reduction occurs within 1 year

$(VSS)_t = [0.7 + 0.4(t-1)] \times VSS$

$= [0.7 + 0.4(5-1)] \times 287{,}667$

= 661,635 lbs

5) Total mass of solids

$\text{Mass}_{Total} = \text{Mass}_{Fixed} + \text{Mass}_{Accumulated}$

$= 616{,}430 + 661{,}635$

= 1,278,065 lbs

6) Volume of solids (assume 15% solids and density = 1.06*8.34 lb/gal)

$V_{Total} = \text{Mass}_{Total} / (0.15 \cdot \text{density})$

= 963,807 gal

Boyle Engineering Corporation

BY: EKM **DATE:** 10/30/2006 **SUBJECT** Southland WWTF Master Plan **JOB NO:** 19996.17
CHKD. BY: ____ **DATE:** ________ Solids Production Calculations

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Potential percentage of solid volume in ponds from past 5 years

Total pond volume (taken from NCSD Southland O&M Manual, July 2000)

Liquid volume = 2 @ 295,700 cf & 2 @ 417,300 cf

Sludge volume = 2 @ 0.5 Mgal & 2 @ 0.7 Mgal

V_{Total} = [2 x 295,700 + 2 x 417,300] x 7.481 gal/cf + 2 x 500,000 + 2 x 700, 000

V_{Total} = 13,067,906 gal

% of solids in pond = $\frac{963,807}{13,067,906}$

= 0.07

= 7% from past 5 years

BOYLE ENGINEERING

ENGINEERS, SURVEYORS, PLANNERS

BY: EKM	**DATE:** 12/1/2006	**SUBJECT:** SOUTHLAND WWTF	**JOB NO.** 19996.17
CHKD. BY:	**DATE:**	TREATMENT CAPACITY FOR FUTURE FLOWS	

BOD Removal in Ponds

$$\frac{C_n}{C_o} = \frac{1}{1+(k/nt)^n}$$ First order for *n* equally sized lagoons in series (ref. M&E p 843)

$$C = \frac{C_o}{1+(kV/Q)}$$ First order for each lagoon with unique volume and/ or removal rate (ref. M&E p 843)

Effluent BOD_5 Goal

C = 80 mg/L* (conserv. assumption of 80% of eff. Limitation)

Influent BOD_5

C_o = 350 mg/ L (Dec 05 - Aug 06 90th percentile BOD_5)

Estimated Inf. BOD_u = 514.5 mg/ L (inf. BOD_5 x 1.47)

$k = k_{20}(1.036)^{T-20}$

k_{20} = 0.276 d^{-1} (first-order rate constant at 20°C)

T_L = 49.4 °F (Approximate ground temp., Dec)

= 9.7 °C 282.8 °K

T_H = 71.5 °F (Approximate ground temp., July)

= 21.9 °C 295.1 °K

k_L = 0.19 d^{-1}

k_H = 0.30 d^{-1}

Flows (projected for 2030)

PDF 3.34 mgd = Q_H

AAF 1.67 mgd = Q_L

MMF 2.24 mgd = Q_{MMF}

Volumes

Primary = 295,700 ft^3

= 2,211,984 gallons

*Fraction of Secondary Ponds for clarification: 0.4

Secondary = 250,380 ft^3 (total volume available for aeration)

= 1,872,968 gallons

Volume of Secondary Ponds without baffle

V = 417300 ft^3

= 3,121,613 gallons

Aeration requirement (oxygen demand)

O_2 demand (lb/ day) = Co x 1.5 x Q_{Ave} x 8.34e-6 Note: 1mg/L = 8.34e-6 lb/gal;

Calculated oxygen demands

Cu = 525 mg/ L (1.5 x Co)

Q_L = 1,670,000 gpd

Q_H = 3,340,000 gpd

Q_{MMF} = 2,237,800 gpd

Oxygen demand for low flow rate: **7,312.1 lb O_2/ day**

Oxygen demand for high flow rate: **14,624.2 lb O_2/ day**

Oxygen demand for permit MMFflow rate: **9,798.2 lb O_2/ day**

Current System Aeration Capacity

Calculate actual oxygen transfer rate for low-speed surface aerators

$$N = N_o \times \frac{B\,C_W - C_l}{C_{S20}} \times 1.024^{T-20} \times a$$

N_o = 2.5 lb O_2/ HP.hr (O_2 transferred under std. cond. for low-speed surface)
B = 1 (salinity-surface tension factor, typically 1)
C_{WL} = 11.0 mg/ L (oxygen saturation concentration at temp 9.7C and 300 ft, M&E)
C_{WH} = 8.5 mg/ L (oxygen saturation concentration at temp 21.9C and 300 ft, M&E)
C_l = 2.0 mg/ L (operating oxygen concentration)
Cs_{20} = 9.08 mg/ L (oxygen saturation concentration at temp 20C)
T_L = 49.4 °F (Approximate ground temp., Dec)
= 9.7 °C
T_H = 71.5 °F (Approximate ground temp., July)
= 21.9 °C
a = 0.82 oxygen transfer correction factor for municipal wastewater

N_L = 1.95 lb O_2/ HP.hr (low temp)
N_H = 2.01 lb O_2/ HP.hr (high temp)

Available HP = 110 HP

$AOTR_L$ = 5140.8 lb O_2/ day (low temp)
$AOTR_H$ = 5295.8 lb O_2/ day (high temp)

Calculate amount of horsepower required to satisfy oxygen demand

Oxygen demand for low flow rate: 7,312.1 lb O_2/ day
Oxygen demand for high flow rate: 14,624.2 lb O_2/ day
Oxygen demand for max month flow rate: 9,798.2 lb O_2/ day

N_L = 1.95 lb O_2/ HP.hr (low temp)
N_H = 2.01 lb O_2/ HP.hr (high temp)

For high flow rate
Total HP = **315.0 HP**

$AOTR_L$ = 14721.3 lb O_2/ day (low temp)
$AOTR_H$ = 15165.2 lb O_2/ day (high temp)

For max month flow rate
Total HP = **210.0 HP**

$AOTR_L$ = 9814.2 lb O_2/ day (low temp)
$AOTR_H$ = 10110.1 lb O_2/ day (high temp)

Ponds in Series - Winter Season (Low temp & low flow condition)

Current System Under 2030 Flow Conditions

Pond #1	V_1 =	2,211,984	gallons
	Q =	1,670,000	gpd
	k_L =	0.19	d^{-1}
	t =	1.32	days
	C_o =	350	mg/L
	C_1 =	279.2	mg/ L
Pond #2	V_2 =	2,211,984	gallons
	Q =	1,670,000	gpd
	k_L =	0.19	d^{-1}
	t =	1.32	days
	C_1 =	279.2	mg/ L
	C_2 =	222.7	mg/ L
Pond #3	V_3 =	1,872,968	gallons
	Q =	1,670,000	gpd
	k_L =	0.19	d^{-1}
	t =	1.12	days
	C_2 =	222.7	mg/ L
	C_3 =	183.3	mg/ L
Pond #4	V_4 =	1,872,968	gallons
	Q =	1,670,000	gpd
	k_L =	0.19	d^{-1}
	t =	1.12	days
	C_3 =	183.3	mg/ L
	C_4 =	**150.9**	**mg/ L**

current % reduction = **57%** total retention time = **4.89** days

Ponds in Series - Winter Season (Low temp & low flow condition)

Remove Baffles from Ponds 3 & 4
Add two ponds, V = 3,121,613 gallons each

Pond #1	V_1 =	2,211,984	gallons
	Q =	1,670,000	gpd
	k_L =	0.19	d^{-1}
	t =	1.32	days
	C_o =	350	mg/L
	C_1 =	279.2	mg/ L
Pond #2	V_2 =	2,211,984	gallons
	Q =	1,670,000	gpd
	k_L =	0.19	d^{-1}
	t =	1.32	days
	C_1 =	279.2	mg/ L
	C_2 =	222.7	mg/ L
Pond #3	V_3 =	3,121,613	gallons
	Q =	1,670,000	gpd
	k_L =	0.19	d^{-1}
	t =	1.87	days
	C_2 =	222.7	mg/ L
	C_3 =	164.0	mg/ L
Pond #4	V_4 =	3,121,613	gallons
	Q =	1,670,000	gpd
	k_L =	0.19	d^{-1}
	t =	1.87	days
	C_3 =	164.0	mg/ L
	C_4 =	**120.8**	**mg/ L**
New Pond 5	V_5 =	3,121,613	gallons
	Q =	1,670,000	gpd
	k_L =	0.19	d^{-1}
	t =	1.87	days
	C_4 =	120.8	mg/ L
	C_5 =	**88.9**	**mg/ L**
New Pond 6	V_6 =	3,121,613	gallons
	Q =	1,670,000	gpd
	k_L =	0.19	d^{-1}
	t =	1.87	days
	C_5 =	88.9	mg/ L
	C_6 =	**65.5**	**mg/ L**

% reduction **81%** total retention time = **10.13 days**

For ponds in series,
Two additional ponds would treat the wastewater to acceptable levels during low temp, low flow conditions

BOYLE ENGINEERING

ENGINEERS, SURVEYORS, PLANNERS

BY:	EKM	**DATE:**	12/1/2006	**SUBJECT:** SOUTHLAND WWTF	**JOB NO.** 19996.17
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Ponds in Series - Summer Season (High temp & high flow condition)

Current System Under 2030 Flow Conditions

Pond #1	V_1 =	2,211,984	gallons			
	Q =	3,340,000	gpd			
	k_H =	0.30	d^{-1}			
	t =	0.66	days			
	C_o =	350	mg/L			
	C_1 =	292.7	mg/ L			
Pond #2	V_2 =	2,211,984	gallons			
	Q =	3,340,000	gpd			
	k_H =	0.30	d^{-1}			
	t =	0.66	days			
	C_1 =	292.7	mg/ L			
	C_2 =	244.8	mg/ L			
Pond #3	V_3 =	1,872,968	gallons			
	Q =	3,340,000	gpd			
	k_H =	0.30	d^{-1}			
	t =	0.56	days			
	C_2 =	244.8	mg/ L			
	C_3 =	210.0	mg/ L			
Pond #4	V_4 =	1,872,968	gallons			
	Q =	3,340,000	gpd			
	k_H =	0.30	d^{-1}			
	t =	0.56	days			
	C_3 =	210.0	mg/ L			
	C_4 =	**180.1**	**mg/ L**	total retention time =	**2.45**	**days**
	% reduction =	**49%**				

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Ponds in Series - Summer Season (High temp & high flow condition)

Remove Baffles from Ponds 3 & 4

Add two ponds, V = 3,121,613 gallons each

Pond #1	V_1 =	2,211,984	gallons
	Q =	3,340,000	gpd
	k_H =	0.30	d^{-1}
	t =	0.66	days
	C_o =	350	mg/L
	C_1 =	292.7	mg/ L
Pond #2	V_2 =	2,211,984	gallons
	Q =	3,340,000	gpd
	k_H =	0.30	d^{-1}
	t =	0.66	days
	C_1 =	292.7	mg/ L
	C_2 =	244.8	mg/ L
Pond #3	V_3 =	3,121,613	gallons
	Q =	3,340,000	gpd
	k_H =	0.30	d^{-1}
	t =	0.93	days
	C_2 =	244.8	mg/ L
	C_3 =	191.8	mg/ L
Pond #4	V_4 =	3,121,613	gallons
	Q =	3,340,000	gpd
	k_H =	0.30	d^{-1}
	t =	0.93	days
	C_3 =	191.8	mg/ L
	C_4 =	**150.3**	**mg/ L**
New Pond 5	V_5 =	3,121,613	gallons
	Q =	3,340,000	gpd
	k_H =	0.30	d^{-1}
	t =	0.93	days
	C_4 =	150.3	mg/ L
	C_5 =	**117.7**	**mg/ L**
New Pond 6	V_6 =	3,121,613	gallons
	Q =	3,340,000	gpd
	k_H =	0.30	d^{-1}
	t =	0.93	days
	C_5 =	117.7	mg/ L
	C_6 =	**92.2**	**mg/ L**

total retention time = **5.06 days**

Two ponds don't reach effluent goal, try additional pond:

New Pond 7	V_7 =	3,121,613	gallons
	Q =	3,340,000	gpd
	k_H =	0.30	d^{-1}
	t =	0.93	days
	C_6 =	92.2	mg/ L
	C_7 =	**72.3**	**mg/ L**

% reduction = **79%** total retention time = **6.00 days**

For ponds in series,
Three additional ponds would treat the wastewater to acceptable levels during high temp, high flow conditions

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Ponds in Series - MMF Summer Season (High temp & MMF flow condition)

Current System Under 2030 Flow Conditions

Pond #1	V_1 =	2,211,984	gallons
	Q =	2,237,800	gpd
	k_H =	0.30	d^{-1}
	t =	0.99	days
	C_o =	350	mg/L
	C_1 =	270.8	mg/ L
Pond #2	V_2 =	2,211,984	gallons
	Q =	2,237,800	gpd
	k_H =	0.30	d^{-1}
	t =	0.99	days
	C_1 =	270.8	mg/ L
	C_2 =	209.6	mg/ L
Pond #3	V_3 =	1,872,968	gallons
	Q =	2,237,800	gpd
	k_H =	0.30	d^{-1}
	t =	0.84	days
	C_2 =	209.6	mg/ L
	C_3 =	168.0	mg/ L
Pond #4	V_4 =	1,872,968	gallons
	Q =	2,237,800	gpd
	k_H =	0.30	d^{-1}
	t =	0.84	days
	C_3 =	168.0	mg/ L
	C_4 =	**134.7**	**mg/ L**
	% reduction =	**62%**	

total retention time = **3.65 days**

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Ponds in Series - MMF Summer Season (High temp & MMF flow condition)

Remove Baffles from Ponds 3 & 4

Add two ponds, V = 3,121,613 gallons each

Pond #1	V_1 =	2,211,984	gallons
	Q =	2,237,800	gpd
	k_H =	0.30	d^{-1}
	t =	0.99	days
	C_o =	350	mg/L
	C_1 =	270.8	mg/ L
Pond #2	V_2 =	2,211,984	gallons
	Q =	2,237,800	gpd
	k_H =	0.30	d^{-1}
	t =	0.99	days
	C_1 =	270.8	mg/ L
	C_2 =	209.6	mg/ L
Pond #3	V_3 =	3,121,613	gallons
	Q =	2,237,800	gpd
	k_H =	0.30	d^{-1}
	t =	1.39	days
	C_2 =	209.6	mg/ L
	C_3 =	148.4	mg/ L
Pond #4	V_4 =	3,121,613	gallons
	Q =	2,237,800	gpd
	k_H =	0.30	d^{-1}
	t =	1.39	days
	C_3 =	148.4	mg/ L
	C_4 =	**105.1**	**mg/ L**
New Pond 5	V_5 =	3,121,613	gallons
	Q =	2,237,800	gpd
	k_H =	0.30	d^{-1}
	t =	1.39	days
	C_4 =	105.1	mg/ L
	C_5 =	**74.4**	**mg/ L**
	% reduction =	**79%**	

total retention time = **6.16 days**

For ponds in series,

One additional pond would treat the wastewater to acceptable levels during high temp, max month flow conditions

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Two parallel flow trains - Winter Season (Low temp & low flow condition)

Current System Under 2030 Flow Conditions

Pond #1	V_1 =	2,211,984	gallons
	Q =	835,000	gpd
	k_L =	0.19	d^{-1}
	t =	2.65	days
	C_o =	350	mg/L
	C_1 =	232.2	mg/ L
Pond #4	V_4 =	1,872,968	gallons
	Q =	835,000	gpd
	k_L =	0.19	d^{-1}
	t =	2.24	days
	C_1 =	232.2	mg/ L
	C_4 =	**162.4**	**mg/ L**
Pond #2	V_2 =	2,211,984	gallons
	Q =	835,000	gpd
	k_L =	0.19	d^{-1}
	t =	2.65	days
	C_o =	350	mg/L
	C_2 =	232.2	mg/ L
Pond #3	V_3 =	1,872,968	gallons
	Q =	835,000	gpd
	k_L =	0.19	d^{-1}
	t =	2.24	days
	C_2 =	232.2	mg/ L
	C_3 =	**162.4**	**mg/ L**

total retention time = **4.89 days**

% reduction = **54%**

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Two parallel flow trains - Winter Season (Low temp & low flow condition)

Remove Baffles from Ponds 3 & 4

Add two ponds, V = 3,121,613 gallons each

Pond #1	V_1 =	2,211,984	gallons
	Q =	835,000	gpd
	k_L =	0.19	d^{-1}
	t =	2.65	days
	C_o =	350	mg/L
	C_1 =	232.2	mg/ L
Pond #4	V_4 =	3,121,613	gallons
	Q =	835,000	gpd
	k_L =	0.19	d^{-1}
	t =	3.74	days
	C_1 =	232.2	mg/ L
	C_4 =	**135.3**	**mg/ L**
New Pond 5	V_5 =	3,121,613	gallons
	Q =	835,000	gpd
	k_L =	0.19	d^{-1}
	t =	3.74	days
	C_4 =	135.3	mg/ L
	C_5 =	**78.9**	**mg/ L**
Pond #2	V_2 =	2,211,984	gallons
	Q =	835,000	gpd
	k_L =	0.19	d^{-1}
	t =	2.65	days
	C_o =	350	mg/L
	C_2 =	232.2	mg/ L
Pond #3	V_3 =	3,121,613	gallons
	Q =	835,000	gpd
	k_L =	0.19	d^{-1}
	t =	3.74	days
	C_2 =	232.2	mg/ L
	C_3 =	**135.3**	**mg/ L**
New Pond 6	V_6 =	3,121,613	gallons
	Q =	835,000	gpd
	k_L =	0.19	d^{-1}
	t =	3.74	days
	C_3 =	135.3	mg/ L
	C_6 =	**78.9**	**mg/ L**

total retention time = **10.13 days**

% reduction = **77%**

For two parallel flow trains,
Two additional ponds would treat the wastewater to acceptable levels during low temp, low flow conditions

Two parallel flow trains - Summer Season (High temp & high flow condition) -

Current System Under 2030 Flow Conditions

Pond #1	V_1 =	2,211,984	gallons
	Q =	1,670,000	gpd
	k_H =	0.30	d^{-1}
	t =	1.32	days
	C_o =	350	mg/L
	C_1 =	251.5	mg/ L
Pond #4	V_3 =	1,872,968	gallons
	Q =	1,670,000	gpd
	k_H =	0.30	d^{-1}
	t =	1.12	days
	C_1 =	251.5	mg/ L
	C_3 =	**188.9**	**mg/ L**
Pond #2	V_2 =	2,211,984	gallons
	Q =	1,670,000	gpd
	k_H =	0.30	d^{-1}
	t =	1.32	days
	C_o =	350	mg/L
	C_2 =	251.5	mg/ L
Pond #3	V_4 =	1,872,968	gallons
	Q =	1,670,000	gpd
	k_H =	0.30	d^{-1}
	t =	1.12	days
	C_2 =	251.5	mg/ L
	C_4 =	**188.9**	**mg/ L**
	% reduction =	**46%**	

total retention time = **2.45 days**

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Two parallel flow trains - Summer Season (High temp & high flow condition)

Remove Baffles from Ponds 3 & 4

Add four ponds, V = 3,121,613 gallons each

Pond #1	V_1 =	2,211,984	gallons
	Q =	1,670,000	gpd
	k_H =	0.30	d^{-1}
	t =	1.32	days
	C_o =	350	mg/L
	C_1 =	251.5	mg/ L
Pond #4	V_3 =	3,121,613	gallons
	Q =	1,670,000	gpd
	k_H =	0.30	d^{-1}
	t =	1.87	days
	C_1 =	251.5	mg/ L
	C_3 =	162.0	mg/ L
New Pond 1	V_3 =	3,121,613	gallons
	Q =	1,670,000	gpd
	k_H =	0.30	d^{-1}
	t =	1.87	days
	C_3 =	162.0	mg/ L
	C_5 =	**104.3**	**mg/ L**
New Pond 2	V_3 =	3,121,613	gallons
	Q =	1,670,000	gpd
	k_H =	0.30	d^{-1}
	t =	1.87	days
	C_5 =	104.3	mg/ L
	C_7 =	**67.2**	**mg/ L**
Pond #2	V_2 =	2,211,984	gallons
	Q =	1,670,000	gpd
	k_H =	0.30	d^{-1}
	t =	1.32	days
	C_o =	350	mg/L
	C_2 =	251.5	mg/ L
Pond #3	V_4 =	3,121,613	gallons
	Q =	1,670,000	gpd
	k_H =	0.30	d^{-1}
	t =	1.87	days
	C_2 =	251.5	mg/ L
	C_4 =	162.0	mg/ L
New Pond 3	V_3 =	3,121,613	gallons
	Q =	1,670,000	gpd
	k_H =	0.30	d^{-1}
	t =	1.87	days
	C_4 =	162.0	mg/ L
	C_6 =	**104.3**	**mg/ L**
New Pond 4	V_3 =	3,121,613	gallons
	Q =	1,670,000	gpd
	k_H =	0.30	d^{-1}
	t =	1.87	days
	C_6 =	104.3	mg/ L
	C_8 =	**67.2**	**mg/ L**

total retention time = **6.93 days**

% reduction = **81%**

For two parallel flow trains,

Four additional ponds are needed treat the wastewater to acceptable levels during high temp, high flow conditions

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Two parallel flow trains - MMF Summer Season (High temp & MMF flow cond.)

Current System Under 2030 Flow Conditions

Pond #1	V_1 =	2,211,984	gallons
	Q =	1,118,900	gpd
	k_H =	0.30	d^{-1}
	t =	1.98	days
	C_o =	350	mg/L
	C_1 =	220.9	mg/ L
Pond #4	V_3 =	1,872,968	gallons
	Q =	1,118,900	gpd
	k_H =	0.30	d^{-1}
	t =	1.67	days
	C_1 =	220.9	mg/ L
	C_3 =	**147.8**	**mg/ L**
Pond #2	V_2 =	2,211,984	gallons
	Q =	1,118,900	gpd
	k_H =	0.30	d^{-1}
	t =	1.98	days
	C_o =	350	mg/L
	C_2 =	220.9	mg/ L
Pond #3	V_4 =	1,872,968	gallons
	Q =	1,118,900	gpd
	k_H =	0.30	d^{-1}
	t =	1.67	days
	C_2 =	220.9	mg/ L
	C_4 =	**147.8**	**mg/ L**

total retention time = **3.65 days**

% reduction = **58%**

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Three Ponds in Series, Two parallel flow trains - MMF Summer Season (High temp & MMF flow cond.)

Remove Baffles from Ponds 3 & 4

Add two ponds, V = 3,121,613 gallons each

Pond #1	V_1 =	2,211,984	gallons
	Q =	1,118,900	gpd
	k_H =	0.30	d^{-1}
	t =	1.98	days
	C_o =	350	mg/L
	C_1 =	220.9	mg/ L

Pond #4	V_3 =	3,121,613	gallons
	Q =	1,118,900	gpd
	k_H =	0.30	d^{-1}
	t =	2.79	days
	C_1 =	220.9	mg/ L
	C_3 =	121.0	mg/ L

New Pond	V_3 =	3,121,613	gallons
	Q =	1,118,900	gpd
	k_H =	0.30	d^{-1}
	t =	2.79	days
	C_3 =	121.0	mg/ L
	C_5 =	**66.3**	**mg/ L**

Pond #2	V_2 =	2,211,984	gallons
	Q =	1,118,900	gpd
	k_H =	0.30	d^{-1}
	t =	1.98	days
	C_o =	350	mg/L
	C_2 =	220.9	mg/ L

Pond #3	V_4 =	3,121,613	gallons
	Q =	1,118,900	gpd
	k_H =	0.30	d^{-1}
	t =	2.79	days
	C_2 =	220.9	mg/ L
	C_4 =	121.0	mg/ L

New Pond	V_3 =	3,121,613	gallons
	Q =	1,118,900	gpd
	k_H =	0.30	d^{-1}
	t =	2.79	days
	C_4 =	121.0	mg/ L
	C_6 =	**66.3**	**mg/ L**

total retention time = **7.56 days**

% reduction = **81%**

For two parallel flow trains,
Two additional ponds would treat the wastewater to acceptable levels during high temp, max month flow conditions

*M&E Reference: Wastewater Engineering Treatment and Reuse, 4th Edition

Boyle Engineering Corporation

BY: EKM **DATE:** 12/1/2006 **SUBJECT** Southland WWTF Master Plan **JOB NO:** 19996.17
CHKD. BY: ____ **DATE:** ____ Future Projected Solids Production (2030)

Determine: Volume of solids added to ponds over 5 years at projected 2030 flowrate.

Assumptions:

AAF = 1.67 mgd Average TSSin = 265 mg/L Average TSSout = 40 mg/L

1) Total volume of wastewater treated in past 5 years

V = Q x t
V = 1.02 mgd x 5 yrs x 365 days/yr
V = 3048 Mgal

2) Mass of TSS removed

Mass = $(TSS_{in} - TSS_{out})$ x V x (8.34 lb/Mgal x mg/L)
Mass = (265 - 40) x (13048) x (8.34)
= 5,719,103 lbs
= 1,143,821 lbs/yr

3) Mass of volatile and fixed solids

$Mass_{VSS}$ = 0.70 x TSS
= 0.70 x (2,054,768)
= 4,003,372 lbs
= 800,674 lbs/yr

$Mass_{Fixed} = Mass_{TSS} - Mass_{VSS}$
= 2,054,768 - 1,438,337
= 1,715,731 lbs
= 343,146 lbs/yr

4) Amount of accumulation at the end of 5 years

Assume 60% VSS reduction occurs within 1 year

$(VSS)_t$ = [0.7 + 0.4(t-1)] x VSS
= [0.7 + 0.4(5-1)] x 489,166
= 1,841,551 lbs

5) Total mass of solids

$Mass_{Total} = Mass_{Fixed} + Mass_{Accumulated}$
= 1,048,213 + 1,125,082
= 3,557,282 lbs

6) Volume of solids (assume 15% solids and density = 1.06*8.34 lb/gal)

$V_{Total} = Mass_{Total}$ / (0.15*density)
= 2,682,595 gal

Boyle Engineering Corporation

BY:	EKM	**DATE:**	12/1/2006	**SUBJECT**	Southland WWTF Master Plan	**JOB NO:**	19996.17
CHKD. BY:		**DATE:**			Future Projected Solids Production (2030)		

Potential percentage of solid volume in ponds over 5 years at projected flowrate

Total pond volume (taken from NCSD Southland O&M Manual, July 2000)
Liquid volume = 2 @ 295,700 cf & 2 @ 417,300 cf
Sludge volume = 2 @ 0.5 Mgal & 2 @ 0.7 Mgal

V_{Total} = [2 x 295,700 + 2 x 417,300] x 7.481 gal/cf + 2 x 500,000 + 2 x 700, 000
V_{Total} = 13,067,906 gal

$$\% \text{ of solids in pond} = \frac{2,682,595}{13,067,906}$$

= 0.21
= 21% of existing pond volume for 5 years at projected future flowrate

APPENDIX C

COST OPINIONS

Revised: 01/24/07

Nipomo Community Services District

UPGRADE TO FRONTAGE ROAD INTERCEPTOR (15" OPEN TRENCH CONSTRUCTION)

SUMMARY

ENGINEER'S OPINION OF PROBABLE CONSTRUCTION COST

Item	Description	Quantity	Unit	Total Unit Price	Amount
1	Mobilization	1	LS	*$50,000.00*	$50,000
2	Pothole Existing Utilities	5	EA	*$750.00*	$3,800
3	Temporary Sewage Bypass	1	LS	*$13,000.00*	$13,000
4	Traffic Control & Regulation	3123	LF	*$10.00*	$31,200
5	Sheeting & Shoring	4208	LF	*$17.50*	$73,600
6	Abandon Existing Pipe in Place	1	LS	*$35,000.00*	$35,000
7	Connect Laterals/Exist Manholes to New Main (8" at Division and Southland)	2	EA	*$4,000.00*	$8,000
8	Connect Trunk/Manhole to New Main (12" at Story)	1	EA	*$8,000.00*	$8,000
9	15-inch PVC Sewer Main (Excavate, Install, backfill, pavement repair)	4208	LF	*$175.00*	$736,500
10	Precast 48-inch I.D. Manholes (15-20 ft)	1	EA	*$9,000.00*	$9,000
11	Precast 48-inch I.D. Manholes (10-14 ft)	7	EA	*$6,000.00*	$42,000
12	Precast 48-inch I.D. Manholes (5-9 ft)	2	EA	*$4,000.00*	$8,000
13	Connect to Existing Metering Manhole at WWTF	1	LS	*$8,000.00*	$8,000
14	Pipeline Cleaning and CCTV Inspection	4208	LF	*$3.00*	$12,600

Sub Total		$1,039,000
Engineering/Administration	30%	$311,700
Contingency	30%	$405,210
Total		$1,756,000

ENR CCI = 7880 (February, 2007)

LS = Lump Sum
EA = Each
LF = Linear Foot

Assumptions for Opinion of Cost (By CR):

1. Sewer upgrade to occur within Frontage Rd. paved ROW, in a new trench parallel to existing 12" interceptor sewer.
2. Review of NCSD water atlas indicates presence of water pipes along Frontage Rd.;
As-builts for 12" interceptor indicate presence of 16" Gas. It is assumed the interceptor upgrade can be aligned within the paved ROW w/o utility conflicts or relocates.
3. It is assumed sewage bypass will only be required for last phase of construction, when lateral/trunk connections/manholes are switched over to new sewer.
4. Traffic control only needed from Division to Southland (not on unpaved part to WWTF)

Nipomo Community Services District

UPGRADE TO FRONTAGE ROAD INTERCEPTOR (21" OPEN TRENCH CONSTRUCTION)
SUMMARY
ENGINEER'S OPINION OF PROBABLE CONSTRUCTION COST

Item	Description	Quantity	Unit	Total Unit Price	Amount
1	Mobilization	1	LS	*$50,000.00*	$50,000
2	Pothole Existing Utilities	5	EA	*$750.00*	$3,800
3	Temporary Sewage Bypass	1	LS	*$13,000.00*	$13,000
4	Traffic Control & Regulation	3123	LF	*$10.00*	$31,200
5	Sheeting & Shoring	4208	LF	*$17.50*	$73,600
6	Abandon Existing Pipe in Place	1	LS	*$35,000.00*	$35,000
7	Connect Laterals/Exist Manholes to New Main (8" at Division and Southland)	2	EA	*$4,000.00*	$8,000
8	Connect Trunk/Manhole to New Main (12" at Story)	1	EA	*$8,000.00*	$8,000
9	21-inch PVC Sewer Main (Excavate, Install, backfill, pavement repair)	4208	LF	*$235.00*	$988,900
10	Precast 48-inch I.D. Manholes (15-20 ft)	1	EA	*$9,000.00*	$9,000
11	Precast 48-inch I.D. Manholes (10-14 ft)	7	EA	*$6,000.00*	$42,000
12	Precast 48-inch I.D. Manholes (5-9 ft)	2	EA	*$4,000.00*	$8,000
13	Connect to Existing Metering Manhole at WWTF	1	LS	*$8,000.00*	$8,000
14	Pipeline Cleaning and CCTV Inspection	4208	LF	*$3.00*	$12,600

Sub Total		$1,291,000
Engineering/Administration	30%	$387,300.0
Contingency	30%	$503,490
Total		$2,182,000

ENR CCI = 7880 (February, 2007)

LS = Lump Sum
EA = Each
LF = Linear Foot

Assumptions for Opinion of Cost (By CR):

1. Sewer upgrade to occur within Frontage Rd. paved ROW, in a new trench parallel to existing 12" interceptor sewer.
2. Review of NCSD water atlas indicates presence of water pipes along Frontage Rd.;
As-builts for 12" interceptor indicate presence of 16" Gas. It is assumed the interceptor upgrade can be aligned within the paved ROW w/o utility conflicts or relocates.
3. It is assumed sewage bypass will only be required for last phase of construction, when lateral/trunk connections/manholes are switched over to new sewer.
4. Traffic control only needed from Division to Southland (not on unpaved part to WWTF)

Nipomo Community Services District
SOUTHLAND WASTEWATER TREATMENT FACILITY
MASTER PLAN
Headworks Improvement Options
OPINION OF PROBABLE CAPITAL COST

Item	Description	Unit	Unit Price	Quantity	Installation Adjustment	Amount
SCREENS						
I. Parkson HLS400 Hycor® HeliSieve®						
1	HeliSieve® HLS500	EA	$65,000.00	2	1.5	$195,000
2	2 Concrete channels, w/common wall	YD^3	$900.00	12		$10,800
3	Miscellaneous piping	LS				$20,000
4	Bypass pipe	LS				$10,000
5	Sitework	LS				$15,000
6	Electrical + Instrumentation	LS				$20,000
7	Bagger (optional)	EA	$2,000.00	2	1.5	$6,000
	Subtotal					$276,800
8	Engineering/Admin (30 % of subtotal)					$83,040
9	Contingency (30% of total)					$107,952
	TOTAL					$468,000
II. Parkson Aqua Guard® AG-MN-A						
1	Aqua Guard® AG-MN-A	EA	$90,000.00	2	1.5	$270,000
2	2 concrete channels, w/common wall	YD^3	$900.00	9		$8,100
3	Misc. piping	LS				$20,000
4	Bypass pipe	LS				$10,000
5	Sitework	LS				$15,000
6	Electrical + Instrumentation	LS				$20,000
7	Parkson Hycor® Screw Wash & Press Unit SWP20-XX (optional)	EA	$40,000.00	2	1.5	$120,000
	Subtotal					$463,100
8	Engineering/Admin (30 % of subtotal)					$138,930
9	Contingency (30% of total)					$180,609
	TOTAL					$783,000

ENR CCI = 7880 (February. 2007)

Nipomo Community Services District
SOUTHLAND WASTEWATER TREATMENT FACILITY
MASTER PLAN
Headworks Improvement Options
OPINION OF PROBABLE CAPITAL COST

Item	Description	Unit	Unit Price	Quantity	Installation Adjustment	Amount
GRIT REMOVAL						
I. Eimco Jones & Attwood JetAir 100 & Screw Classifier 100						
1	JetAir + Classifier + assoc. equipment	EA	$89,000.00	2	1.5	$267,000
2	Concrete	YD^3	$900.00	20		$18,000
3	Misc. piping	LS				$20,000
4	Electrical + Instrumentation	LS				$15,000
5	Sitework	LS				$5,000
6	Bagger (optional)	EA	$2,000.00	2	1.5	$6,000
	Subtotal					$331,000
7	Engineering/Admin (30 % of subtotal)					$99,300
8	Contingency (30% of total)					$129,090
	TOTAL					$560,000
II. Aerated Grit Chamber (two at 6' x 6' x 24')						
1	2 concrete chambers	LS				$120,000
3	Air Piping	LS				$30,000
4	Diffusers	LS				$35,000
5	Misc. piping	LS				$25,000
6	Electrical + Instrumentation	LS				$15,000
7	Sitework	LS				$5,000
8	Grit classifier	LS				$88,500
	Subtotal					$318,500
8	Engineering/Admin (30 % of subtotal)					$95,550
9	Contingency (30% of total)					$124,215
	TOTAL					$539,000

ENR CCI = 7880 (February. 2007)

LS = Lump sum
EA = Each
LF = Linear Foot
YD^3 = Cubic Yard

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Nipomo Community Services District

SOUTHLAND WASTEWATER TREATMENT FACILITY

MASTER PLAN

Future Treatment Alternatives

OPINION OF PROBABLE CAPITAL COST

Item	Description	Unit	Unit Price	Quantity	Installation Adjustment	Amount
I. Expansion of Aerated Ponds (4)						
1	Excavation for 4 ponds	YD^3	$25.00	118,550	1.0	$2,963,800
2	Fill for 4 ponds	YD^3	$25.00	40,400	1.0	$1,010,000
3	Grading for 4 ponds	FT^2	$0.20	207,500	1.0	$41,500
3	4 HDPE Liners (40 mil)	FT^2	$0.33	341,900	1.7	$191,800
3	Mechanical Aerators (15 HP)	EA	$24,000.00	14	1.7	$571,200
	Subtotal					$4,778,300
4	Piping (10% subtotal)					$477,830
5	Electrical (10% subtotal)					$477,830
6	Engineering/Admin (20 % of subtotal)					$955,660
7	Contingency (30% of total)					$2,006,886
	Total					$8,697,000
II. EIMCO Carrousel ® 3000 (Oxidation Ditch)						
1	Mobilization (3% of subtotal)					$101,100
2	Oxidation Ditch System	LS	$1,550,000.00	1	1.0	$1,550,000
3	(2) Secondary Clarifiers	LS	$910,000.00	2	1.0	$1,820,000
	Subtotal					$3,370,000
4	Sitework (20% of Subtotal)					$674,000
5	Piping (15% subtotal)					$505,500
6	Electrical (15% subtotal)					$505,500
7	Engineering/Admin (20 % of subtotal)					$674,000
8	Contingency (30% of total)					$1,718,700
	Total					$7,549,000
III. Parkson Biolac® Wave Oxidation System						
1	Biolac® System in 2 secondary ponds	EA	$520,000.00	1	1.7	$884,000
2	(2) HDPE Liner (40 mil)	FT^2	$0.33	170,968	1.7	$95,900
3	Concrete (integral clarifier)	YD^3	$900.00	900	1.0	$810,000
4	Earthwork (fill part of retrofitted ponds)	YD^3	$20.00	12250	1.0	$245,000
5	Instrumentation	LS				$100,000
5	Modification of air piping	LF	$50.00	970	1.0	$48,500
	Subtotal					$2,183,400
6	Piping (15% of subtotal)					$327,510
7	Electrical (15% of subtotal)					$327,510
8	Engineering/Admin (20 % of subtotal)					$436,680
9	Contingency (30% of total)					$982,530
	Total					$4,258,000

ENR CCI = 7880 (February. 2007)

Nipomo Community Services District
SOUTHLAND WASTEWATER TREATMENT FACILITY
MASTER PLAN
Future Treatment Alternatives
OPINION OF PROBABLE CAPITAL COST

Item	Description	Unit	Unit Price	Quantity	Installation Adjustment	Amount
IV. Completely Mixed Activated Sludge						
1	Mobilization (3% of subtotal)					$129,000
2	(2) Aeration Basins	LS				$860,000
3	(2) Primary Clarifiers	LS				$1,720,000
4	(2) Secondary Clarifiers	LS				$1,720,000
	Subtotal					$4,300,000
5	Sitework (5% of Subtotal)					$215,000
6	Piping (15% of subtotal)					$645,000
7	Electrical (15% of subtotal)					$645,000
8	Engineering/Admin (20 % of subtotal)					$860,000
9	Contingency (30% of total)					$1,999,500
	Total					$8,794,000

ENR CCI = 7880 (February. 2007)

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EA = Each
LF = Linear Foot
YD^3 = Cubic Yard

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Nipomo Community Services District

SOUTHLAND WASTEWATER TREATMENT FACILITY

MASTER PLAN

AERATED POND SYSTEM vs. BIOLAC SYSTEM

OPINION OF PROBABLE OPERATING AND MAINTENANCE COST

Life cycle costs to 2030

I. AERATED POND SYSTEM

Year	Capital Cost	Power Cost	Parts Cost	Total Cost	Cumulative Cost
2007	$8,697,000	$178,500	$0	$8,875,500	$8,875,500
2008	$0	$178,500	$0	$178,500	$9,054,000
2009	$0	$178,500	$0	$178,500	$9,232,500
2010	$0	$178,500	$0	$178,500	$9,411,000
2011	$0	$178,500	$0	$178,500	$9,589,500
2012	$0	$178,500	$0	$178,500	$9,768,000
2013	$0	$178,500	$0	$178,500	$9,946,500
2014	$0	$178,500	$0	$178,500	$10,125,000
2015	$0	$178,500	$0	$178,500	$10,303,500
2016	$0	$178,500	$0	$178,500	$10,482,000
2017	$0	$178,500	$336,000	$514,500	$10,996,500
2018	$0	$178,500	$0	$178,500	$11,175,000
2019	$0	$178,500	$0	$178,500	$11,353,500
2020	$0	$178,500	$0	$178,500	$11,532,000
2021	$0	$178,500	$0	$178,500	$11,710,500
2022	$0	$178,500	$0	$178,500	$11,889,000
2023	$0	$178,500	$0	$178,500	$12,067,500
2024	$0	$178,500	$0	$178,500	$12,246,000
2025	$0	$178,500	$0	$178,500	$12,424,500
2026	$0	$178,500	$0	$178,500	$12,603,000
2027	$0	$178,500	$336,000	$514,500	$13,117,500
2028	$0	$178,500	$0	$178,500	$13,296,000
2029	$0	$178,500	$0	$178,500	$13,474,500
2030	$0	$178,500	$0	$178,500	$13,653,000

Notes:

1. Project is built in 2007 for 2030 design flows.
2. Parts replacement consists of 14 aerators, replaced every 10 years.
3. Power is based on required power for 2018, 210 hp.

Nipomo Community Services District
SOUTHLAND WASTEWATER TREATMENT FACILITY
MASTER PLAN
AERATED POND SYSTEM vs. BIOLAC SYSTEM
OPINION OF PROBABLE OPERATING AND MAINTENANCE COST
Life cycle costs to 2030

II. BIOLAC SYSTEM

Year	Capital Cost	Power Cost	Parts Cost	Total Cost	Cumulative Cost
2007	$4,258,000	$76,500	$0	$4,334,500	$4,334,500
2008	$0	$76,500	$0	$76,500	$4,411,000
2009	$0	$76,500	$0	$76,500	$4,487,500
2010	$0	$76,500	$0	$76,500	$4,564,000
2011	$0	$76,500	$56,600	$133,100	$4,697,100
2012	$0	$76,500	$0	$76,500	$4,773,600
2013	$0	$76,500	$0	$76,500	$4,850,100
2014	$0	$76,500	$205,300	$281,800	$5,131,900
2015	$0	$76,500	$0	$76,500	$5,208,400
2016	$0	$76,500	$56,600	$133,100	$5,341,500
2017	$0	$76,500	$0	$76,500	$5,418,000
2018	$0	$76,500	$0	$76,500	$5,494,500
2019	$0	$76,500	$0	$76,500	$5,571,000
2020	$0	$76,500	$0	$76,500	$5,647,500
2021	$0	$76,500	$56,600	$133,100	$5,780,600
2022	$0	$76,500	$205,300	$281,800	$6,062,400
2023	$0	$76,500	$0	$76,500	$6,138,900
2024	$0	$76,500	$0	$76,500	$6,215,400
2025	$0	$76,500	$0	$76,500	$6,291,900
2026	$0	$76,500	$56,600	$133,100	$6,425,000
2027	$0	$76,500	$0	$76,500	$6,501,500
2028	$0	$76,500	$0	$76,500	$6,578,000
2029	$0	$76,500	$0	$76,500	$6,654,500
2030	$0	$76,500	$205,300	$281,800	$6,936,300

Notes:

1. Project is built in 2007 for 2030 design flows.

2. Parts replacement consists of diffusers, replaced every 5 years, and air hoses, replaced every 8 years.

3. Power is based on required power for 2018, 90 hp.

Note: These opinions of probable construction costs prepared by Boyle represent its judgment as a design professional and are supplied for the general guidance of NCSD. Since Boyle has no control over the cost of labor and materials, over delays in project bidding or award, or over competitive bidding or market conditions, Boyle does not guarantee the accuracy of such opinions as compared to design-level cost opinions, contractor bids, or actual cost to NCSD.

Nipomo Community Services District
SOUTHLAND WASTEWATER TREATMENT FACILITY
MASTER PLAN
Tertiary Treatment Alternatives
OPINION OF PROBABLE CAPITAL COST

Item	Description	Unit	Unit Price	Quantity	Installation Adjustment	Amount
FILTRATION						
I. Parkson Dynasand						
1	Coagulation & Mixing System	LS				$100,000
2	Pumping System	LS				$200,000
3	Filter Module	EA	$29,200.00	12	1.7	$595,700
4	Air compressors	EA	$12,500.00	2	1.7	$42,500
5	Concrete	YD^3	$900.00	270	1.0	$243,000
6	Ladders, handrails, grates	LS				$80,000
7	Instrumentation & Controls	LS				$50,000
	Subtotal					$1,311,200
8	Sitework (10% of subtotal)					$131,120
9	Piping (10% subtotal)					$131,120
10	Electrical (10% subtotal)					$131,120
11	Engineering/Admin (20 % of subtotal)					$262,240
12	Contingency (30% of total)					$590,040
	Total					$2,557,000
II. Aqua-Aerobic Aquadisk						
1	Coagulation & Mixing System	LS				$100,000
2	Pumping System	LS				$200,000
3	Filter Unit (10 disk) with controls	EA	$317,400.00	2	1.7	$634,800
4	Concrete foundation	YD^3	$900.00	24	1.0	$21,600
5	Ladders, handrails, grates	LS				$50,000
	Subtotal					$1,006,400
6	Sitework (5% of Subtotal)					$50,320
7	Piping (10% subtotal)					$100,640
8	Electrical (10% subtotal)					$100,640
9	Engineering/Admin (20 % of subtotal)					$201,280
10	Contingency (30% of total)					$437,784
	Total					$1,898,000
DISINFECTION						
I. Chlorine Contact Basin						
1	(2) Concrete basins	YD^3	$900.00	352	1.0	$316,800
2	Chlorine feed system & storage	LS				$350,000
3	Instrumentation & controls	LS				$100,000
	Subtotal					$766,800
5	Sitework (10% of subtotal)					$76,680
6	Piping (15% of subtotal)					$115,020
7	Electrical (10% of subtotal)					$76,680
8	Engineering/Admin (20 % of subtotal)					$153,360
9	Contingency (30% of total)					$356,562
	Total					$1,546,000

ENR CCI = 7880 (February, 2007)

Nipomo Community Services District
SOUTHLAND WASTEWATER TREATMENT FACILITY
MASTER PLAN
Tertiary Treatment Alternatives
OPINION OF PROBABLE CAPITAL COST

Item	Description	Unit	Unit Price	Quantity	Installation Adjustment	Amount
II. Trojan UV3000 Plus™						
1	UV banks and equipment	LS	$678,000.00		1.7	$1,152,600
2	Concrete	YD^3	$900.00	37	1.0	$33,300
3	Instrumentation & controls	LS				$100,000
4	Ladders, handrails, and grates	LS				$80,000
	Subtotal					$1,365,900
5	Sitework (10% of Subtotal)					$136,590
6	Piping (15% of subtotal)					$204,885
7	Electrical (15% of subtotal)					$204,885
8	Engineering/Admin (20 % of subtotal)					$273,180
9	Contingency (30% of total)					$655,632
	Total					$3,994,000

ENR CCI = 7880 (February, 2007)

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EA = Each
LF = Linear Foot
YD^3 = Cubic Yard

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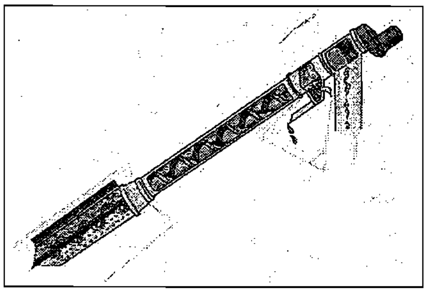
APPENDIX D

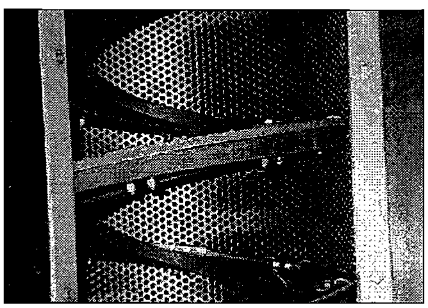
PRODUCT INFORMATION

PARKSON CORPORATION

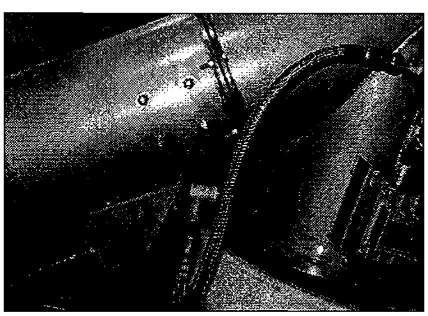
Hycor® Helisieve® In-Channel Fine Screen Model HLS



Combines screening, conveying and dewatering into one reliable, automatic, cost-efficient system.

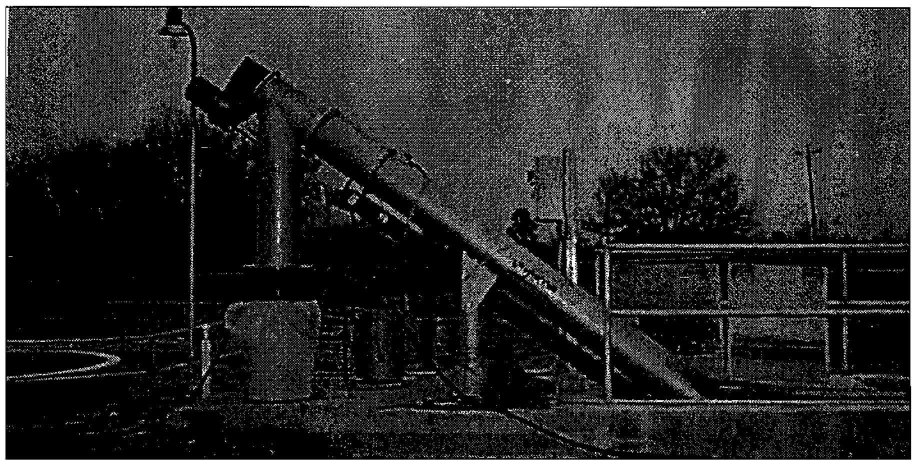


Durable spiral brush keeps the screen clean.



Close-up view of the new drain box with optional explosion-proof wiring.

All-in-one screening, conveying and dewatering system



The Helisieve system uses shaftless spiral technology to perform screening, solids conveying and dewatering in one cost efficient operation. The heart of the system is a heavy-duty carbon steel spiral that conveys screenings to the dewatering zone and dewaters them to acceptable landfill requirements. The spiral is fabricated in a continuous flight to assure a strong, stable structure. It is surrounded by a stainless steel tube that encloses screenings, minimizes odors and provides clean, hygienic operation.

The Helisieve's shaftless core handles a greater volume of solids than shafted screw designs. Fibrous and bulky solids have a clear, barrier-free path to the dewatering zone. The shaftless design also eliminates the need for maintenance-intensive bottom support bearings and intermediate hanger bearings.

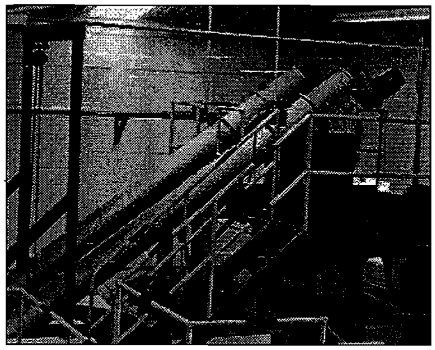
The Helisieve system performs three operations in one:

Screening. Influent moves into the fine screening area where the perforated screen removes solids. A spiral-mounted brush keeps the screen surface clean.

Conveying. The spiral moves the screenings upward through the transport area. There is no shaft to restrict flow or become entangled with long, stringy solids.

Dewatering. Solids are dewatered by compression against a plug of material formed in the flightless zone. Liquid is discharged through a perforated screen. A removable drain box simplifies access to the screen and solids plug. Solids at 40% dry weight are common.

Put Hycor® shaftless spiral technology to work for you!



- Cost-effective — integrates three processes: screening, conveying and dewatering, in one compact unit.

- Efficient — the shaftless spiral provides greater conveying capacity and eliminates entanglement of solids around a shaft.

- Lowers disposal costs — dewatering reduces weight and volume. Forty percent dry weight solids are common.

- Hygienic — screens are enclosed by the stainless steel tube and can be discharged directly into sealed containers to minimize odor and handling. Optional bagger assemblies simplify disposal.

- Designed to last — rugged steel alloy spiral fabricated in a continuous flight to tight manufacturing tolerances.

- Compact and easy to install — shipped assembled, with flexible seals, for quick channel positioning, or in its own tank housing.

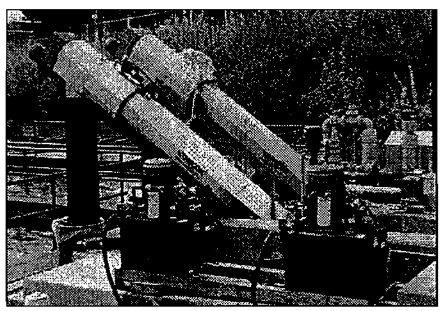
- Economical — one low horsepower gearmotor drives the entire system.

- Up-front serviceability — pivots out for easy access for above-channel maintenance.

- Low maintenance — no troublesome submerged end bearings or intermediate hanger bearings.

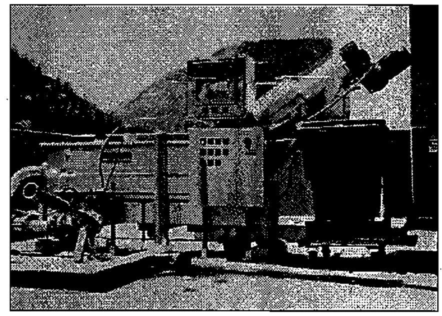
Screen openings

0.125" and 0.250" (6 mm) diameter and .040" x .4" perforated slots. Other opening sizes are possible.



Shown with optional hydraulic drive design and heat trace jacket.

Helisieve Plus® in-tank system for pumped flows



Screens, conveys and dewaters like the Helisieve unit, but is self-contained in a stainless steel tank. Suitable for industrial and municipal processes.

PARKSON CORPORATION

www.parkson.com

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P.O. Box 408399
Fort Lauderdale, FL 33340-8399
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29850 N. Skokie Hwy. (U.S. 41)
Lake Bluff, IL 60044-1192
P(847) 473-3700 • F(847) 473-0477

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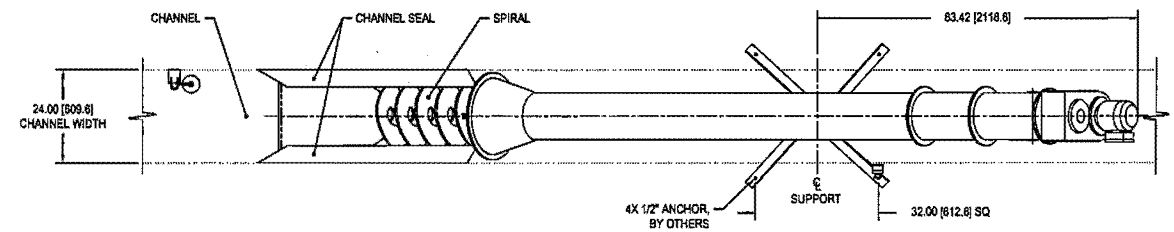
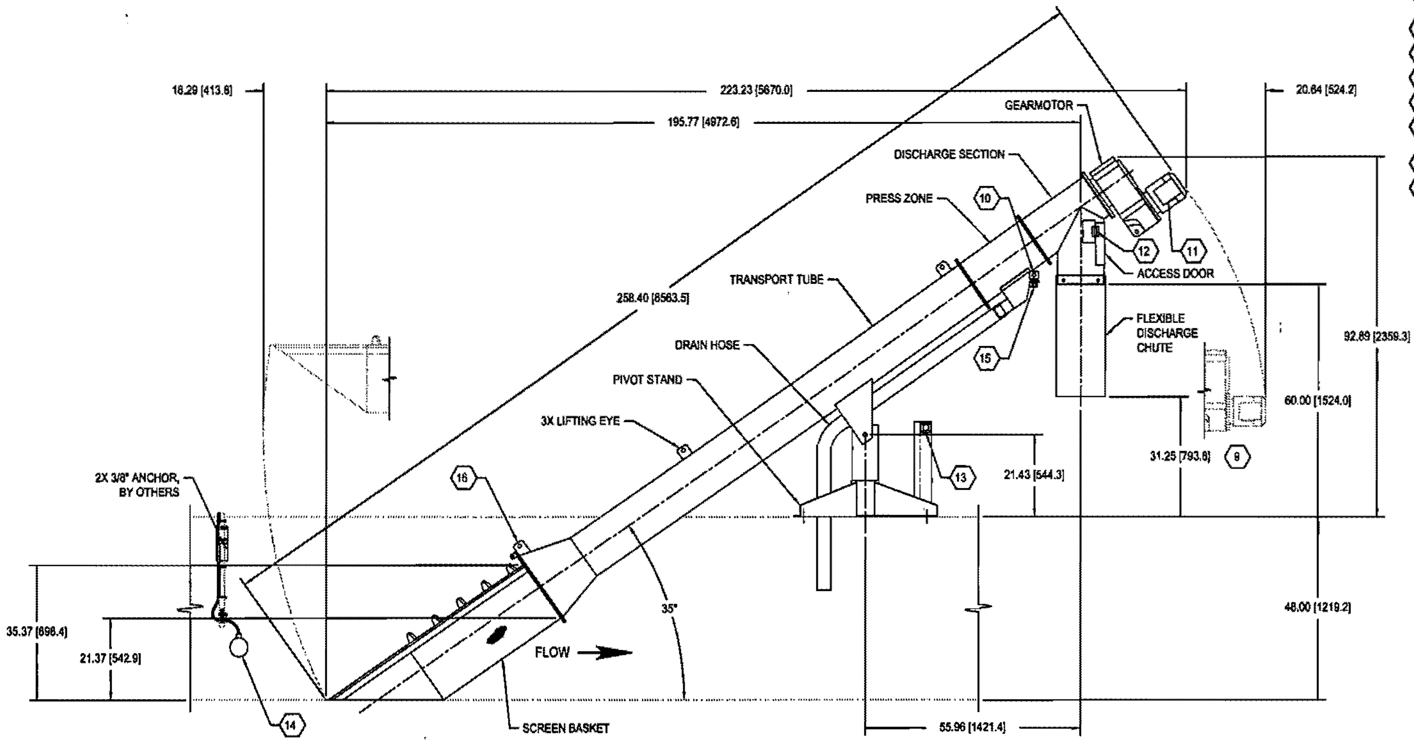


TOP VIEW



SIDE VIEW
(CHANNEL SEALS NOT SHOWN)

NOTE:

1. ALL 304L STAINLESS STEEL CONSTRUCTION EXCEPT FOR REDUCER, MOTOR, SPIRAL, ELECTRICAL FIXTURES, DISCHARGE CHUTE, AND CHANNEL SEALS.
2. GEARMOTOR: 1.5 HP [1.1 kW], 1800 RPM, 230/460 V, 3 PH, 60 HZ, TEFC, SEVERE DUTY.
3. SPIRAL SPEED: 7.4 RPM.
4. SCREEN OPENING: Ø.25 [Ø6.4].
5. RECOMMENDED CLEARANCE TO BE 36.00 [914.4] AROUND AND ABOVE UNIT.
6. WEIGHT: 1,755 LB [795 kg].
7. DIMENSIONS WRITTEN IN INCHES [mm] UNLESS OTHERWISE SPECIFIED.
8. PROVIDE SUFFICIENT FLEXIBILITY IN WATER AND ELECTRICAL CONNECTIONS TO ALLOW THE UNIT TO PIVOT OUT OF THE CHANNEL. ALL INTERCONNECTING WIRING, CONDUIT AND PIPING FROM UNIT MOUNTED DEVICES WILL BE SUPPLIED BY OTHERS.
9. GROUND CLEARANCE FOR DISCHARGE RECEPTACLE. DO NOT REMOVE FLEXIBLE DISCHARGE CHUTE/GUARD.
10. NEMA 4X SOLENOID VALVE: 1/2" NPT CONDUIT CONNECTION.
11. MOTOR: 2X 1/2" NPT CONDUIT CONNECTION.
12. NEMA 4X INTERLOCK SWITCH: 6 FOOT [1.8 M] LONG INTEGRAL CABLE.
13. NEMA 4X LOCAL E-STOP: 1/2" NPT CONDUIT CONNECTION.
14. FLOAT SWITCH: 20 FOOT [6.1 M] LONG INTEGRAL CABLE (MOUNTING BRACKET INCLUDED; 1" PIPE PROVIDED BY OTHERS).
15. 3/4" NPT WATER SPRAY CONNECTION.
16. UNIT IS BASKET END HEAVY. CUSTOMER MUST PROVIDE LIFTING DEVICE TO PIVOT UNIT OUT OF CHANNEL. LIFTING CAPABILITY MUST EQUAL A MINIMUM OF 60% OF UNIT WEIGHT, APPLIED AT LIFTING POINT SHOWN. CHANNEL MUST BE EMPTY AND SCREEN BASKET CLEAR OF SOLIDS.
17. STANDARD UNIT SHOWN. CONSULT PARKSON CORPORATION OR YOUR LOCAL HYCOR PRODUCTS REPRESENTATIVE FOR AVAILABLE OPTIONS.

PARKSON CORPORATION

The Owner, Project Engineer, and all others involved with the project design must implement and follow all safety standards required by local, state and federal laws when incorporating Parkson Corporation equipment into the overall project design. Parkson Corporation will not be responsible for location and/or placement of equipment in the plant design, nor is Parkson Corporation responsible for plant safety design and for the failure to follow appropriate safety precautions in the operation and maintenance of Parkson Corporation equipment.

REV	DESCRIPTION	DATE	BY

DRAWN BY	DATE
CHECKED BY	DATE
SCALE 1/32" = 1"	

REFERENCE INFORMATION
REV DATE: 03/31/04

INFORMATION ONLY

PROJECT NAME

TITLE
HLS500
HYCOR® HELISIEVE® UNIT

DRAWING NO

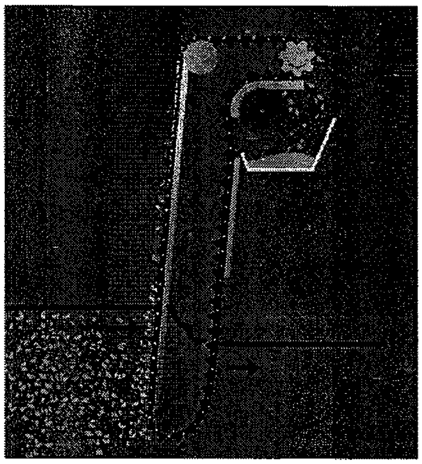
REV

PARKSON CORPORATION

Aqua Guard®

Self-Cleaning Moving Media Channel Screen

The Aqua Guard screen is a self-cleaning, in-channel screening device that utilizes a unique filter element system designed to automatically remove a wide range of floating and suspended solids from wastewater.

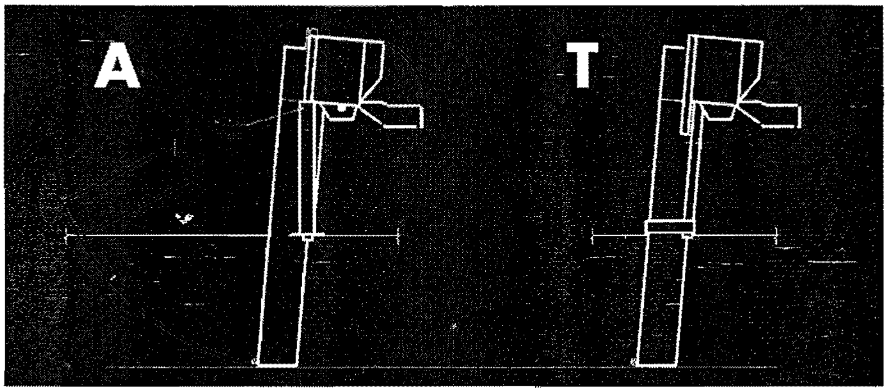


A specific configuration of filter elements is mounted on a series of parallel shafts to form an endless moving belt that collects, conveys and discharges solids greater than the element spacing. Spacing from 0.04" (1 mm) to 1.18" (30 mm) is available.

Principle of Operation Solids contained in a wastewater flow are captured on the filter elements and carried upward on the belt assembly to discharge at the rear of the unit. Two-stage screening is achieved which results in minimal headloss. Coarse filtration occurs on the forward screen face and fine filtration on the recessed face.

As the rake tip of one row of filter elements passes between the shank arm of the lower row, the elements automatically clean themselves. The unit is equipped with a rotating brush that provides additional removal of solids.

Features	Benefits
• Low power consumption (1.0 HP or less) • Self-cleaning • Intermittent operation • No submerged bearings • All moving parts can be accessed and serviced above water level • Screens pivots out of channel	Low Operation Costs & Ease of Maintenance
• Coarse and fine screening in one unit • Ability to build precoat	High capture rates
• Flows to 100 MGD in a single unit	High capacity
• Delivered fully assembled • No attachment to sides or bottom of channel	Ease of installation



The Aqua Guard® Screen styles A and T are available in Standard or Heavy Duty design.

Design Parameters	Model MN (Standard)	Model S (Heavy Duty)
Minimum Channel Width	12	24
Maximum Screen Width	66	108
Maximum Design Headloss	10	20
Fine Horizontal Spacing	1/24 (1mm)	1/24 (1mm)
	1/8 (3mm)	1/8 (3mm)
	1/4 (6mm)	1/4 (6mm)
	5/8 (15mm)	5/8 (15mm)
		1 1/4 (30mm)
Coarse Horizontal Spacing	1/8 (4mm)	1/8 (4mm)
	3/8 (8mm)	3/8 (8mm)
	5/8 (14mm)	5/8 (14mm)
	1 3/8 (34mm)	1 3/8 (34mm)
		2 5/8 (69mm)
Fine Spacing Contact Surface Area	0.81	0.901
	0.73	0.733
	0.63	0.694
	0.57	0.591
		0.547
*Trash Capacity		
	0.75	2.32
	0.50	1.27
	0.28	0.99
Filtration Dual	(Coarse & Fine)	(Coarse & Fine)

**Based on yds³/hr per one foot of effective width*

Design Parameters Standard screen widths are 1.0' to 9.0' depending on the model with flow rates up to 100 MGD with a single unit. Two frame styles are available depending on space and channel depth requirements. Type A is a pivoting design and Type T is a stationary design.

The Aqua Guard screen can be installed at angles of 60°, 75° and 85° depending on the frame and model selected. For maximum efficiency of operation, greater flow rate and higher solids removal, the recommended angle of inclination is 75°.

The screen conveys solids up and out of the channel at a speed of 7ft/min. The maximum amount of debris, in cubic yards per hour, that can be removed from the stream is a function of model and angle.

Movement of the screen can be continuous or intermittent. However, intermittent operation is recommended. This allows a mat of solids to build on the filter-rake elements which increases the solids capture rate.

Performance Parkson has over 5,000 installations in a wide variety of municipal and industrial applications.

Aqua Guard MN 75° 1.5' x 12' in operation

PARKSON CORPORATION

ISO 9001:2000 Certified
Quality Management System

www.parkson.com

AN AXEL JOHNSON INC. COMPANY

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Parkson do Brasil Ltda.
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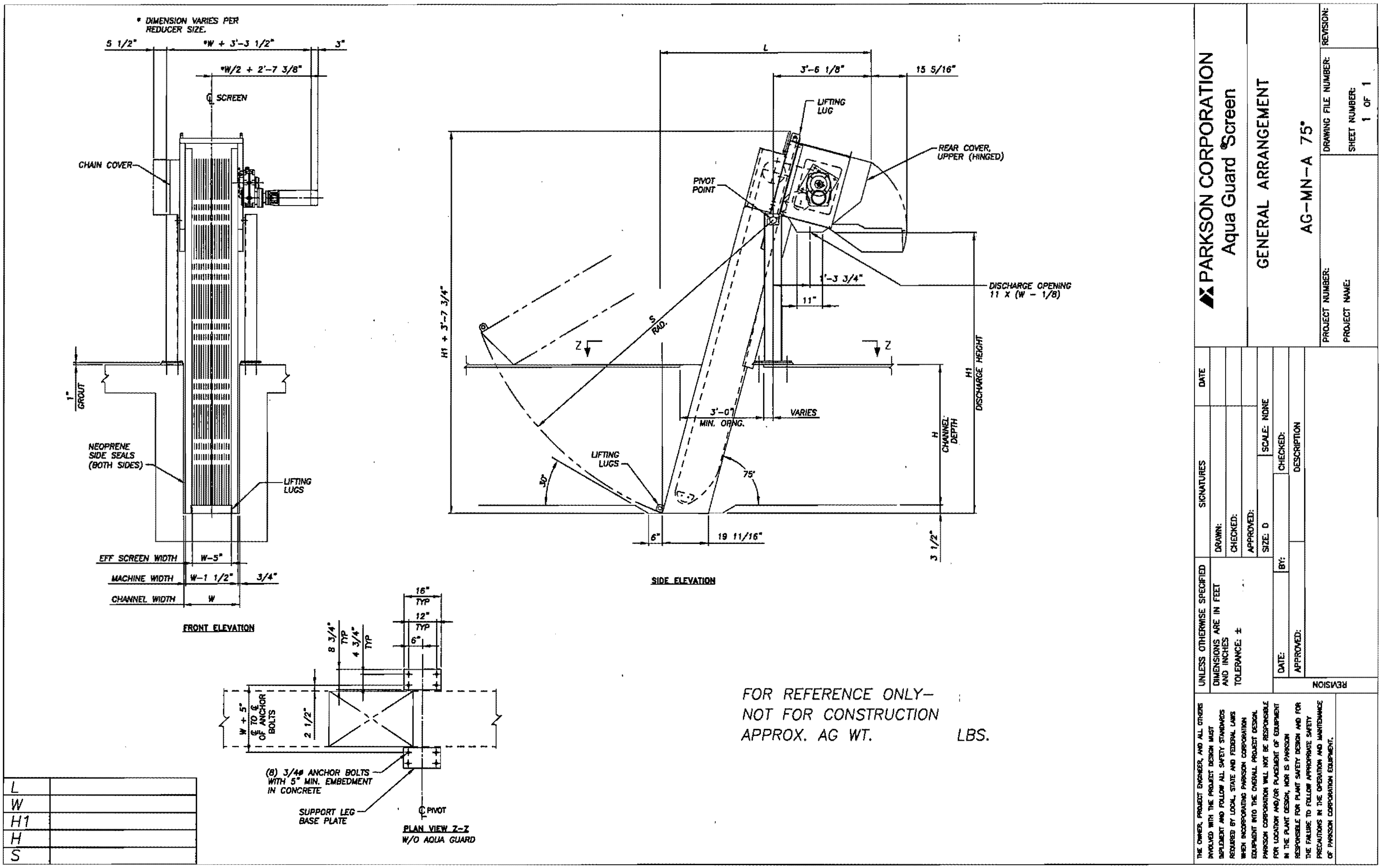
Barueri, Sao Paulo
CEP 06453-000
Brazil
P/F 55.11.4195.5084
P/F 55.11.4688.0336


* DIMENSION VARIES PER REDUCER SIZE.
5 1/2"
*W + 3'-3 1/2"
3"
*W/2 + 2'-7 3/8"
℄ SCREEN
CHAIN COVER
1" GROUT
NEOPRENE SIDE SEALS (BOTH SIDES)
LIFTING LUGS
EFF SCREEN WIDTH
W-5"
MACHINE WIDTH
W-1 1/2"
3/4"
CHANNEL WIDTH
W
FRONT ELEVATION
L
3'-6 1/8"
15 5/16"
LIFTING LUG
REAR COVER, UPPER (HINGED)
PIVOT POINT
1'-3 3/4"
11"
DISCHARGE OPENING 11 X (W - 1/8)
H1 + 3'-7 3/4"
S RAD.
Z
Z
H1 DISCHARGE HEIGHT
3'-0" MIN. OPNG.
VARIES
H CHANNEL DEPTH
LIFTING LUGS
30°
75°
6"
19 11/16"
3 1/2"
SIDE ELEVATION
16" TYP
12" TYP
6"
8 3/4" TYP
4 3/4" TYP
W + 5" ℄ TO ℄ OF ANCHOR BOLTS
2 1/2"
(8) 3/4ø ANCHOR BOLTS WITH 5" MIN. EMBEDMENT IN CONCRETE
SUPPORT LEG BASE PLATE
℄ PIVOT
PLAN VIEW Z-Z
W/O AQUA GUARD
L
W
H1
H
S
PARKSON CORPORATION
Aqua Guard ® Screen
GENERAL ARRANGEMENT
AG-MN-A 75°
REVISION:
DRAWING FILE NUMBER:
SHEET NUMBER: 1 OF 1
PROJECT NUMBER:
PROJECT NAME:
DATE
SIGNATURES
DRAWN:
CHECKED:
APPROVED:
SIZE: D
SCALE: NONE
UNLESS OTHERWISE SPECIFIED
DIMENSIONS ARE IN FEET AND INCHES
TOLERANCE: ±
DATE:
BY:
CHECKED:
APPROVED:
DESCRIPTION
REVISION
THE OWNER, PROJECT ENGINEER, AND ALL OTHERS INVOLVED WITH THE PROJECT DESIGN MUST IMPLEMENT AND FOLLOW ALL SAFETY STANDARDS REQUIRED BY LOCAL, STATE AND FEDERAL LAWS WHEN INCORPORATING PARKSON CORPORATION EQUIPMENT INTO THE OVERALL PROJECT DESIGN. PARKSON CORPORATION WILL NOT BE RESPONSIBLE FOR LOCATION AND/OR PLACEMENT OF EQUIPMENT IN THE PLANT DESIGN, NOR IS PARKSON RESPONSIBLE FOR PLANT SAFETY DESIGN AND FOR THE FAILURE TO FOLLOW APPROPRIATE SAFETY PRECAUTIONS IN THE OPERATION AND MAINTENANCE OF PARKSON CORPORATION EQUIPMENT.
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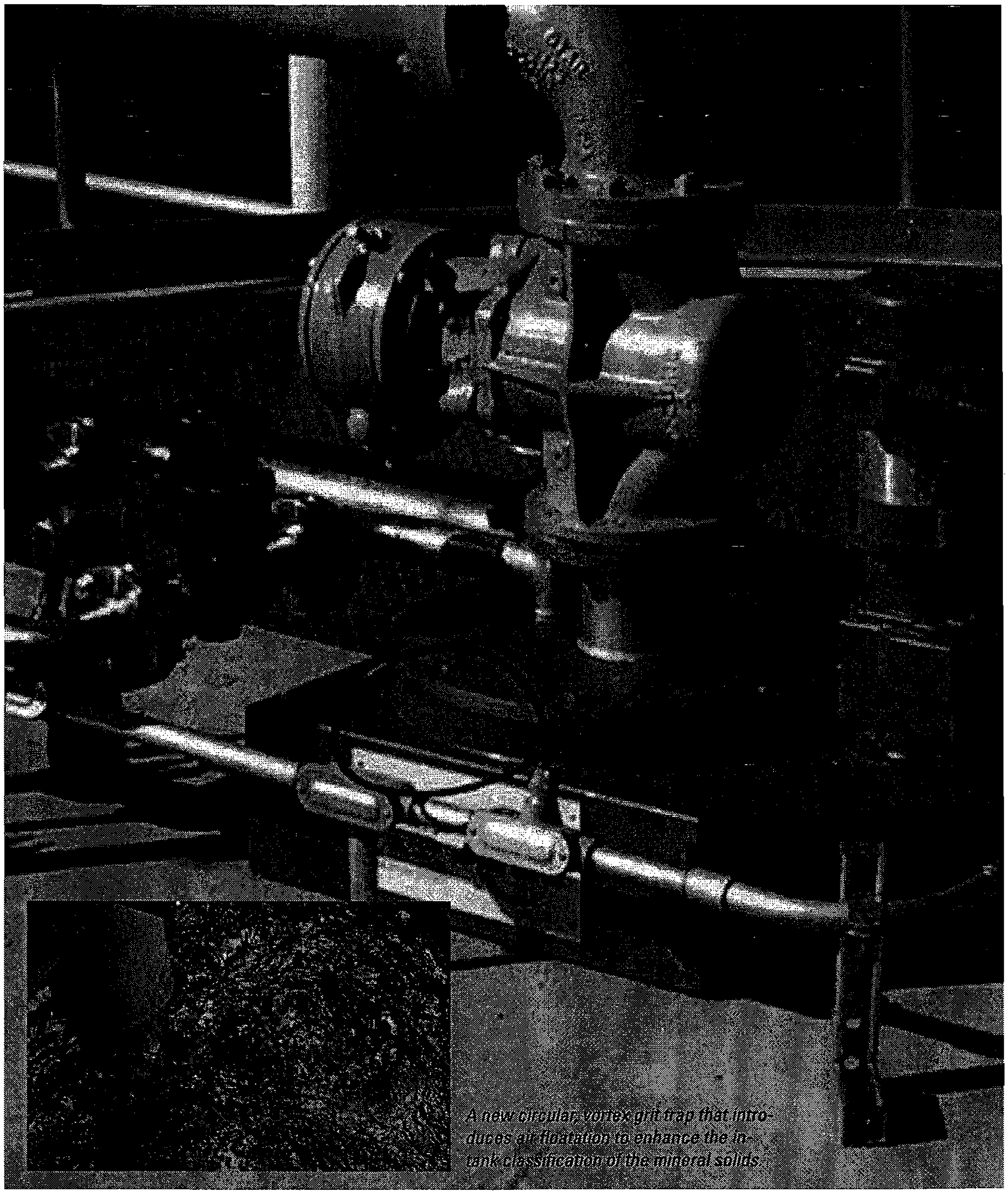
APPROX. AG WT. LBS.

EIMCO WATERTECHNOLOGIES

Jones+Attwood® JetAir

The New Advanced Grit Removal System



Patents applied for

A GL&V COMPANY

Introduction

The circular chamber, vortex flow and tangential entry grit traps are now an established method of grit removal from waste water. They form an integral part of the headworks to the waste water treatment plant.

Pista SA of Switzerland introduced the original circular grit trap in 1960. Jones + Attwood were given a world wide selling agreement by Pista for the life of the patent. Jones + Attwood have installed thousands of grit traps throughout the world and lead the field with grit removal technologies.

The new Jetair is the third generation of 'grit traps'. Each in its own right has expanded the boundaries of efficiency for performance and reliability.

Now, the functions of the mechanism have been analysed further and this new development allows the two most fundamental features to be enhanced separately and therefore achieve a maximum result for both.

All grit traps currently available include a means of achieving the rotary motion around the chamber, thus inducing the vortex that encourages solids to migrate to the centre of the chamber for collection. The impeller or propeller is so shaped and sized (and in some cases adjustable) to perform classification of the solids. Combining these two important functions inevitably results in compromises being made and one or both features will have their effectiveness reduced.

The Jetair provides an impeller that is designed to create the rotary motion only. The correct flow pattern is therefore achievable with this new fixed geometry impeller. Classification of the grit is achieved by the continuous aeration that surrounds the periphery of the impeller.

Low pressure air is delivered to the impeller which expels it in a controlled way from its periphery. The rotation of the impeller drags the air and increases its flow path. This results in the annulus between the edge of the impeller and the grit hopper wall being filled with small air bubbles. The solids that will normally find their way to the hopper with the grit particles are now rejected by the floatation provided by the bubbles. The unwanted solids, rags, paper and other light materials are floated upwards where the surface currents move these solids out of the trap.

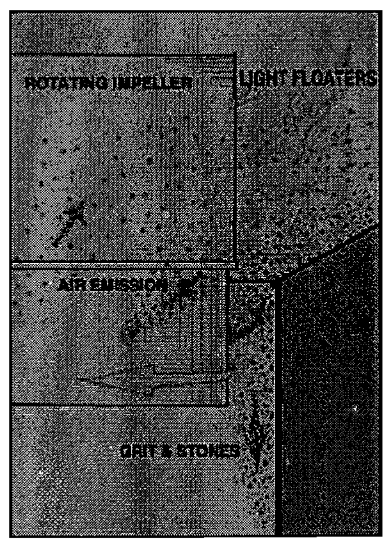
This innovation provides the ideal vortex inducing flow pattern, whilst every solid particle that will enter the 'trapped zone' will pass through the selective air curtain. Therefore the two main features of a grit trap, circular flow and classification, are satisfactorily provided.

The continuous aeration of the incoming flow at this location in the headworks is beneficial to the treatment process.

The illustration shows the importance of providing a controlled aperture for the passage of grit and stones to the collection hopper. The whole of the aperture (annulus) is filled with air bubbles.

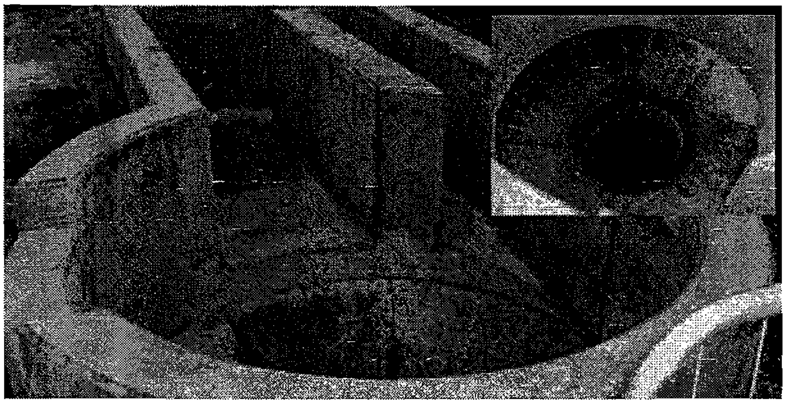
There are no fixed supports or pipes to interfere with the passage of the heavy solids.

The vanes of the impeller are now independent of the classification and serve the purpose only of generating the vortex flow.

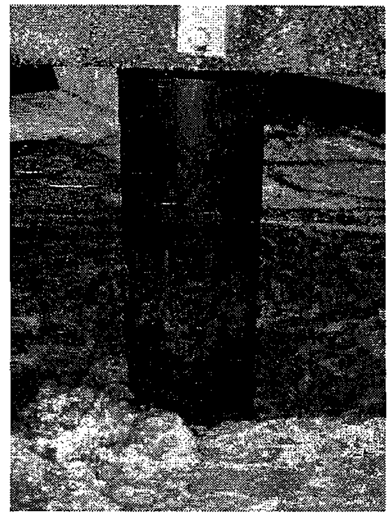


Pumping of the grit/water mixture can be performed by air-lift pump or motorised grit pumps.

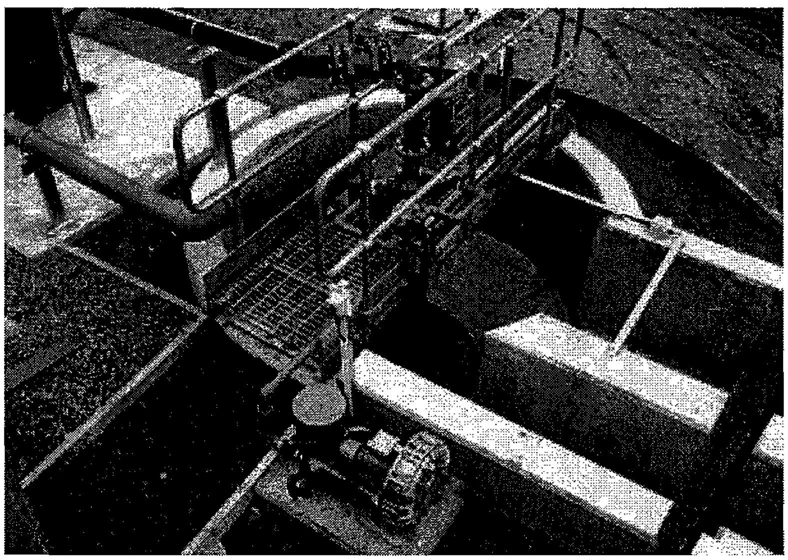
Eimco Water Technologies manufacture and supply the full range of grit separation and grit processing equipment.



Civil construction and installation.

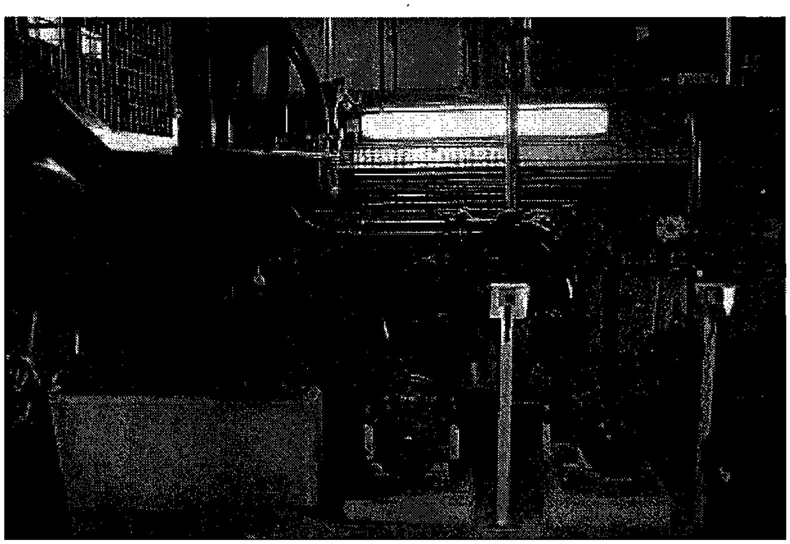


The effects of the continuous aeration can be clearly seen on the tank surface.



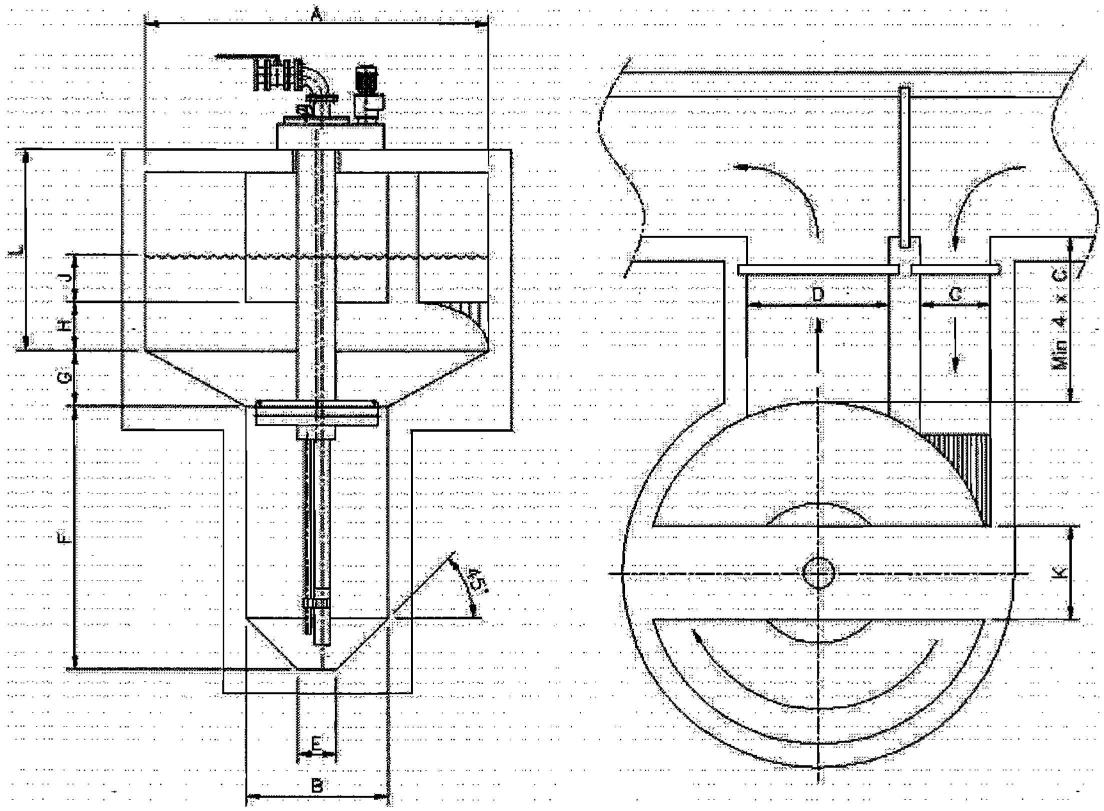
The completed Jetair installation.

The small additional blower is designed for quiet operation.



The new Jetair Grit Trap will be supplied with the conventional methods of grit transfer.

Jones+Attwood® JetAir



JetAir Grit Trap dimensions in metres

Jetair Size	Flow 1/sec	A	B	C	D	E	F	G	H	J	K	L
A50	50	1.83	1.0	0.305	0.61	0.30	1.40	0.30	0.30	0.20	0.80	1.10
A100	110	2.13	1.0	0.380	0.76	0.30	1.40	0.30	0.30	0.30	0.80	1.10
A200	180	2.43	1.0	0.450	0.90	0.30	1.35	0.40	0.30	0.40	0.80	1.15
A300	310	3.05	1.0	0.610	1.20	0.30	1.55	0.45	0.30	0.45	0.80	1.35
A550	530	3.65	1.5	0.750	1.50	0.40	1.70	0.60	0.51	0.58	0.80	1.45
A900	880	4.87	1.5	1.00	2.00	0.40	2.20	1.00	0.51	0.60	0.80	1.85
A1300	1320	5.48	1.5	1.10	2.20	0.40	2.20	1.00	0.61	0.63	0.80	1.85
A1750	1750	5.80	1.5	1.20	2.40	0.40	2.50	1.30	0.75	0.70	0.80	1.95
A2000	2200	6.10	1.5	1.20	2.40	0.40	2.50	1.30	0.89	0.75	0.80	1.95

Please note – larger sizes are available. Request details if required.

EIMCO WATERTECHNOLOGIES

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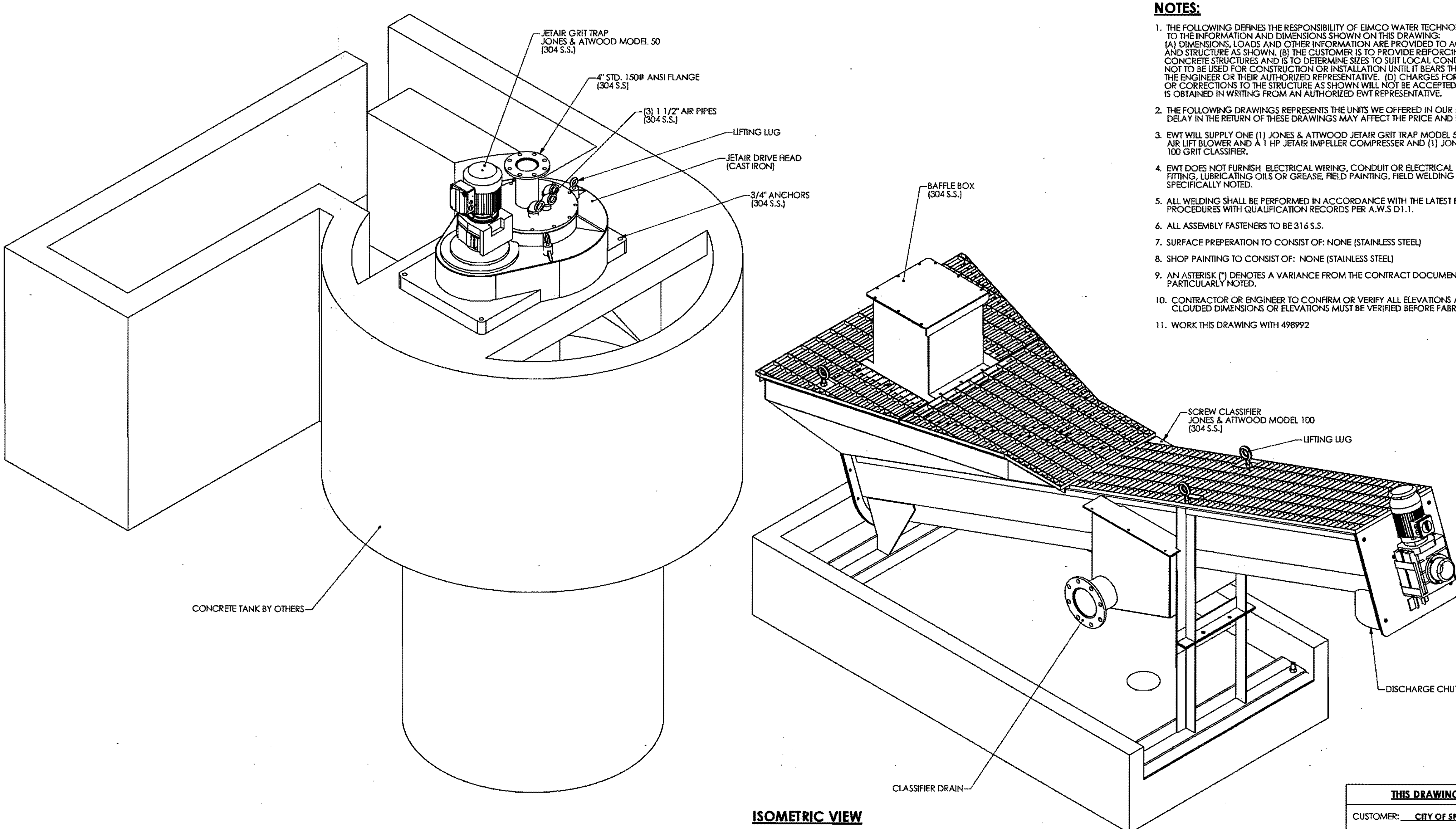
Fax: +44 (0) 1384 371937



ISOMETRIC VIEW

NOTES:

1. THE FOLLOWING DEFINES THE RESPONSIBILITY OF EIMCO WATER TECHNOLOGIES (EWT) WITH REGARD TO THE INFORMATION AND DIMENSIONS SHOWN ON THIS DRAWING:
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2. THE FOLLOWING DRAWINGS REPRESENTS THE UNITS WE OFFERED IN OUR PROPOSAL. ALTERATIONS OR DELAY IN THE RETURN OF THESE DRAWINGS MAY AFFECT THE PRICE AND DELAY SHIPMENT.
3. EWT WILL SUPPLY ONE (1) JONES & ATTWOOD JETAIR GRIT TRAP MODEL 50 WITH A 5 HP AIR SCOUR, AIR LIFT BLOWER AND A 1 HP JETAIR IMPELLER COMPRESSER AND (1) JONES & ATTWOOD MODEL 100 GRIT CLASSIFIER.
4. EWT DOES NOT FURNISH ELECTRICAL WIRING, CONDUIT OR ELECTRICAL EQUIPTMENT, PIPING, VALVES OR FITTING, LUBRICATING OILS OR GREASE, FIELD PAINTING, FIELD WELDING OR ERECTION EXCEPT AS SPECIFICALLY NOTED.
5. ALL WELDING SHALL BE PERFORMED IN ACCORDANCE WITH THE LATEST EDITION OF A.W.S. WELDING PROCEDURES WITH QUALIFICATION RECORDS PER A.W.S D1.1.
6. ALL ASSEMBLY FASTENERS TO BE 316 S.S.
7. SURFACE PREPERATION TO CONSIST OF: NONE (STAINLESS STEEL)
8. SHOP PAINTING TO CONSIST OF: NONE (STAINLESS STEEL)
9. AN ASTERISK (*) DENOTES A VARIANCE FROM THE CONTRACT DOCUMENTS AND SHOULD BE PARTICULARLY NOTED.
10. CONTRACTOR OR ENGINEER TO CONFIRM OR VERIFY ALL ELEVATIONS AND DIMENSIONS. CLOUDED DIMENSIONS OR ELEVATIONS MUST BE VERIFIED BEFORE FABRICATION.
11. WORK THIS DRAWING WITH 498992

THIS DRAWING IS CERTIFIED FOR:

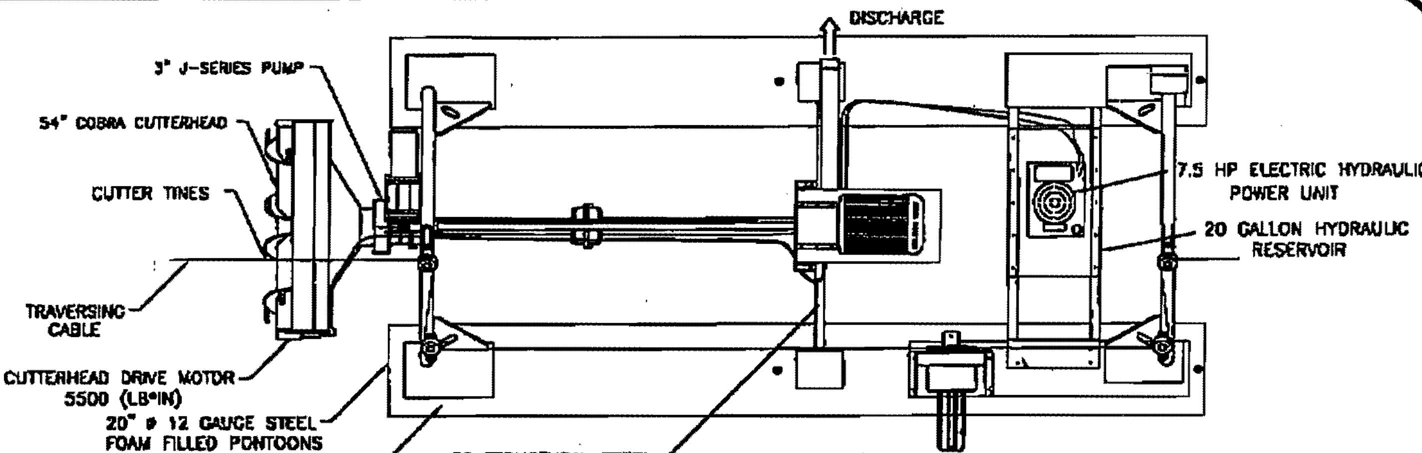
CUSTOMER: **CITY OF SPRINGFIELD, GEORGIA**
CUSTOMER ORDER NUMBER: **001206**
EWT ORDER NUMBER: **CSW0000028**
PROJECT: **CITY OF SPRINGFIELD WWTF**
PROJECT LOCATION: **SPRINGFIELD, GA**
CONSULTING ENGINEER: **NONE**
BY: **KURT BOUWHUIS** DATE: **JULY 7, 2006**

EIMCO WATER TECHNOLOGIES — A GL&V COMPANY — D

REVISION	EN	BY	APPR.	DATE			
					D	ORIGINAL S.O.	
					C	CSW0000028	
					B	DATE	7/7/2006
INITIAL RELEASE					A	DRAWN	KRB
						CHECK'D	JLQ

GENERAL ARRANGEMENT
MODEL50 JETAIR W/MODEL 100 CLASSIFIER

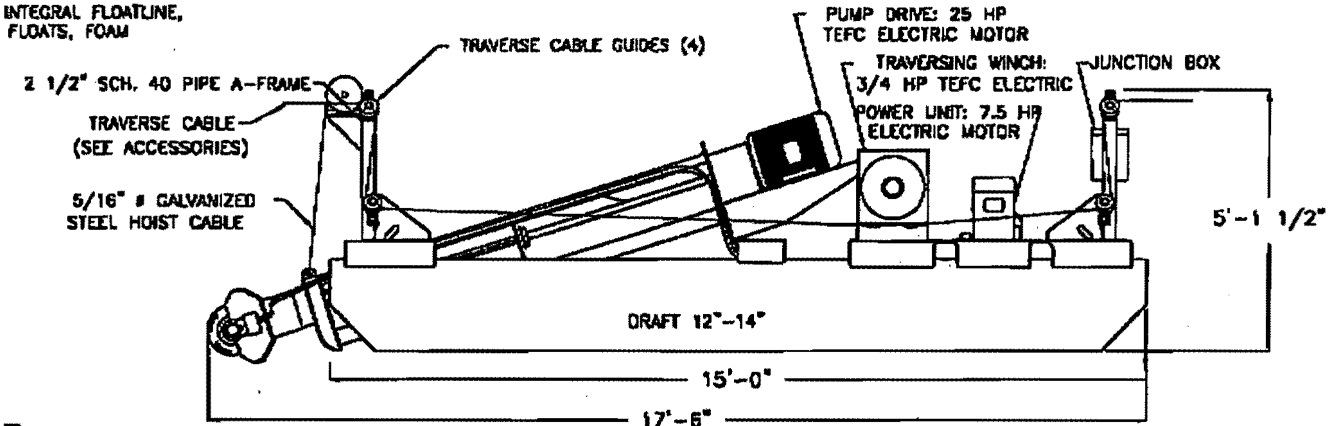
DO NOT SCALE PRINTS
REF. FROM
SHEET 1 OF 1
DWG. NO. 498991
REV A



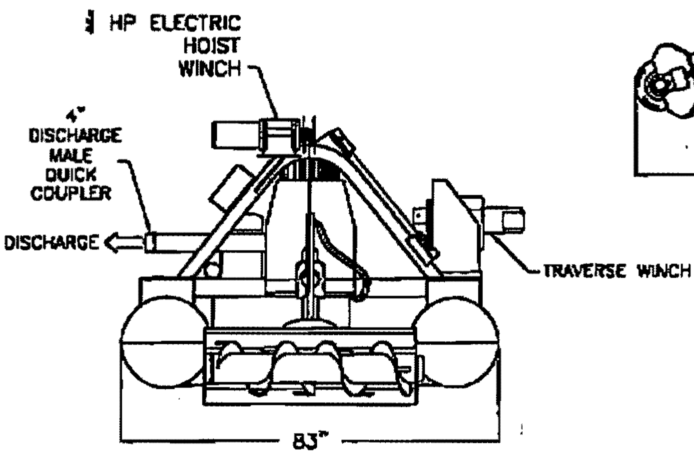
TOP VIEW

AVAILABLE ACCESSORIES:

TRAVERSE SYSTEMS..........................2 POST, 4 POST
CONTROL PANEL..............................NEMA 3R STANDARD
CONTROL CABLE..............................TYPE SO
FLOATING DISCHARGE SYSTEMS.......ALUMINUM INTEGRAL FLOATLINE, BALL TYPE FLOATS, FOAM



SIDE VIEW



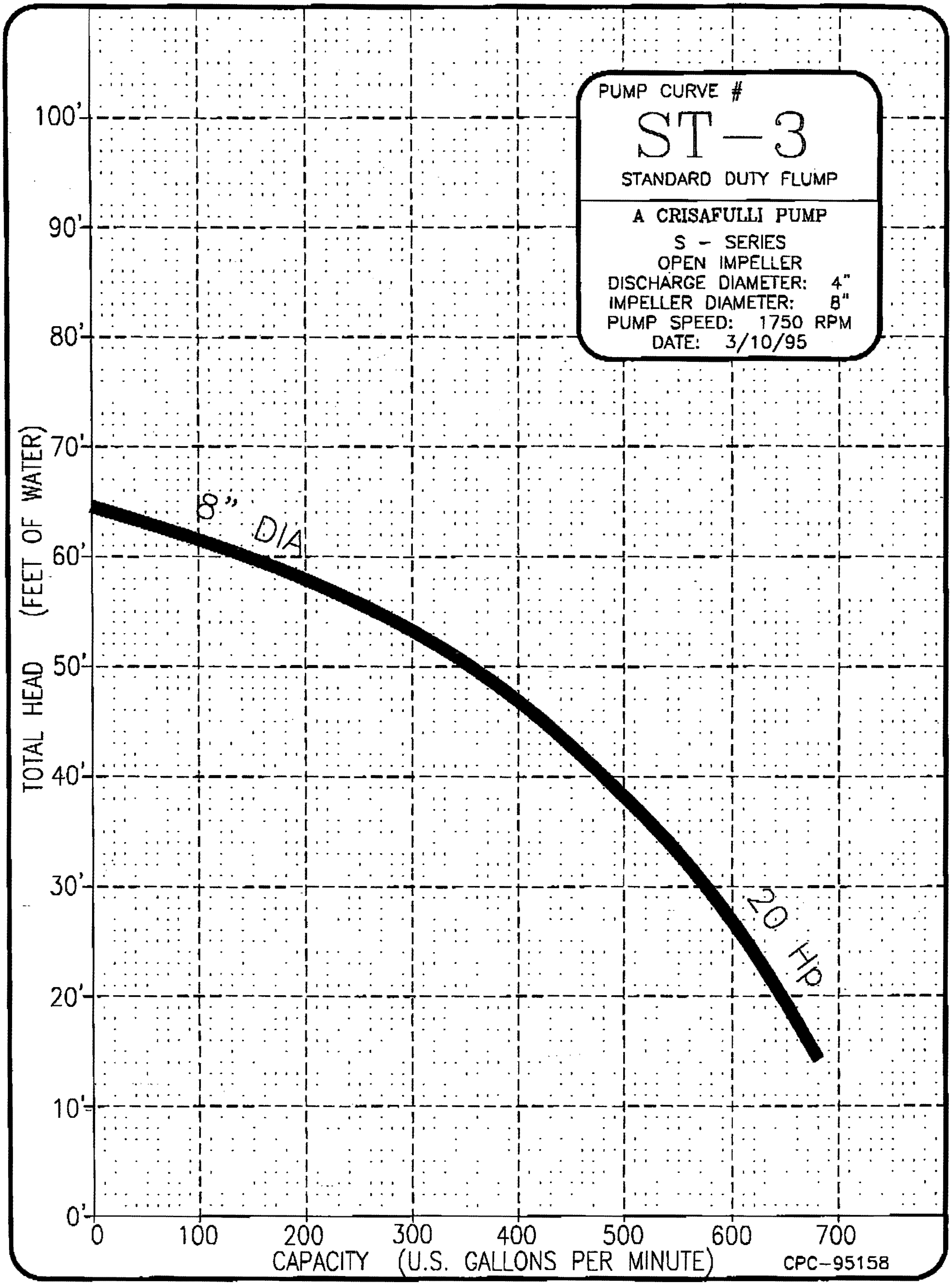
FRONT VIEW

REV. 3 MDM 1/8/03

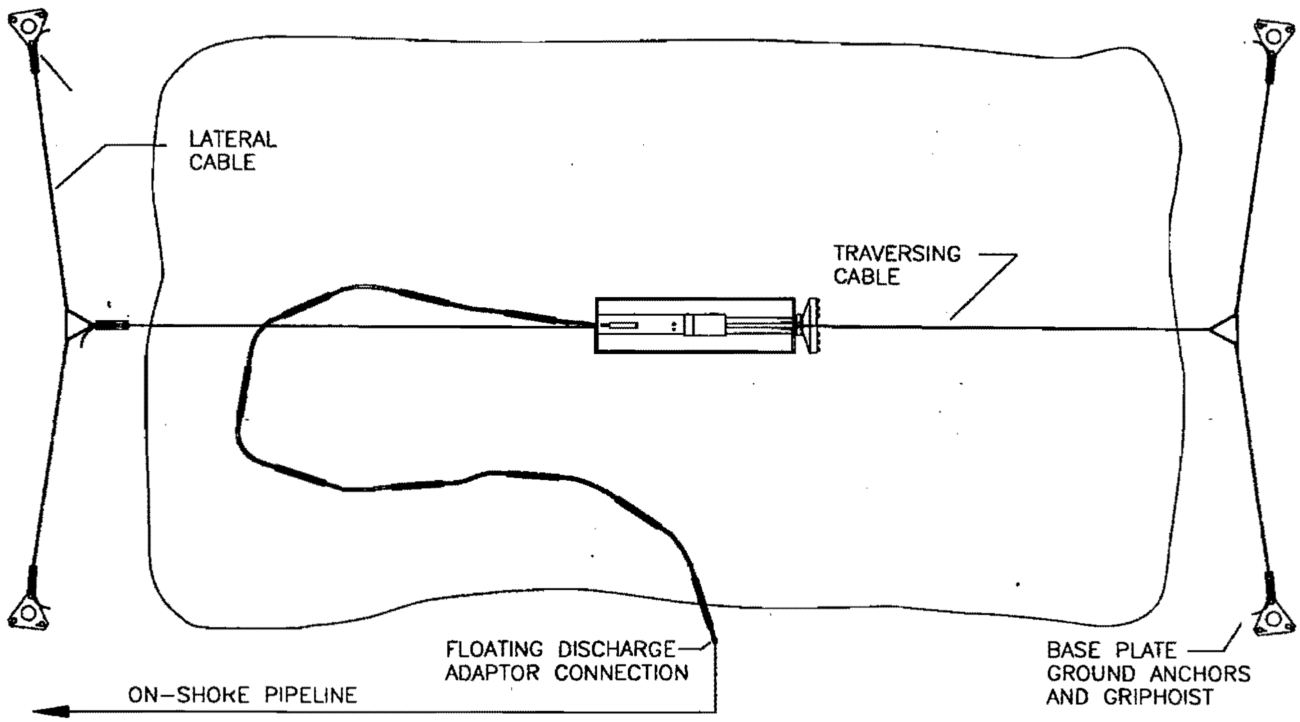
SPS Crisafulli
PHONE: (406) 365-3393
FAX: (406) 365-8088
Sludge Removal Systems

STANDARD DUTY FLUMP

Dwn By: MJB	Ckd.: D.R.T.	Date: 4-6-95	Dwg.#: CPC 94524

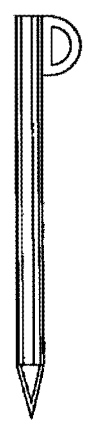

PUMP CURVE #
ST-3
STANDARD DUTY FLUMP
A CRISAFULLI PUMP
S - SERIES
OPEN IMPELLER
DISCHARGE DIAMETER: 4"
IMPELLER DIAMETER: 8"
PUMP SPEED: 1750 RPM
DATE: 3/10/95
8" DIA
20 HP
TOTAL HEAD (FEET OF WATER)
0'
10'
20'
30'
40'
50'
60'
70'
80'
90'
100'
0
100
200
300
400
500
600
700
CAPACITY (U.S. GALLONS PER MINUTE)
CPC-95158

PROPULSION • POSITIONING • TRAVERSING • DREDGING



*FEATURES

TRAVERSING CABLE	400' STANDARD, 3/8" DIAMETER 7X19 GALVANIZED STEEL CABLE (OPTIONAL LENGTHS AVAILABLE)
TENSION TRIANGLES	(2) 1/2" DIAMETER A36 STEEL TENSION TRIANGLES WITH FULL PENETRATION WELDS
LATERAL CABLES	210' STANDARD, C-16 GALVANIZED STEEL CABLE (OPTIONAL LENGTHS AVAILABLE)
GRIPHOISTS (TM)	(5) TWO TON GRIPHOIST CABLE TENSIONERS WITH HANDLES
BASE PLATES	(4) 3/16" STEEL TRIANGLE BASE PLATES
GROUND ANCHORS	(12) 48" LONG A36 STEEL ANCHORS, POINTED WITH D-RING HANDLES



OPTIONAL FEATURES

CORROSION RESISTANT	STAINLESS STEEL COMPONENTS (EXCEPT GRIPHOISTS)

GROUND ANCHORS

THE 4 LATERAL CABLES MAY EITHER BE ANCHORED WITH 3 GROUND ANCHORS & TRIANGLE BASE PLATE, A TREE, A ROCK OR ANYTHING SOLID.

* THESE FEATURES MAY CHANGE WITHOUT NOTICE

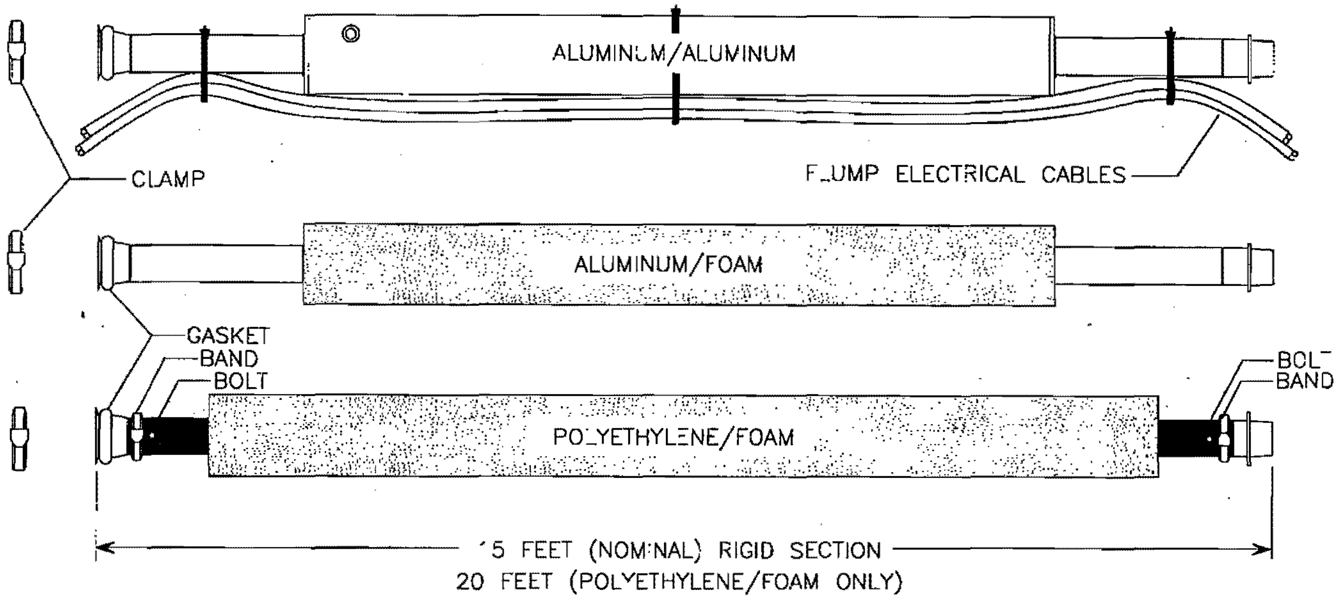
REV. (4) BY J.L.B. 10-19-95

SRS Crisafulli — Sludge Removal Systems

DREDGE
4-POST MANUAL TRAVERSING
CABLING SYSTEM

Dwn By: CKR	Ckd.:	Date: 7/10/91	Dwg.#: CPC-91344

LIGHTWEIGHT • FLOATING • SLUDGE/SLURRY TRANSFER • BOLTLESS



*FEATURES

ASSEMBLED LENGTH . . 20 FEET (6.1 METERS)

RIGID SECTION . . . 15 FEET LONG (4.57 METERS) RIGID PIPE SECTION WITH MALE/FEMALE IRRIGATION QUICK COUPLERS BANDED**& BOLTED (POLY ONLY) ON EACH END, RUBBER GASKET, LOCKING CLAMP, AND AN INTEGRAL 10 FOOT FLOAT.

MATERIALS OF CONSTRUCTION . . . ALUMINUM OR PE3408 UHMW POLYETHYLENE

FLEXIBLE SECTION . 5 FEET LONG (1.52 METERS) FLEXIBLE 100 PSI HOSE SECTION WITH MALE/FEMALE IRRIGATION QUICK COUPLERS (2)2-BOLT CLAMPS*** ON EACH END, RUBBER GASKET, AND A LOCKING CLAMP.

MATERIALS OF CONSTRUCTION . . AN ABRASION RESISTANT CORE, NYLON (OR VYTACORD) REINFORCING AND AN EXTERIOR ABRASION RESISTANT COVER.

QUICK COUPLERS . . GALVANIZED STEEL AND/OR ALUMINUM IRRIGATION QUICK COUPLERS.

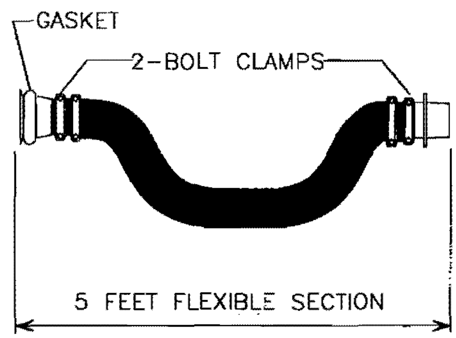
CABLE CLAMPS . . . . ADJUSTABLE ELASTOMERIC HOSE/CABLE CLAMPS (2 PER FLOAT ASSEMBLY)

OPTIONAL FEATURES

CORROSION RESISTANT . STAINLESS STEEL FASTENERS AND COUPLERS

ASSEMBLY LENGTH . 10 FEET TO 40 FEET

*THESE FEATURES MAY CHANGE WITHOUT NOTICE. ** STAINLESS STEEL
*** MILD STEEL ZINC COATED



LIGHTWEIGHT FLEXIBILITY

THE CRISAFULLI INTEGRAL FLOATING DISCHARGE LINE SOLVES THE PROBLEM GENERALLY ASSOCIATED WITH STANDARD DISCHARGE SYSTEMS. THE SYSTEM IS DESIGNED TO ALLOW MAXIMUM FLEXIBILITY WITHOUT HOSE KINKING OR WITHOUT FRETTING THE HOSE. THE INTEGRAL FLOATS REDUCE THE AMOUNT OF DRAG CAUSED BY THE FLOATS AS THEY MOVE THROUGH THE LIQUID OR SLUDGE. THE INTEGRAL FLOATS ALSO ALLOW THE SECTIONS TO BE STACKED WITHOUT THE QUICK DISCONNECTS BEING DAMAGED. EACH SECTION CAN BE HANDLED EASILY AND QUICKLY SET UP.

NOMINAL DIAMETER	RIGID SECTION						FLEX SECTION
	ALUM/ALUM		ALUM/FOAM		POLY/FOAM		
	RATED PSI	LBS.	RATED PSI	LBS.	RATED PSI	LBS.	LBS.
3"	125	30	125	30	160	40	10
4"	125	35	125	35	110	50	15
6"	125	40	125	40	80	90	20
8"	95	65	95	55	65	125	30
10"			75	75	50	165	45
12"			75	105	50	235	55

REV. 2 (JLB) 6-27-96

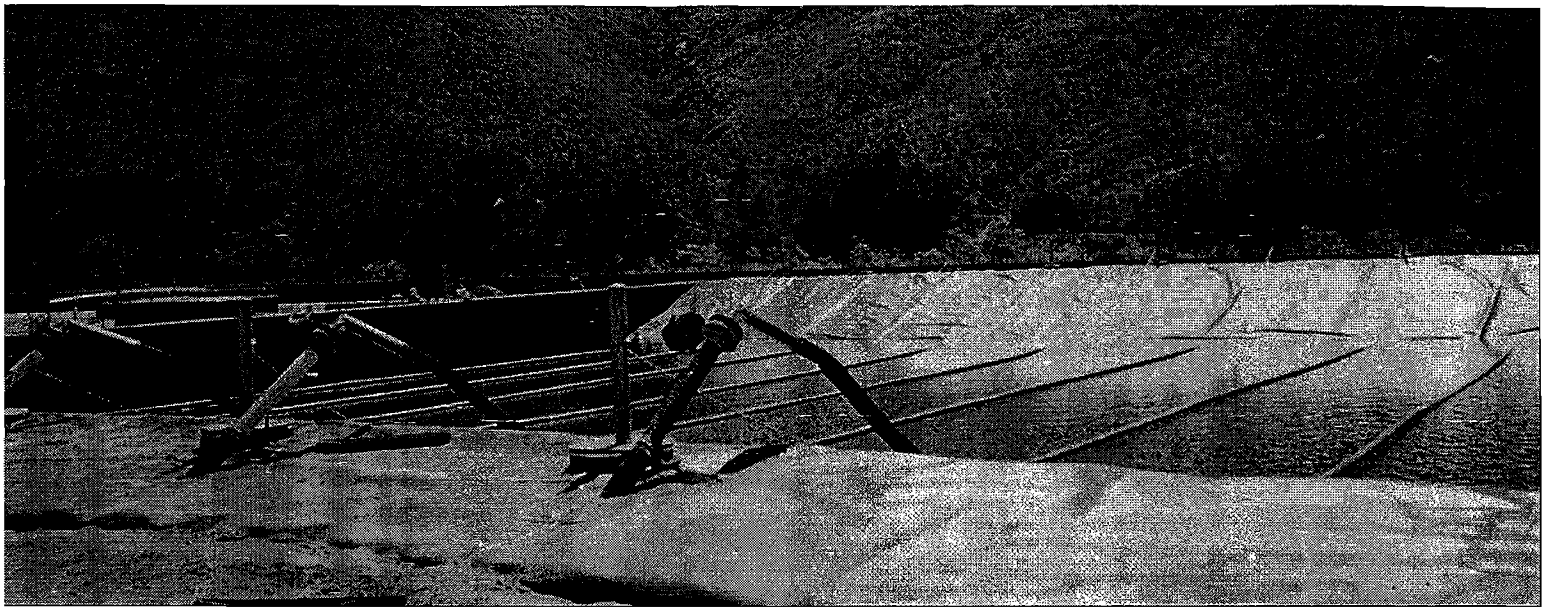
SRS
Crisafulli Sludge Removal Systems

INTEGRAL FLOATING DISCHARGE PIPELINE

Dwn. By: CKR | Ckd.: | Date: 7/3/91 | Dwg.#: CPC-91338

PARKSON CORPORATION

BIOLAC® WASTEWATER TREATMENT SYSTEM



Biolac® Wastewater Treatment System
Extended sludge age biological technology

This innovative process features

- Low-loaded activated sludge technology
- High oxygen transfer efficiency delivery system
- Exceptional mixing energy from controlled aeration chain movement
- Simple system construction

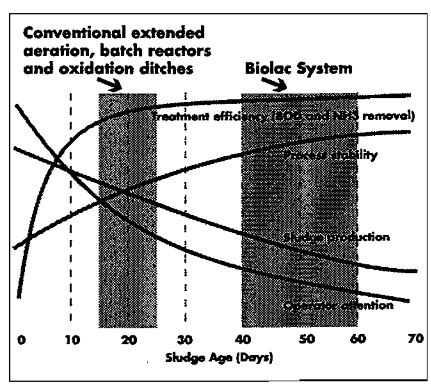
The Biolac System is an innovative activated sludge process using extended retention of biological solids to create an extremely stable, easily operated system.

The capabilities of this unique technology far exceed ordinary extended aeration treatment. The Biolac process maximizes the stability of the operating environment and provides high efficiency treatment. The design ensures the lowest-cost construction and guarantees operational simplicity. Over 500 Biolac Systems are installed throughout North America treating municipal wastewater and many types of industrial wastewater.

The Biolac system utilizes a longer sludge age than other aerobic systems. Sludge age, also known as SRT (solids retention time) or MCRT (mean cell residence time), defines the operating characteristics of any aerobic biological treatment system. A longer sludge age dramatically lowers effluent BOD and ammonia levels. The Biolac long sludge age process produces BOD levels of less than 10 mg/l and complete nitrification (less than 1 mg/l ammonia). Minor modifications to the system will extend its capabilities to denitrification and biological phosphorus removal.

While most extended aeration systems reach their maximum mixing capability at sludge ages of approximately 15-25 days, the Biolac System efficiently and uniformly mixes the aeration volumes associated with 30-70 day sludge age treatment.

The large quantity of biomass treats widely fluctuating loads with very few operational changes. Extreme sludge stability allows sludge wasting to non-aerated sludge ponds or basins and long storage times.



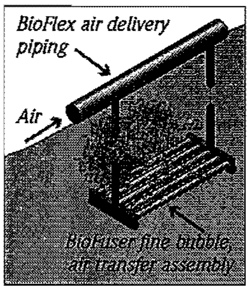
Aeration Components

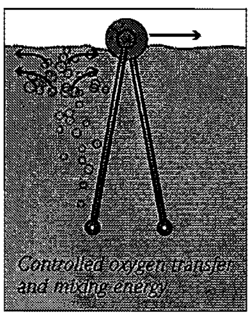
SIMPLE PROCESS CONTROL AND OPERATION

The control and operation of the Biolac® process is similar to that of conventional extended aeration. Parkson provides a very basic system to control both the process and aeration. Additional controls required for denitrification, phosphorus removal, dissolved oxygen control and SCADA communications are also available.

AERATION SYSTEM COMPONENTS

The ability to mix large basin volumes using minimal energy is a function of the unique BioFlex® moving aeration chains and the attached BioFuser® fine bubble diffuser assemblies. The gentle, controlled back and forth motion of the chains and diffusers distributes the oxygen transfer and mixing energy evenly throughout the basin area. No additional airflow is required to maintain mixing.





Stationary fine-bubble aeration systems require 8-10 CFM of air per 1000 cu. ft. of aeration basin volume. The Biolac System maintains the required mixing of the activated sludge and suspension of the solids at only 4 CFM per 1000 cu.ft. of aeration basin volume. Mixing of a Biolac basin typically requires 35-50 percent of the energy of the design oxygen requirement. Therefore, air delivery to the basin can be reduced during periods of low loading without the risk of solids settling out of the wastewater.

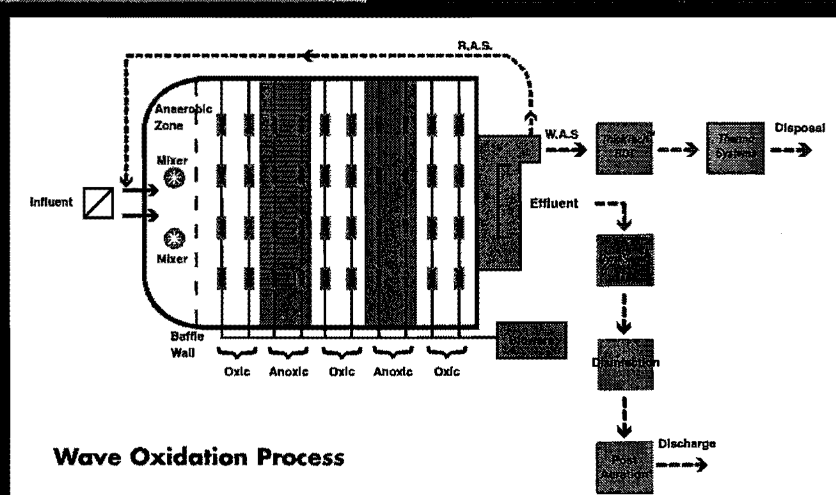
SYSTEM CONSTRUCTION

A major advantage of the Biolac system is its low installed cost. Most systems require costly in-ground concrete basins for the activated sludge portion of the process. A Biolac system can be installed in earthen basins, either lined or unlined. The BioFuser fine bubble diffusers require no mounting to basin floors or associated anchors and leveling. These diffusers are suspended from the BioFlex aeration chains above the basin floor. The only concrete structural work required is for the simple internal clarifier(s) and blower/control buildings.

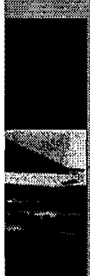
Biological Nutrient Removal

Simple control of the air distribution to the BioFlex chains creates moving waves of oxic and anoxic zones within the basin. This repeated cycling of environments nitrifies and denitrifies the wastewater without recycle pumping or additional external basins. This mode of Biolac operation is known as the Wave Oxidation™ process. No additional in-basin equipment is required and simple timer-operated actuator valves regulate manipulation of the air distribution.

Biological phosphorus removal can also be accomplished by incorporating an anaerobic zone.

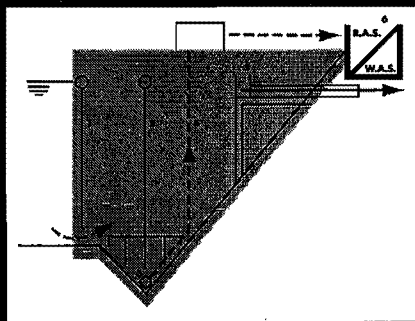


Wave Oxidation Process



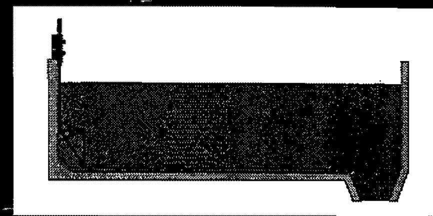
Type "R" Clarifier

Land space and hydraulic efficiencies are maximized using the type "R" clarifier. The clarifier design incorporates a common wall between the clarifier and aeration basin. The inlet ports in the bottom of the wall create negligible hydraulic headloss and promote efficient solids removal by filtering the flow through the upper layer of the sludge blanket. The hopper-style bottom simplifies sludge concentration and removal, and minimizes clarifier HRT. The sludge return airlift pump provides important flexibility in RAS flows with no moving parts. All maintenance is performed from the surface without dewatering the clarifier.



Type "SS" Clarifier

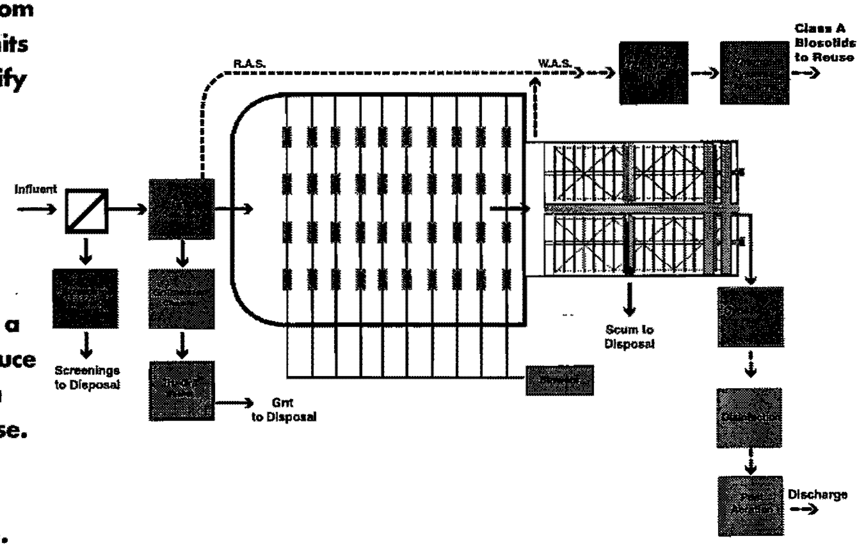
Higher flow systems incorporate a flat-bottom internal clarifier utilizing the Parkson SuperScraper™ sludge removal system. This clarifier design maintains the efficiencies of the common wall layout while providing ample clarification surface area within the footprint of the aeration basin width. The SuperScraper system moves settled solids along the bottom of the clarifier to an integral collection trough. The unique design of the scraper blades and gentle forward movement of the SuperScraper system concentrates the biological solids as they are moved along the bottom of the clarifier without disturbing the sludge blanket.



A Parkson *Complete* Wastewater Treatment System

The Parkson "Complete" system featured here utilizes the Biolac® process with two flat-bottom internal Type SS clarifiers. SuperScraper™ units are installed in the clarifier bottoms to simplify sludge removal. Influent screening with grit removal and appropriate residuals management such as washing, dewatering and conveying are included.

Sludge from the clarifiers is sent to the ThickTech™ rotary drum thickener and on to a THERMO-SYSTEM™ solar sludge dryer to reduce the volume of sludge by 50% and produce a Class "A" product suitable for beneficial reuse. Clarifier effluent is polished by a DynaSand® filter followed by disinfection and post-aeration as the final steps prior to discharge.



PARKSON CORPORATION

ISO 9001:2000 Certified
Quality Management System

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Brazil
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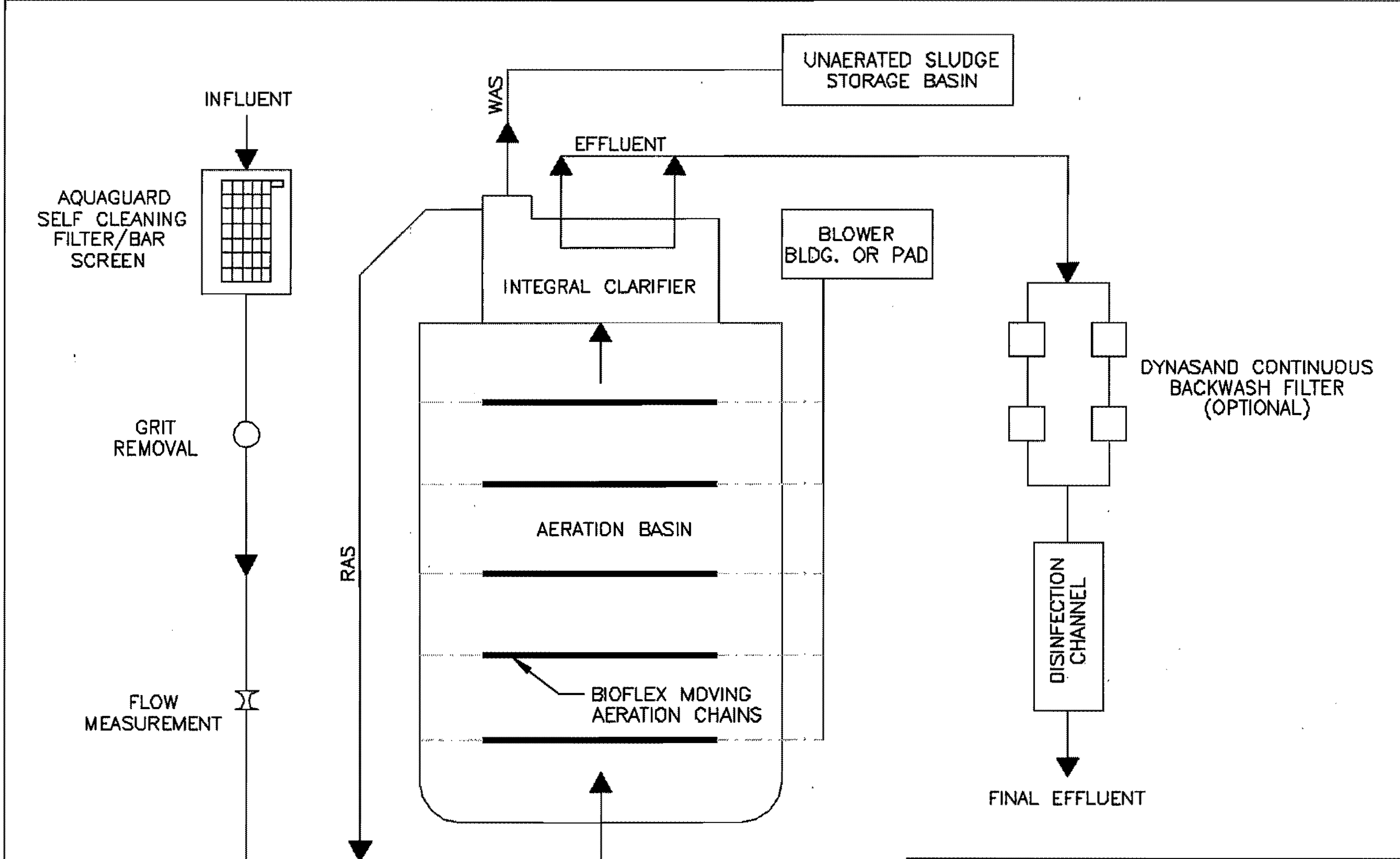
P/F 55.11.4688.0336

BL012406



PARKSON CORPORATION

BIOLAC LONG SLUDGE AGE SYSTEM
FLOW DIAGRAM

Drawn By	Checked By	Approved By	Micro Rev.	CAD No.	Loc. status
G.C.				SD1	SD1
Date 2/1/96	Date	Date	Date	DWG Scale NONE	CAD Int scale 1
Location			Dwg. No. SD-1		Rev. A

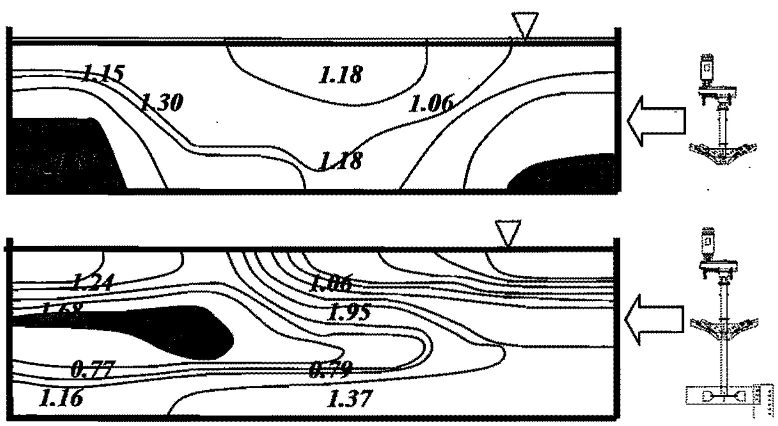
INTRODUCING THE CARROUSEL® 3000

When EIMCO introduced the Carrousel System in the 1970s, most communities were simply trying to achieve secondary treatment—20/20 (BOD/TSS) permits. Over the last three decades, permits have become more stringent (usually requiring nutrient removal), the desire to save power more important, and space available for new plants more limited. The Carrousel 3000, the culmination of more than 29 years of continuous improvement of the Carrousel System, has responded to these market changes. Some milestones in the Carrousel process are shown below:

- *1976 - EIMCO brings the Carrousel® oxidation ditch to the U.S*
- *1979 - EIMCO installs the <u>first BNR plant in the U.S.</u> designed on process kinetics*
- *1987 - EIMCO introduces the DenitIR® Carrousel® system for <u>free internal recycle</u>*
- *1989 - EIMCO introduces the <u>dual-impeller</u> aerator*
- *1990 - EIMCO introduces the A^2C process, <u>reducing the biological nutrient removal process from five stages to three.</u>*
- *2000 - EIMCO introduces the Deep Tank Carrousel for <u>depths greater than 20 ft.</u>*
- *2001 - EIMCO introduces the <u>ACE™</u> control system to control power use 24-hours/day.*
- *2004 - EIMCO introduces the **<u>ExcellAerator</u>** for maximum process control & energy savings*

EIMCO's pilot-scale plant in Salt Lake City, Utah

The EIMCO **ExcellAerator** incorporates a lower turbine system on a common shaft with the surface aerating impeller. Velocity enhancing baffles (patent pending) are installed near the lower turbine. The ExcellAerator allows 70-85% power turndown while maintaining sufficient mixing throughout the basin.



INSIDE WALL | **FLOOR** | OUTSIDE WALL

The EIMCO ExcellAerator, inherent power turndown capability, innovative basin configurations, and our effective (but simple) ACE™ control system:

- Lower Power Costs
- Improve Nitrogen Removal
- Reduce Footprint
- Reduce Maintenance Requirements

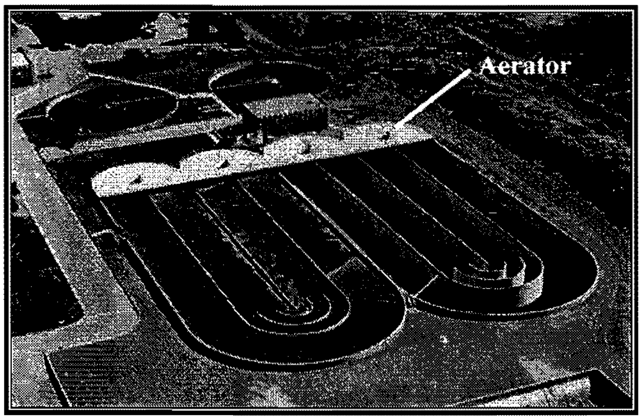
(see pages iv and v)

VELOCITY PROFILE IN A FULL-SCALE OXIDATION DITCH

Numbers are velocities in feet per second in the channel cross-section from <u>a full-scale test</u>. The low velocities are shown in red. The low floor velocities along the inside and outside walls are eliminated with the addition of the EIMCO lower turbine system.

The EIMCO Carrousel® System Description

Award Winning Process For Biological Treatment



KEY FEATURES

- ***BOD, TSS, AND NH_3-N REMOVAL***
- ***FEWER PIECES OF EQUIPMENT MEANS LOWER INSTALLED COST***
- ***SIMPLE AND EASY TO OPERATE***
- ***WON OVER 70 EPA, STATE AND LOCAL AWARDS SINCE 1988***
- ***HYDRAULICALLY EFFICIENT SO 70-85% POWER TURNDOWN IS POSSIBLE***
- ***ON SITE PROCESS TRAINING AND EIMCO'S TECHNICAL SUPPORT***

Background

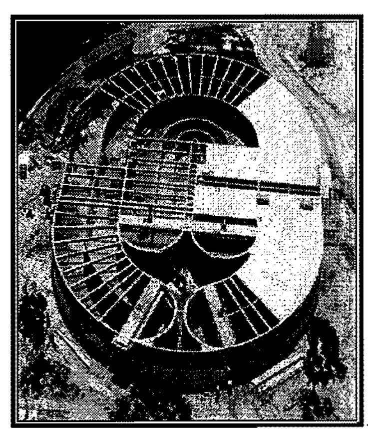
The EIMCO Carrousel System is one of the most successful and widely accepted processes available for biological wastewater treatment. More than 619 treatment plants in the United States and 950 worldwide depend on Carrousel Systems to remove organic contaminants and provide biological nutrient removal. Among owners and operators, the Carrousel System is universally praised for its stability, simplicity, ease of operation and maintenance, low operating cost, and consistent effluent quality.

Developed by DHV Consulting Engineers of the Netherlands, the Carrousel System is unique in that every installation is custom engineered using a proprietary hydraulic model. Eimco Water Technologies engineers use this model to evaluate the energy requirements of a proposed design, to efficiently match treatment capacity to actual requirements, and to define the most affordable layout for a specific site.

As a result, Carrousel System plants display extraordinary operating flexibility and energy economy. Their hydraulic efficiency provides full solids suspension with minimal mixing energy, allowing aeration input to be varied from full power to 15% -30% of the installed power. The ability to actively manage energy use in response to daily, seasonal and service life demand cycles offers the owner significant opportunities to minimize operating expense while maintaining strict permit compliance.

Physical Description

The Carrousel System is a closed loop, oxidation ditch reactor that provides the aerobic component of a very efficient activated sludge system. The layout is a typically a "hotdog" (schematic next page) or "folded over" (photo at top) design. Internal partition walls define flow channels. More creative design configurations are possible as shown in the picture to the right. Vertically mounted, large diameter, low-speed surface aerators are installed at the channel turns, slightly offset in the direction of flow from the centerline of internal partition walls. This arrangement allows the aerators to function as large-scale pumps, driving mixed liquor from upstream to downstream channels and establishing a constant flow velocity. It also divides the basin volume into complete mix and plug flow hydraulic environments, where short intervals of intense aeration and mixing alternate with longer intervals of relatively quiescent, but fully mixed conditions.

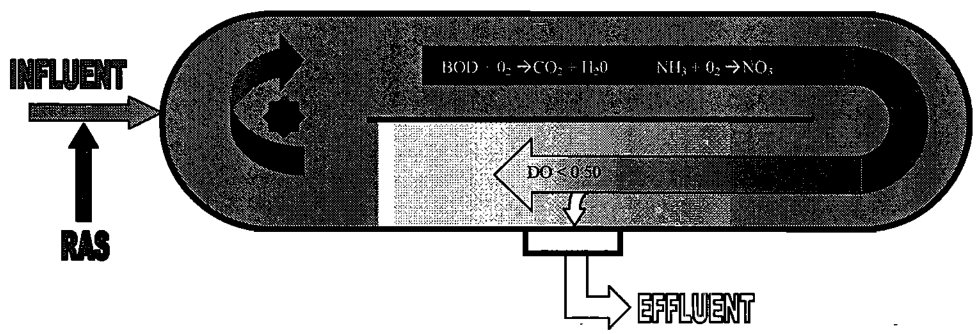


The EIMCO Carrousel® System Description (cont'd)

Award Winning Process For Biological Treatment

Operating Description

In the aeration zone, influent wastewater and returned activated sludge (RAS) are introduced under intense, concentrated mixing action, providing immediate dilution in a mixed liquor volume of 50 to 100 times the influent flow and eliminating the possibility of short circuiting. The concentration of aeration power in a confined volume enhances oxygen transfer efficiency and establishes a uniform dissolved oxygen profile throughout the channel depth.



As mixed liquor enters the downstream channel, the complete mix conditions give way to a plug flow environment in which the channel velocity maintains an energy level high enough to keep solids suspended, but low enough to allow progressive bioflocculation of the mixed liquor solids. In the channels, natural respiration of the biomass produces a gradual drop in DO concentration, which can be managed for various process objectives, including denitrification. The low DO entering the aeration zone also increases oxygen transfer. An overflow weir is located upstream of the aeration zone to take maximum advantage of oxygen management practices and bioflocculation in the downstream channels.

By concentrating the input of mixing and aeration energy in a small portion of the basin volume, and by using the channel velocity to maintain solids suspension in the larger volume, the Carrousel System provides more flexible, efficient aeration with fewer aerators than other oxidation ditch systems and with significantly lower overall power requirements than complete mix systems. The reduced number of aerators and their convenient location simplify and greatly reduce mechanical maintenance requirements.

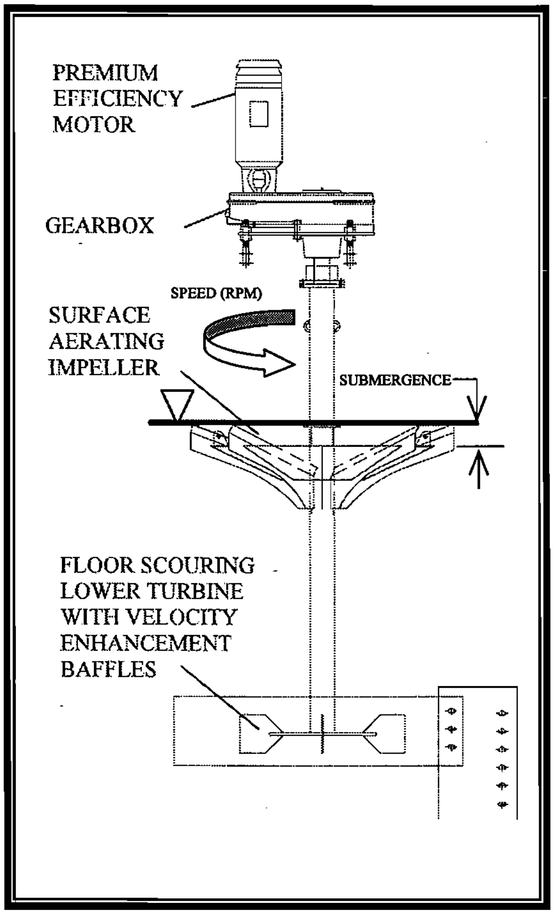
Maximum Mixing, Minimum Power

The operating economies described above depend on a reactor basin where channel velocity is maintained with **the smallest possible input of aeration energy**. All dimensions and specifications that influence this capability are evaluated using the DHV Carrousel System hydraulic model, including impeller type, impeller diameter, aerator rotational speed, aeration zone depth, channel depth and width. The resulting hydraulic efficiency ensures that solids remain in suspension using only a fraction of the installed power.

A Proposal of Excellence

The EIMCO Carrousel System proposed in this document will ensure your client of wastewater treatment performance that will reliably meet the plant's specified effluent discharge limits. In addition, it will provide the owner with a treatment system that is simpler, more stable, easier to operate and maintain and less expensive to operate than any other oxidation ditch configuration. It will provide a flexible platform for future upgrades should they be required by service area growth or more restrictive discharge regulations. Eimco engineers provide process training and start-up technical support so that Carrousel systems perform to their specifications from Day 1. For these reasons, the Carrousel system is a responsible technology investment for you and your client.

THE EXCELL™ AERATOR AND ACE™ CONTROL SYSTEM



The Carrousel process is an inherently efficient system, but it is the EIMCO ExcellAerator that extends that efficiency to all phases of a plant's life—from start-up to maturity. Most plants spend much of their life receiving influent loadings that are less than the design loadings. The ExcellAerator has a surface aerating impeller to provide aeration and mixing and a patented lower turbine system. The lower turbine increases floor velocity by 10-15% compared to older single-impeller designs. The ExcellAerator can draw only 15-30% of nameplate power and maintain sufficient mixing! Power to the aerator is controlled by (1) the rotational speed (rpm) of the impeller and (2) the submergence of the impeller blades.

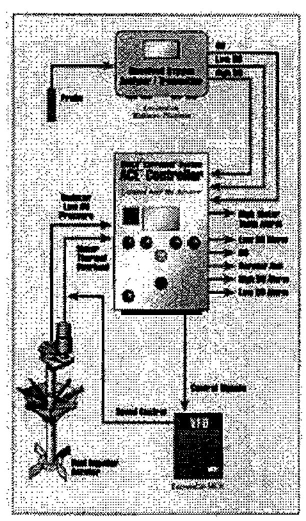
Power turndown saves communities thousands of dollars in energy annually. In addition, power turndown (or, more specifically, aeration turndown) is essential for nutrient removal plants. Without adequate power turndown, over-aeration often exhibits itself by producing copious quantities of "pin floc".

Engineers must design plants with installed aeration capacity that accommodates future loading and redundancy requirements. With the EIMCO process, operators can run the ExcellAerator at much less than the installed power, saving energy and achieving nutrient removal throughout the life of the plant.

EIMCO EXCELLAERATOR

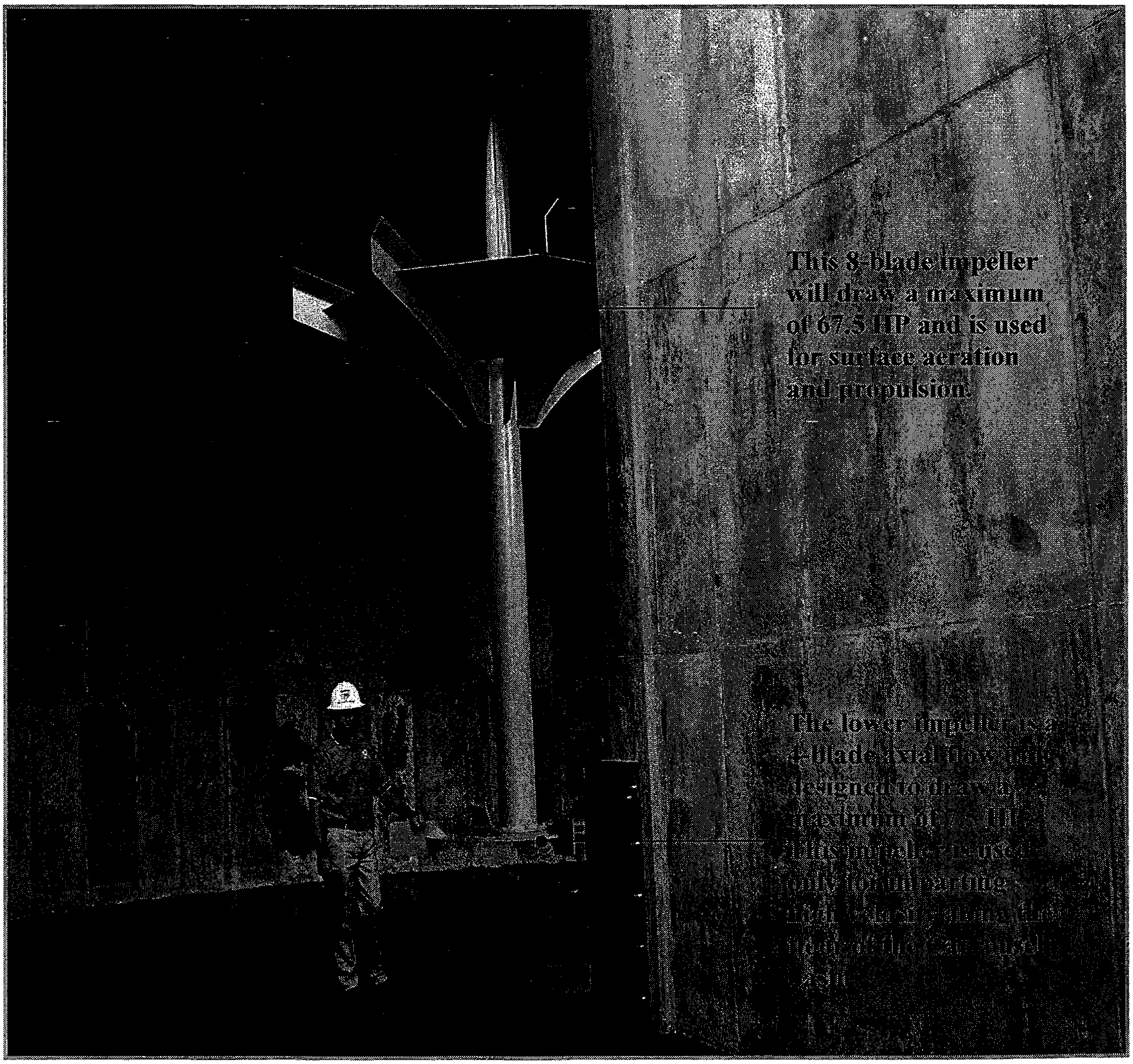
MAXIMUM POWER TURNDOWN
DESIGNED FOR THE LIFE OF THE PLANT

The EIMCO Automated Control of Energy (ACE™) System:



Eimco Water Technologies offers the optional ACE system to match delivered aeration power to the oxygen demand of the influent wastewater. The ACE system adjusts aerator power (by adjusting rotational speed of the impeller) to maintain dissolved oxygen in the Carrousel basin at an optimum setpoint. The ACE system is compatible with most plant SCADA systems and dissolved oxygen probes. The ACE system is custom-programmed by an Eimco engineer for each installation—taking into account the specific dissolved oxygen profile in the system, impeller size, and treatment goals. Our customers typically find the cost of the ACE system can be recovered in 2-4 years, based on power savings alone. The process benefits of the ACE system are equally important in nutrient removal plants. Through simple control of dissolved oxygen, the ACE system maximizes nitrogen and phosphorus removal 24 hours per day.

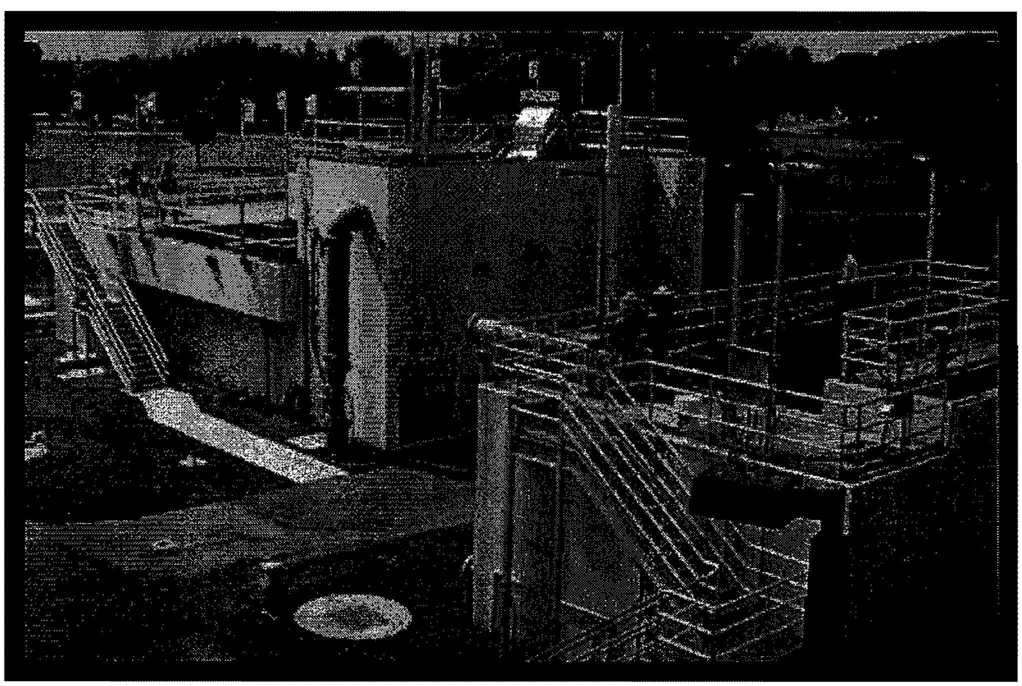
Eimco Dual Impeller Aerator



PARKSON CORPORATION

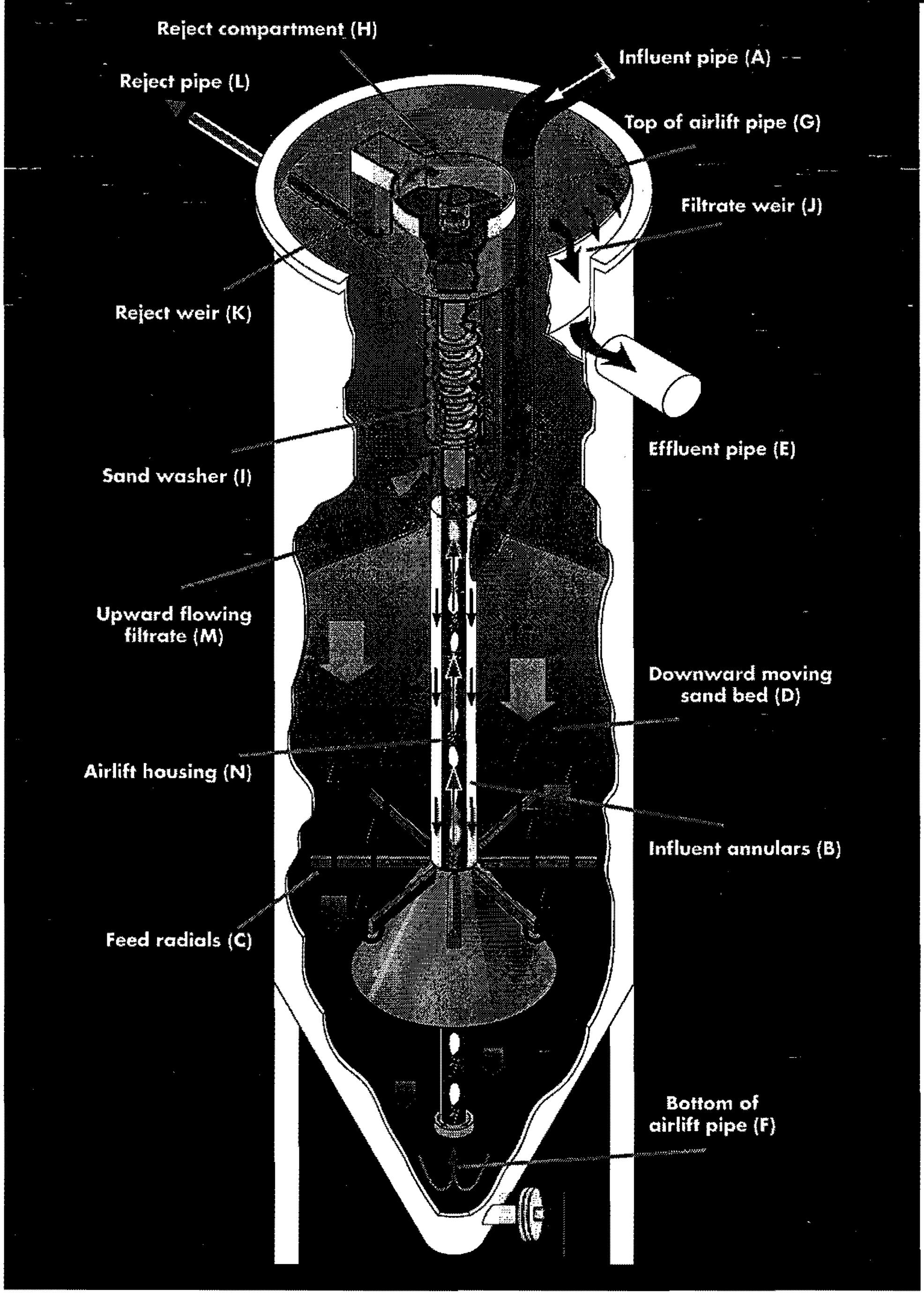
DYNASAND®

CONTINUOUS, UPFLOW, GRANULAR MEDIA FILTER



The DynaSand® Filter

Simplicity, low maintenance, outstanding performance



The DynaSand filter is an upflow, deep bed, granular media filter with continuous backwash. The filter media is cleaned by a simple internal washing system that does not require backwash pumps or storage tanks. The absence of backwash pumps means low energy consumption.

The DynaSand filter's deep media bed allows it to handle high levels of suspended solids. This heavy-duty performance may eliminate the need for pre-sedimentation or flotation steps in the treatment process in some applications.

The DynaSand filter is available in various sizes and configurations. This flexibility allows for customization to fit specific site and application requirements.

DynaSand Principles of Operation

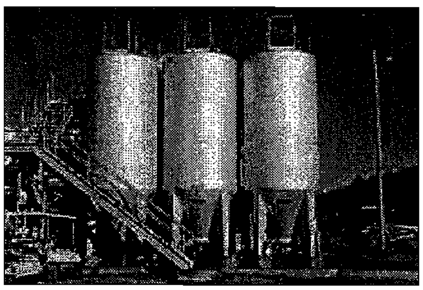
Influent Filtration Influent feed is introduced at the top of the filter (A) and flows downward through an annular section (B) between the influent feed pipe and airlift housing. The feed is introduced into the bottom of the sand bed through a series of feed radials (C) that are open at the bottom. As the influent flows upward (M) through the downward moving sand bed (D), organic and inorganic impurities are captured by the sand. The clean, polished filtrate continues to move upward and exits at the top of the filter over the filtrate weir (J) and out through the effluent pipe (E).

Sand Cleaning The sand bed containing captured impurities is drawn downward into the center of the filter where the airlift pipe (F) is located. A small volume of compressed air is introduced at the bottom of the airlift, drawing the sand into the airlift pipe. The sand is scoured within the airlift pipe at an intensity of 100-150 SCFM/ft^2. The effectiveness of this scouring process is vastly greater than what can be expected in conventional sand filtration backwash. The scouring dislodges any solid particles attached to the sand grains.

The dirty slurry is pushed to the top of the airlift (G) and into the reject compartment (H). From the reject compartment, the sand falls into the sand washer (I) and the lighter reject solids are carried over the reject weir (K) and out the reject pipe (L). As the sand cascades down through the concentric stages of the washer, it encounters a small amount of polished filtrate moving upward, driven by the difference in water level between the filtrate pool and the reject weir. The heavier, coarser sand grains fall through this small countercurrent flow while the remaining contaminants are carried back up to the reject compartment. The clean, recycled sand is deposited on the top of the sand bed where it once again begins the influent cleaning process and its eventual migration to the bottom of the filter.

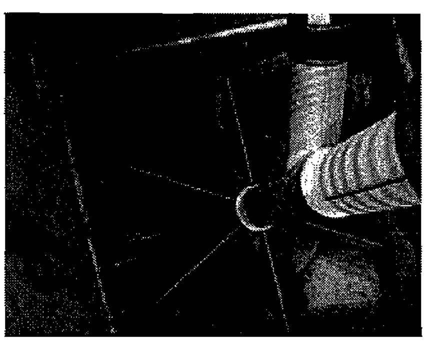
DynaSand® Filter Configurations

The DynaSand filter is available as either stand alone package units or in a modular concrete design. The package units are constructed of either 304 SST or FRP. Materials of construction for the internal components of both package and concrete units are SST and/or FRP. Filters are available in 40" standard bed or 80" deep-bed design depending on the nature of the application. Concrete modules are frequently used for high flow capacity systems by placing multiple modules into a common filter cell. The modules in a filter cell share a common filter bed where cones at the bottom of each module distribute sand to their respective airlifts and sand washers.

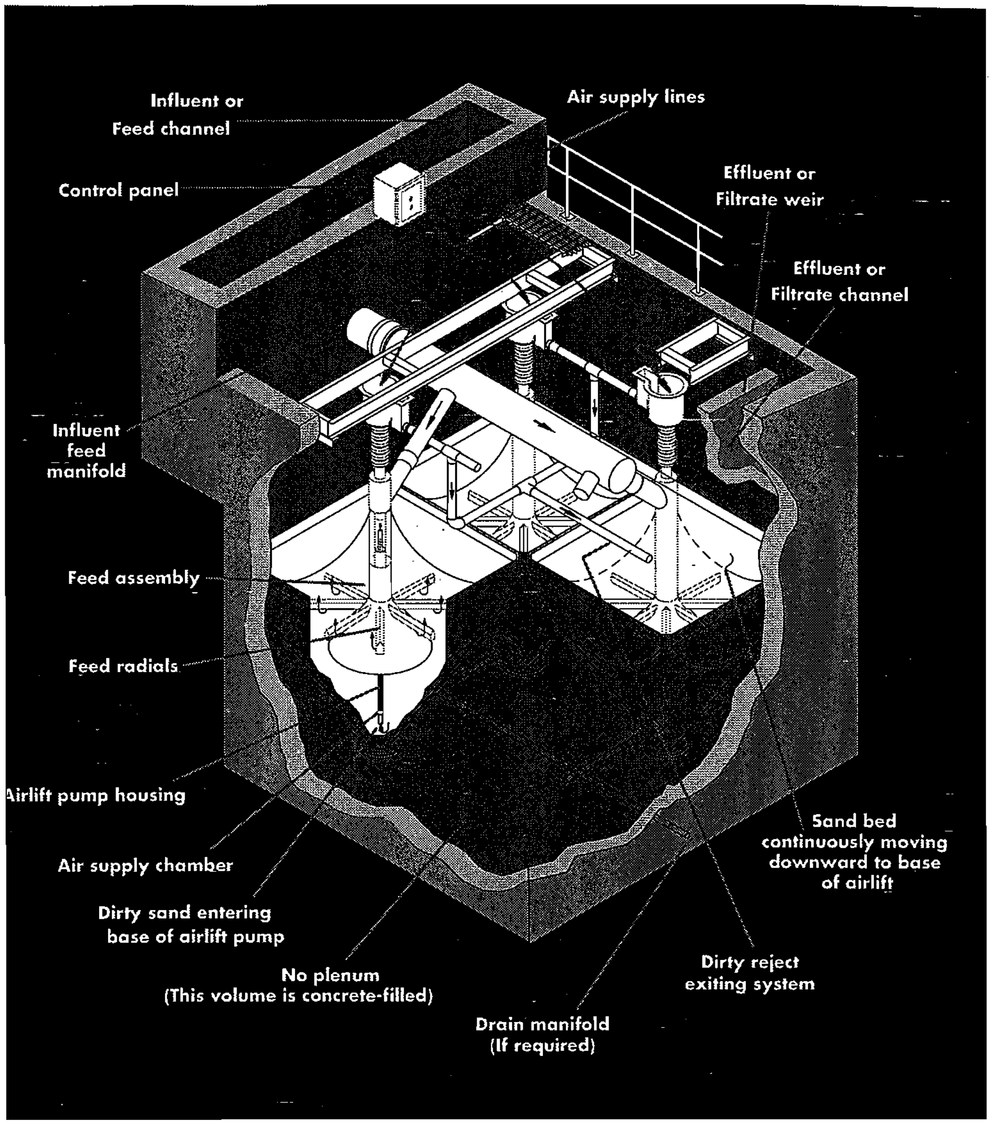


DynaSand Filter above ground package units

A concrete DynaSand installation can be designed for any size filter area. This enables the technology to be applied to any size water or wastewater treatment plant. Since all filter beds are being continuously cleaned, the pressure drop remains low and even throughout all the filters. Equal pressure drop ensures even distribution of feed to each filter without the need for splitter boxes or flow controls. Therefore, a typical multiple unit installation can use a common header pipe with feed connections and isolation valves for each filter.



DynaSand Filter modules in concrete basin

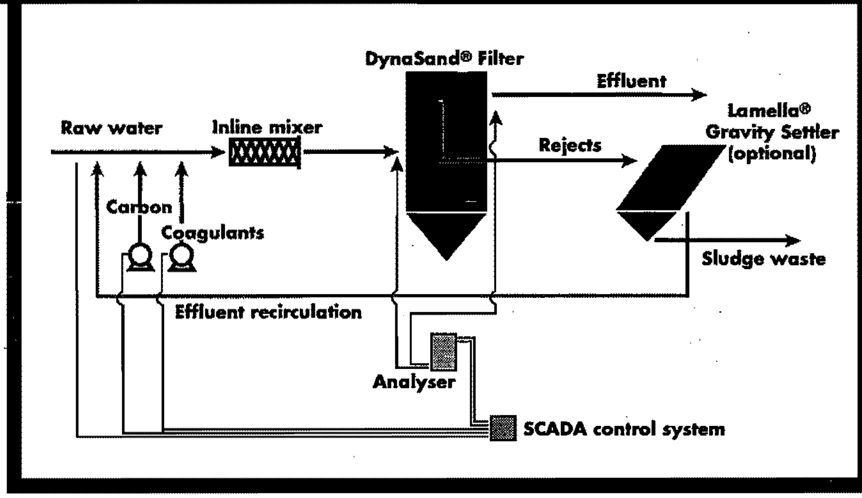


Features	Benefits
Continuously Cleaned Sand Bed	**No shutdown for backwash cycles** **Elimination of ancillary backwash equipment** **No flow control valves, splitter boxes, or backwash controls**
No Underdrains or Screens	**No short-circuiting**
Sand Washed with Filtrate	**Optimum sand-washing efficiency** **Superior filtrate quality**
No Level Control	**Reduced operator attention** **Minimizes overall pressure-drop**
Internal, Vertical Airlift	**Reduces potential for pluggage** **Significantly reduces wear/maintenance** **Can be easily maintained without filter shutdown**
Low Power Requirements	**Up to 70% less compressed air vs. other self-cleaning filters**

DynaSand® Filter Continuous Contact Filtration Process

Water and wastewater treatment in conventional plants typically involves flocculation, clarification and filtration. Direct filtration eliminates clarification but still requires flocculation. The DynaSand filter utilizes a proprietary process known as Continuous Contact Filtration. The DynaSand filter's 80" media bed depth provides greater hydraulic residence times and more opportunity for floc formation and attachment. Thus, coagulation, flocculation and separation can be performed within the sand bed, eliminating the need for external flocculators and clarifiers. Equipment savings can be substantial, up to 85% compared to conventional treatment and 50% compared to direct filtration. The DynaSand Continuous Contact Filtration process is better suited to remove small floc, which can help reduce chemical requirements by 20-30% over conventional treatment.

Applications The DynaSand filter is currently providing exceptional treatment in over 8,000 installations worldwide in a wide variety of applications.



DynaSand Applications – partial list

Tertiary filtration • Algae removal • Potable water (turbidity and color) • Oil removal • Process water • Brine filtration • Metal finishing • Cooling tower blowdown • Steel mill scale • Chemical processing • Phosphorus removal • Product recovery • Denitrification • Cryptosporidium and Giardia removal • Surface water • Ground water • Arsenic removal • Effluent reuse

Typical Data	Loading rate (gpm/ft²)	Influent solids	Filtrate solids
Tertiary Filtration	3-5	20-50 ppm SS	5-10 ppm SS
Potable Water – Turbidity	4-5	10-30 NTU	0.1-0.5 NTU
Potable Water – Color	4-5	10–120 ACU	1-5 ACU
Process Water	5	10-30 NTU	0.1-0.5 NTU
Metal Finishing	4-6	20-50 ppm SS	2-5 ppm SS
Steel Mill Scale	8-10	50-300 ppm SS	5-10 ppm SS
Phosphorus Removal	3-5	<1 ppm Total P	<0.1 ppm Total P
Algae Removal	2-4	100 ppm SS	10-20 ppm SS
Denitrification	3-4	10-15 ppm TN	<3 ppm TN
Oil Removal	2-6	<50 ppm O&G	5-10 ppm O&G

PARKSON CORPORATION

ISO 9001:2000 Certified
Quality Management System

www.parkson.com

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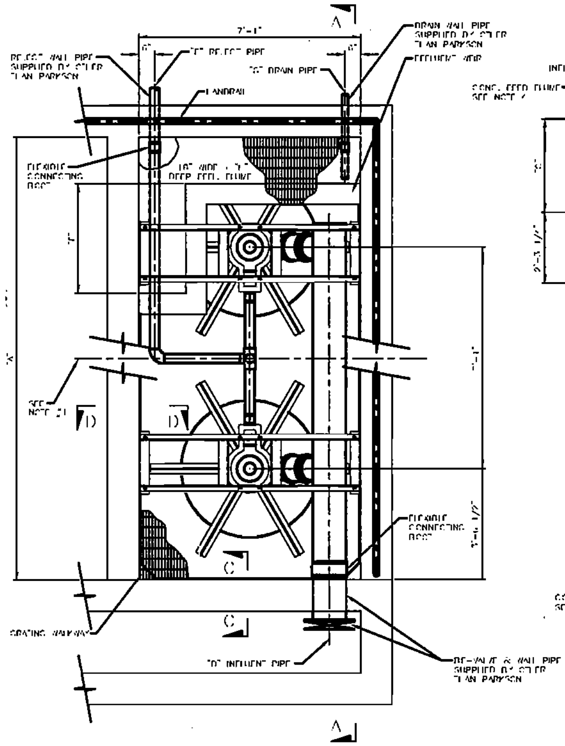
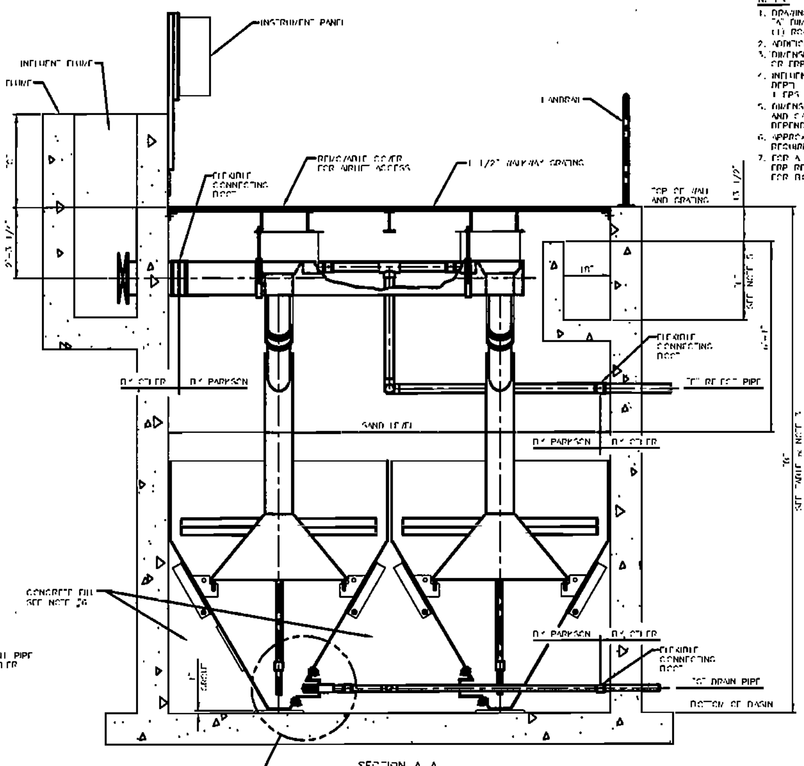
NOTES

1. DRAWING REPRESENTS (1) ROW (2) MODULES. "A" DIMENSION CAN BE LENGTHENED UP TO (1) ROW (6) MODULES.
2. ADDITIONAL CELLS MAY BE ADDED AS REQUIRED.
3. DIMENSION "B" IS BASED ON 1 1/2" HIGH ALUM. OR FRP GRATING.
4. INFLUENT FLUME SHOULD BE SIZED (WIDTH AND DEPTH BELOW GRATING LEVEL) FOR VELOCITY OF 1 FPS AT DESIGN FLOW.
5. DIMENSION "F" IS THE EFFLUENT FLUME DEPTH AND CAN VARY BETWEEN 2'-6" TO 5'-0" DEPENDING ON THE TOTAL NUMBER OF MODULES.
6. APPROXIMATELY 8.125 CU. YD. OF CONCRETE FILL REQUIRED PER MODULE.
7. FOR A (1) ROW (1) MODULE ARRANGEMENT, AN FRP REMOVABLE EFFLUENT TROUGH IS NECESSARY FOR BOTTOM CONE INSTALLATION.



PLAN VIEW



SECTION A-A

FOR SECTION AND DETAILS SEE DRAWING F00383

ROWS-MODULES		DIM. "A"	DIM. "B" STANDARD BED	DIM. "B" DEEP BED	DIM. "C" STANDARD BED	DIM. "C" DEEP BED	DIM. "D" INFL. PIPE	DIM. "E" REJECT PIPE	DIM. "F" EFFLUENT FLUME	DIM. "G" DRAIN PIPE
1	1	7'-1"	16'-2 1/2"	19'-6 1/2"	3'-0"	4'-0"	8"	3"	SEE NOTE #7	2"
1	2	14'-2"	16'-2 1/2"	19'-6 1/2"	3'-0"	4'-0"	10"	3"	--------	2"
1	3	21'-3"	16'-2 1/2"	19'-6 1/2"	3'-0"	4'-0"	12"	3"	--------	3"
1	4	28'-4"	16'-2 1/2"	19'-6 1/2"	3'-0"	4'-0"	14"	3"	--------	3"
1	5	35'-5"	16'-2 1/2"	19'-6 1/2"	3'-0"	4'-0"	16"	4"	4'-0"	3"
1	6	42'-6"	16'-2 1/2"	19'-6 1/2"	3'-0"	4'-0"	18"	4"	7'-0"	3"

LOADING CONDITIONS ON CONCRETE FLOOR
PER MODULE FOR STANDARD BED

22.0 KIPS OF DRY SAND
1.5 KIPS OF FILTER INTERNALS
24.3 KIPS OF WATER
33.0 KIPS OF CONCRETE FILL
TOTAL: 80.8 KIPS OR 1616#/SQ.FT.

LOADING CONDITIONS ON CONCRETE FLOOR
PER MODULE FOR DEEP BED

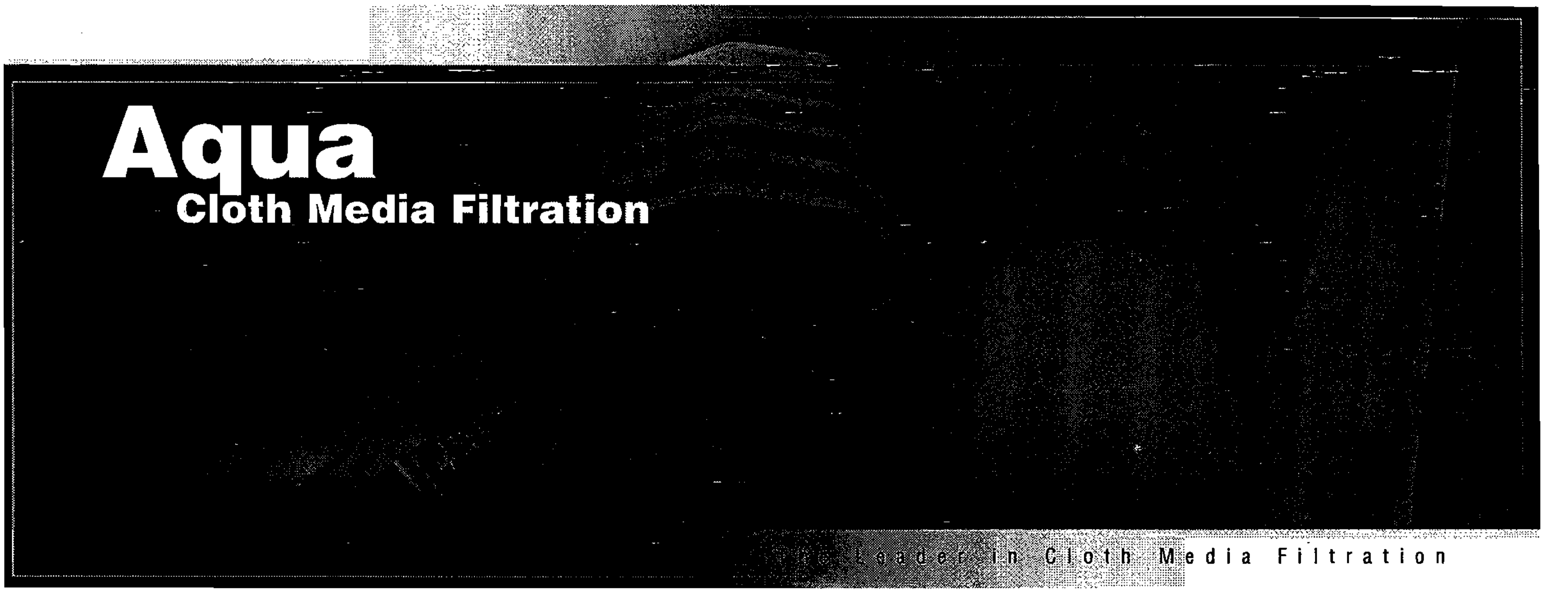
30.0 KIPS OF DRY SAND
1.5 KIPS OF FILTER INTERNALS
28.2 KIPS OF WATER
33.0 KIPS OF CONCRETE FILL
TOTAL: 100.7 KIPS OR 2014#/SQ.FT.

THE OWNER, PROJECT ENGINEER, AND ALL OTHERS INVOLVED WITH THE PROJECT DESIGN MUST IMPLEMENT AND FOLLOW ALL SAFETY STANDARDS REQUIRED BY ... WHEN INCORPORATING PARKSON CORPORATION EQUIPMENT INTO THE OVERALL PROJECT DESIGN. PARKSON CORPORATION WILL NOT BE RESPONSIBLE FOR LOCATION AND OR PLACEMENT OF EQUIPMENT IN THE PLANT DESIGN, NOR IS PARKSON RESPONSIBLE FOR ... SAFETY DESIGN AND ... OPERATION AND MAINTENANCE OF ... EQUIPMENT.

PARKSON CORPORATION

DynaSand® Filter
SALES DRAWING
TOP FEED
50 SF MODULES (1 ROW)

F00382 — A



Aqua-Aerobic Systems, Inc.

Cloth Media Leader

for Solutions

For over twenty five years, Aqua-Aerobic Systems, Inc. has been dedicated to maintaining a leadership role in the process of solid/liquid separation for the purification of water and wastewater.

Our success is justified by our reliable designs, application expertise, quality manufacturing and ongoing research and development. We pledge to continue to partner with our customers, providing solutions with innovative and proven technologies.

A product of our commitment to developing the best solutions for the needs of our customers is the unique media utilized in Aqua's family of cloth media filtration systems. These media have been carefully engineered for quality, durability and performance to provide several process and mechanical advantages compared to alternative filtration media. Aqua's cloth media has been adapted to a variety of mechanical configurations to maximize performance and value. A variety of cloth media are available to provide customized solid/liquid separation solutions for a broad range of municipal and industrial applications.

Advantages

- Unique cloth media
- Reuse quality effluent
- Low backwash rate
- Small footprint
- Low head requirements
- No downtime for backwashing
- Less maintenance than sand filters
- New plants or retrofits
- Lowest life-cycle cost

Applications

Municipal Reuse/Recycle

- 29.8 MGD Avg. Daily Flow
- AquaDisk® filters handle flows in excess of design while maintaining effluent quality.

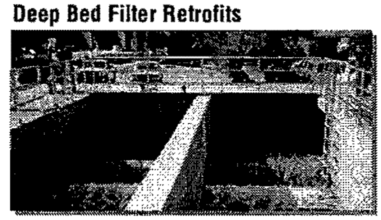
Phosphorus Removal

- 3 MGD Avg. Daily Flow
- AquaDisk® filter's small footprint and ability to expand without adding equipment are advantages with limited land space.

Traveling Bridge Filter Retrofits

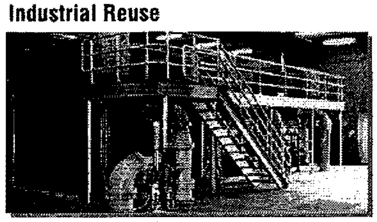
- 36 MGD Avg. Daily Flow
- AquaDiamond™ filter retrofitted into existing 16' sand filter bed and doubled the sand filter's maximum design hydraulic capacity.

Deep Bed Filter Retrofits



- 25 MGD Avg. Daily Flow
- AquaDisk® filter retrofitted into existing 16' deep bed filter eliminating the need for construction of new basins.

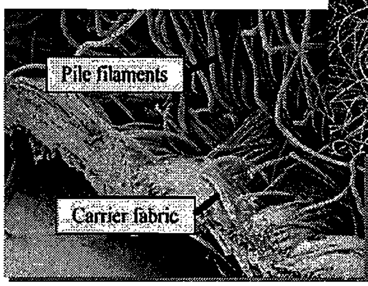
Industrial Reuse



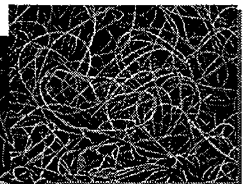
- 3 MGD Avg. Daily Flow
- AquaDisk® filter effluent is reused at a nearby power plant as cooling tower supply water.

Cloth Media The Key Component

Unique Cloth Media



Microscopic view of pile media.



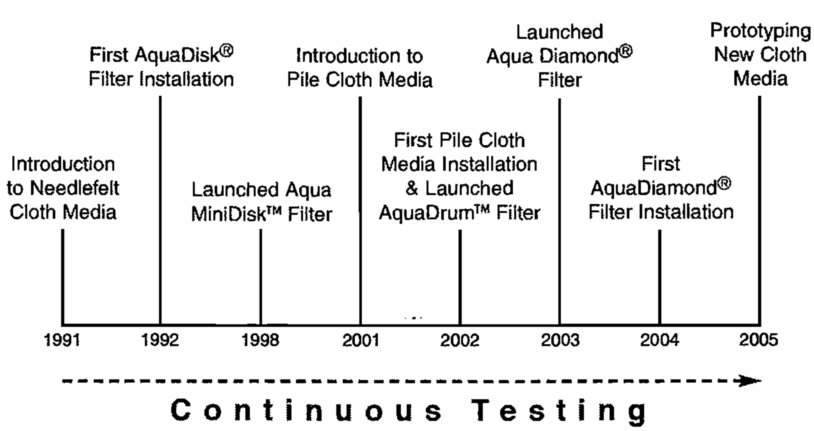
Microscopic view of needlefelt media.

Aqua's cloth media filtration systems utilize state-of-the-art cloth media. Only Aqua offers a variety of "true" cloth media, each with distinctive characteristics which can be custom-applied to your specific application. The depth of the media is inherent to the cloth's ability to consistently store and remove solid particles, resulting in optimal effluent quality.

Ongoing Commitment

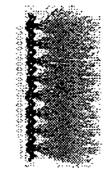
Aqua's proactive experience with research and development results in cloth media filtration products that virtually meet any tertiary requirements. We are dedicated to obtaining extensive knowledge on media, textile construction, durability, and impact on performance by working directly with textile manufacturers and independent testing laboratories. Our research efforts include continued development through partnerships with universities who test our products for durability and performance. Our commitment to research and development and piloting programs provides our customers with more media and configuration options to suite individual application needs.

Evolution of Aqua's Cloth Media Technology

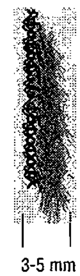


Pile Cloth Operation

Natural State



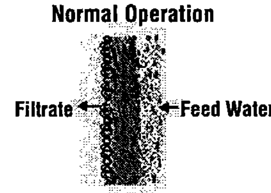
Normal Operation



Active Filter Depth

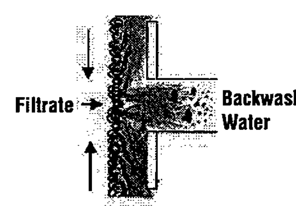
When wetted and mounted in a vertical configuration, densley packed fibers overlay one another, creating depth for the efficient removal and storage of solids.

Normal Operation



Solids retained on and within the cloth form an additional filter layer which provides enhanced filtration.

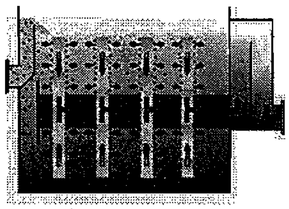
Backwash



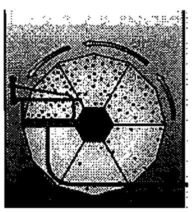
During backwash, filtrate is drawn back through the cloth. The suction causes the pile fibers to revert back to a natural state.

Cloth Media Configurations

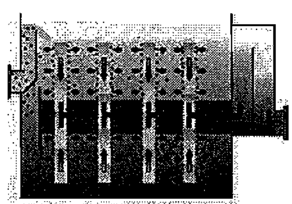
Operation



Inlet wastewater enters the tank or basin, completely submerging the cloth media. By gravity, liquid passes through the cloth media. As solids accumulate on and within the media, a mat is formed and the liquid level in the tank or basin increases. The filtered liquid enters the internal portion of the disk where it is directed to final discharge through the center shaft.



At a predetermined level or time, the backwash cycle will be initiated. Solids are backwashed from the surface by liquid suction from both sides of each disk. During backwash, disks are cleaned in multiples of two, unless a single disk unit is utilized. Disks rotate slowly, allowing each segment to be cleaned. Backwash water is directed to the headworks. Filtration is not interrupted during this cycle.



The filtration process requires no moving parts. Heavier solids are allowed to settle to the bottom portion of the filter tank. These solids are then pumped on an intermittent basis back to the headworks, digester or other solids collection area of the treatment plant.

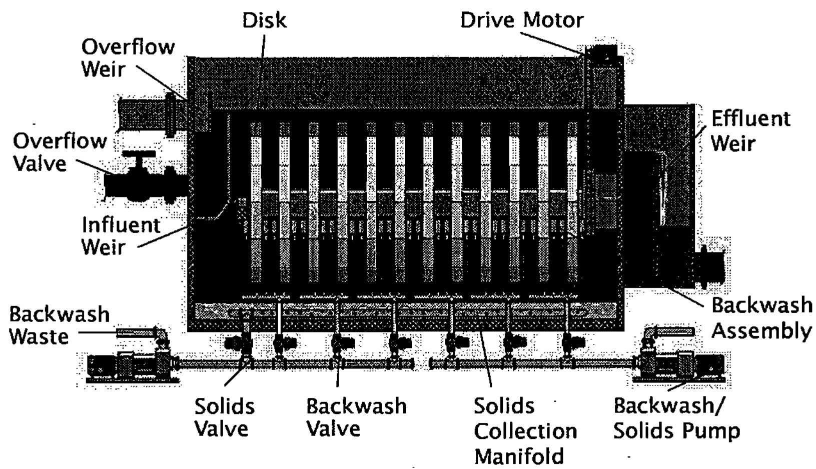
AquaDisk®

Aqua was first in the market, dating back to 1991, with the cloth media disk configuration as an alternative to conventional granular media filtration technologies. A history of exceptional operating experience and durability continue to make AquaDisk® the disk filter of choice.

Two AquaDisk® Filters with walkway access.

Features

- Up to 12 vertically oriented disks per unit
- Gravity flow operation
- Average hydraulic capacity from 0.25 to 3.0 MGD per unit
- Available in painted steel, stainless steel or concrete tanks
- Steel tank package units minimize field installation requirements
- Fully automatic, PLC based control system



Aqua MiniDisk™

The Aqua MiniDisk™ filter provides the solution for smaller flows. It is based on the same operating strategies as its larger counterpart, the AquaDisk®, but with smaller diameter disks.

Internal view of 4-disk Aqua MiniDisk™

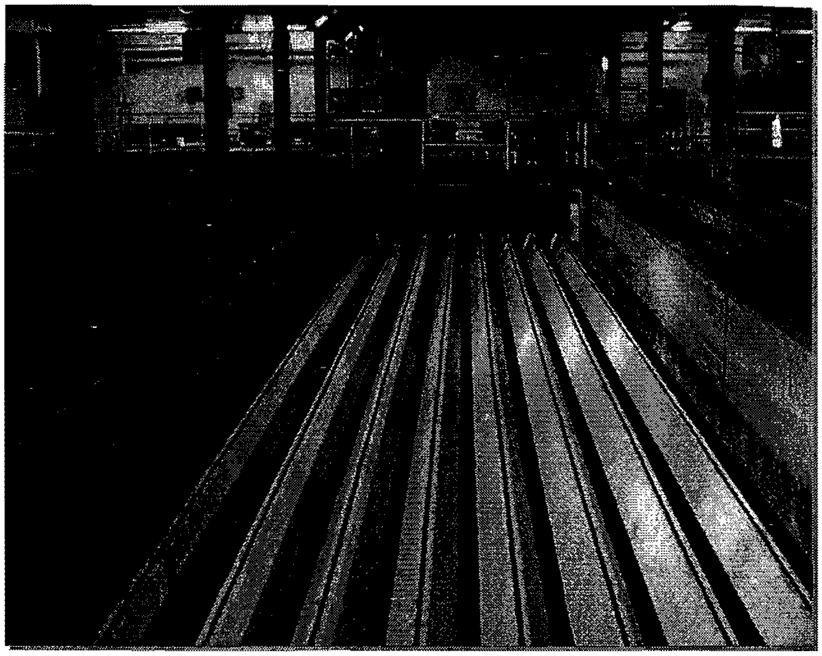
Features

- Up to 6 vertically oriented disks per unit
- Average hydraulic capacity from 50,000 to 300,000 GPD
- Available in painted steel or stainless steel tanks
- Gravity flow operation
- Steel tank packaged units minimize field installation requirements
- Fully automatic, PLC based control system

Cloth Media Configurations

AquaDiamond®

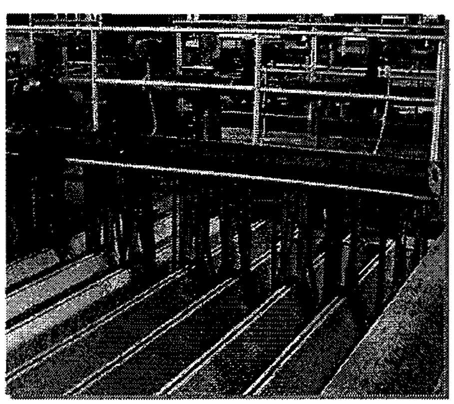
The AquaDiamond® is a unique combination of two time-proven technologies; traveling bridge and cloth media filtration. The result is three times the flow capacity of a traveling bridge filter with an equivalent footprint, making it ideal for new plants or sand filter retrofits.



Overview of AquaDiamond® filter retrofitted into a 16' wide sand filter cell.

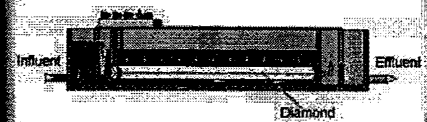
Features

- Up to 8 vertically oriented, diamond-shaped cloth media laterals per unit
- Gravity flow operation
- Available in concrete tanks
- Variable speed drive platform and backwash pump for immediate response to solids excursions
- Four-wheel drive platform designed for better guidance and traction
- Fully automatic, PLC based control system

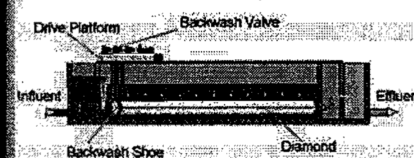


AquaDiamond® backwash assembly and laterals.

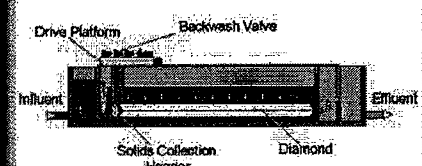
Operation



The cloth media is completely submerged during filtration. Solids are deposited on the outside of the cloth as the influent wastewater flows through. The filtered effluent is collected inside the diamond lateral and flows by gravity, to discharge. The filtration process requires no moving parts. Increased headloss due to the deposited solids automatically initiates periodic backwashing.



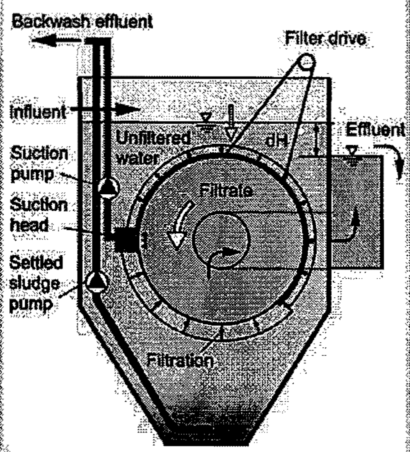
During backwash, a pump provides suction to the vacuum heads, allowing solids to be vacuumed from the cloth as the platform traverses the length of the diamond laterals. The platform operates only during backwashing and solids collection.



Because of the vertical orientation of the media, some solids will settle to the basin floor during normal operation. Small suction headers provide a means for collecting and discharging the settled solids. The solids collection process utilizes the backwash pump for suction.

Cloth Media Configurations

Operation

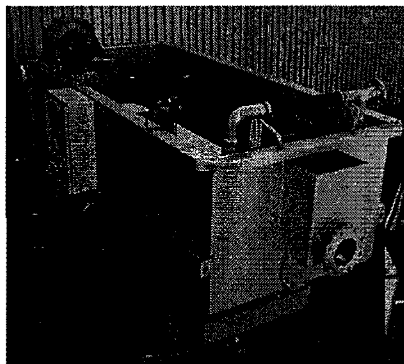


Solids are deposited on the outside of the cloth as the influent wastewater flows through. The filtered effluent is collected inside the drum and is discharged. Increased headloss due to the deposited solids automatically initiates periodic backwashing.

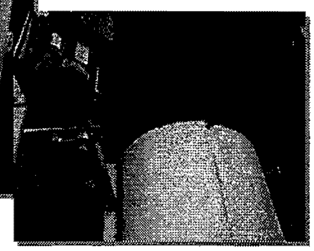
A pump provides suction to the vacuum head, allowing solids to be vacuumed from the cloth as the drum slowly rotates. Likewise, solids settling in the tank are suctioned and discharged. The drum only rotates during backwashing.

AquaDrum™

A drum style support structure covered with our unique cloth media is the basis of design for the AquaDrum™. It provides another small flow solution where driving head is particularly limited.



Overall view of an AquaDrum™ filter.



Internal view of AquaDrum™ filter.

Features

- One cloth media covered drum per unit
- Gravity flow operation
- Average hydraulic capacity from 60,000 to 375,000 GPD
- Available in stainless steel or concrete tanks

Technology Comparison

Of course, performance is not the only factor in choosing the right filter technology. Life-cycle cost plays an equally important role in the decision making process. Several other key factors should also be considered during the evaluation process.

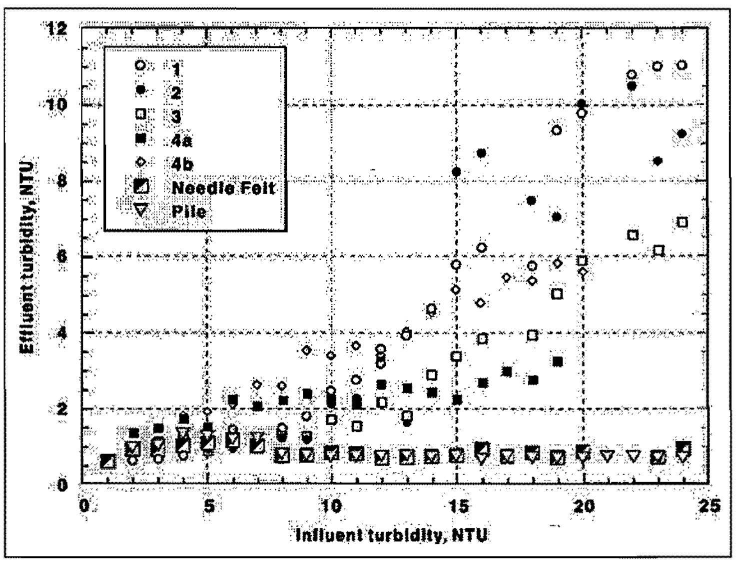
	Aqua-Aerobic Cloth Media	Granular Media	Micro Screens
Depth of Filtration	■	■	
High Solids Loading	■		
Small Footprint	■		■
Ease of Media Handling	■		■
Multiple Media Options	■	■	
Retrofits	■		■
Configuration options provided by a single manufacturer	■		◩

Cloth Media Performance

Documented Testing & Operating Data

The exceptional performance of Aqua's cloth media filtration technology has been fully documented through years of testing and gathering of operating data from full-scale installations.The table below resulted from independent testing and summarizes the performance of both our needlefelt and pile cloth media in comparison to other, more conventional wastewater filtration technologies. It shows that Aqua's unique cloth media produces consistently lower effluent turbidity values over a wider range of influent turbidities than the other technologies tested. This high standard of performance has been demonstrated on all of the cloth media mechanical configurations offered by Aqua-Aerobic.

This chart indicates the comparison of effluent versus influent turbidity for cloth media filtration at 14.7 m/hr and various filters at 9.8 m/hr.



- ○ Deep-bed, continuous backwash upflow mono-medium filters
- ● Shallow depth, automatic backwash mono, dual and multi-medium downward flow filters
- □ Deep-bed, mono-medium downward and/or upward filters
- ■ Shallow-depth, mono-medium filters
- ◇ Shallow-depth, dual medium filters
- ◩ Cloth Media Disk Filter (needlefelt media)
- ▽ Cloth Media Disk Filter (pile media)

Service Capabilities

Application and Engineering - Aqua has process, mechanical and electrical engineers on staff.

Laboratory Testing - Aqua can evaluate a sample of your wastewater and provide you with an analysis.

Piloting - Pilot filter units are available to evaluate effluent results for any application.

Aftermarket - Aqua offers parts sales and numerous service programs including: SpareCare, 24/7 Customer Service, Cloth Media Replacement and Rental and Lease options.

Operator Training - Aqua offers installation supervision and training to help you understand how your equipment/ system operates and and preventative maintenance that keeps your equipment operating efficiently.

Technical Seminars - Aqua provides a one-day Process and Product Application Seminar with Cloth Media Filtration as a main topic.

AquaDisk pilot unit

Aqua-Jet®
Surface Aerators

Aqua-Jet II®
Contained Flow Aerators

AquaDDM®
Direct Drive Mixer-Blenders

Aqua MixAir®
Aeration Systems

Aqua EnduraDisc®
Fine Bubble Diffusers

Aqua EnduraTube®
Fine Bubble Diffusers

Aqua CB-24®
Coarse Bubble Diffusers

AquaSBR®
Sequencing Batch Reactors

AquaExcel™
Batch Reactors with AquaEnsure™

AquaEnsure™
Maintenance-Free Decanter

Aqua MSBR®
Modified Sequencing Batch Reactor

AquaPASS™
Phased Activated Sludge Systems

AquaMB Process™
Multiple Barrier Membrane System

AquaDisk®
Cloth Media Filters

Aqua MiniDisk™
Cloth Media Filters

AquaDiamond®
Cloth Media Filters

AquaDrum™
Cloth Media Filters

AquaABF®
Automatic Backwash Filters

ThermoFlo®
Surface Spray Coolers

IntelliPRO™
Process Management System

Contact Your Local Representative:

Aqua-Aerobic Systems, Inc.
6306 N. Alpine Rd. • P.O. Box 2026 • Rockford, IL 61130
Phone: 815/654-2501 • Fax: 815/654-2508 • Toll Free: 877/214-9625
Email: solutions@aqua-aerobic.com • www.aqua-aerobic.com

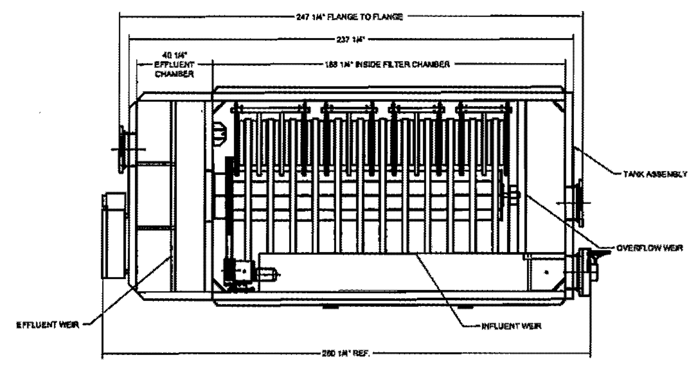
The information contained herein relative to data, dimensions and recommendations as to size, power and assembly are for purpose of estimation only. These values should not be assumed to be universally applicable to specific design problems. Particular designs, installations and plants may call for specific requirements. Consult Aqua-Aerobic Systems, Inc. for exact recommendations or specific needs.
Patents Apply. Patents Pending.

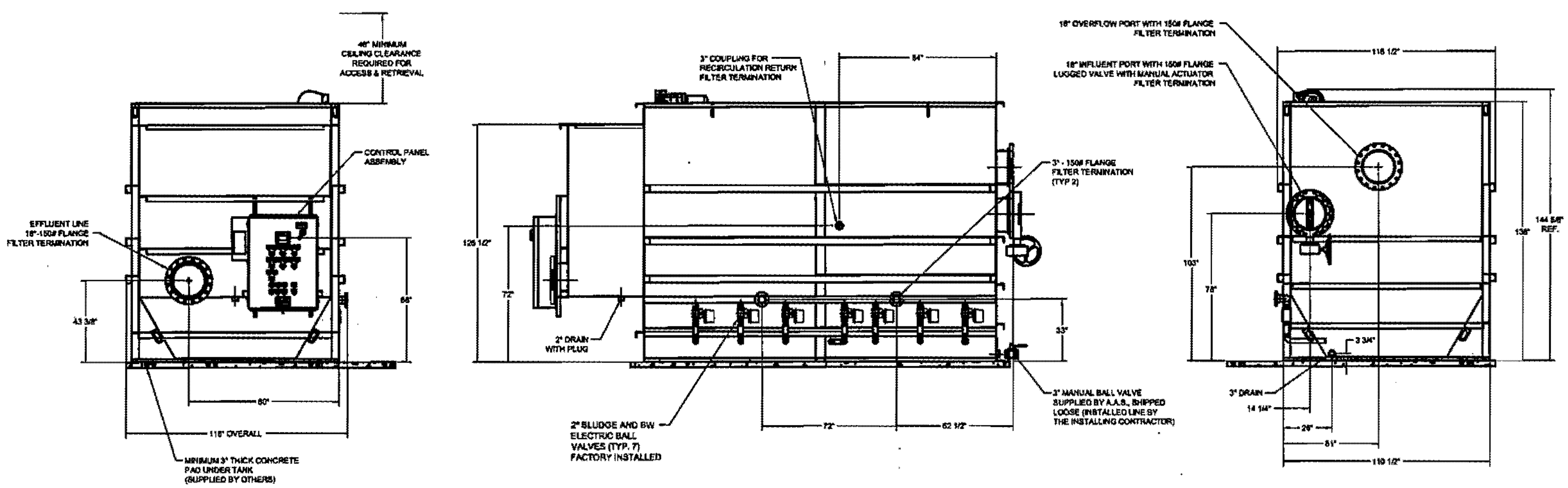
Bulletin #600G 07/05

NOTE:

1.) FREEZE NOTE: IF FREEZING IS A CONCERN, AQUA-AEROBIC SYSTEMS RECOMMENDS THE FILTERS BE PLACED IN A HEATED BUILDING. IF A BUILDING IS NOT PROVIDED, ANY NECESSARY PROTECTION, INCLUDING BUT NOT LIMITED TO, HEAT TRACING AND INSULATION OF PUMPS AND PIPING, AS WELL AS PROTECTION AGAINST INTERNAL TANK FREEZING, SHALL BE PROVIDED BY THE INSTALLING CONTRACTOR.

2.) THE GRAPHIC ELEMENTS OF THIS COMPUTER GENERATED DRAWING ARE DRAWN FULL SIZE. THE DIMENSIONS ARE ASSOCIATIVE. IF THE SIZE OF THE GRAPHIC ELEMENTS IS CHANGED THE DIMENSIONS WILL NOT BE CORRECT.



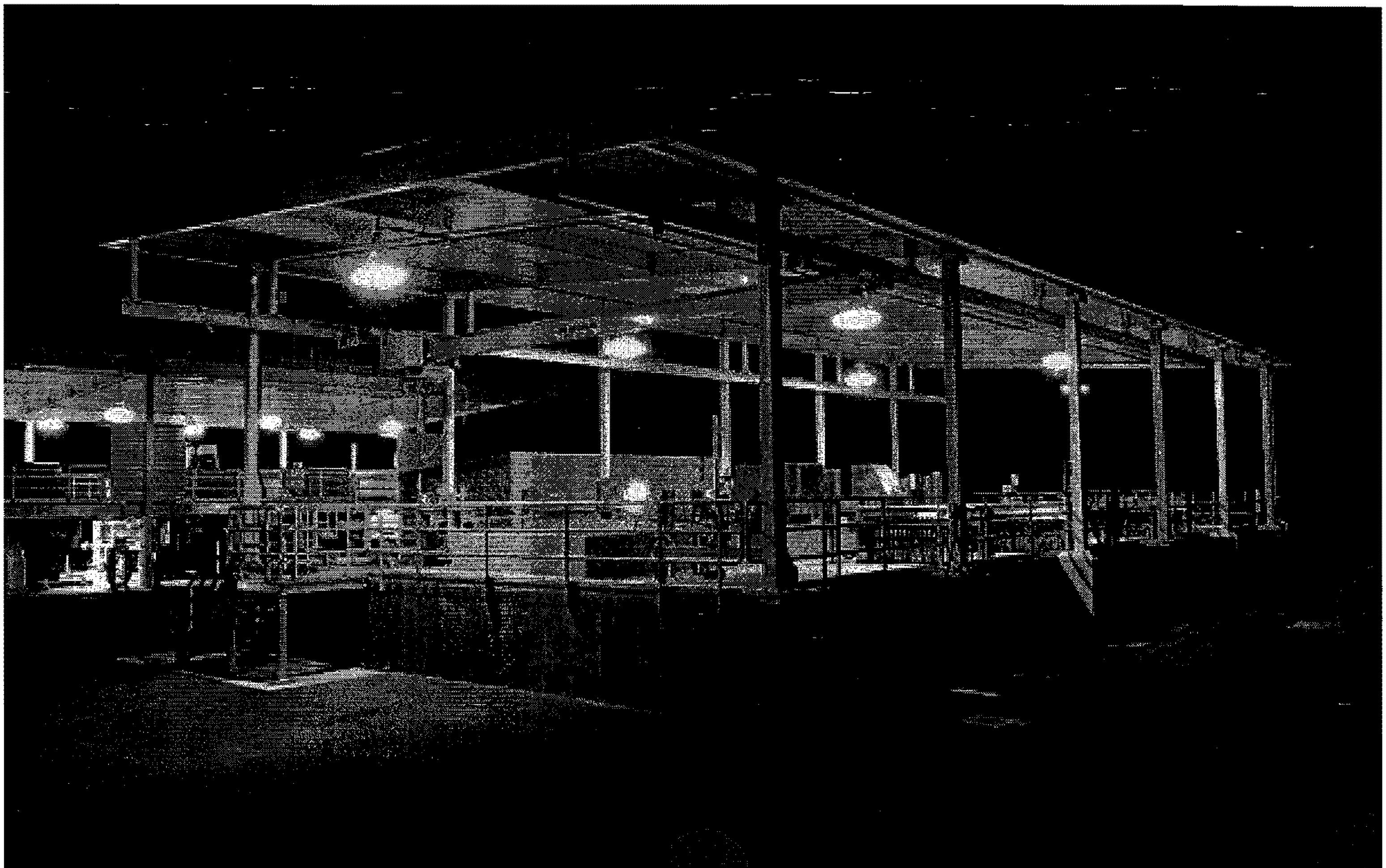


DRY WT (LBS)	OPER. WT. (LBS)
22,200	132,750

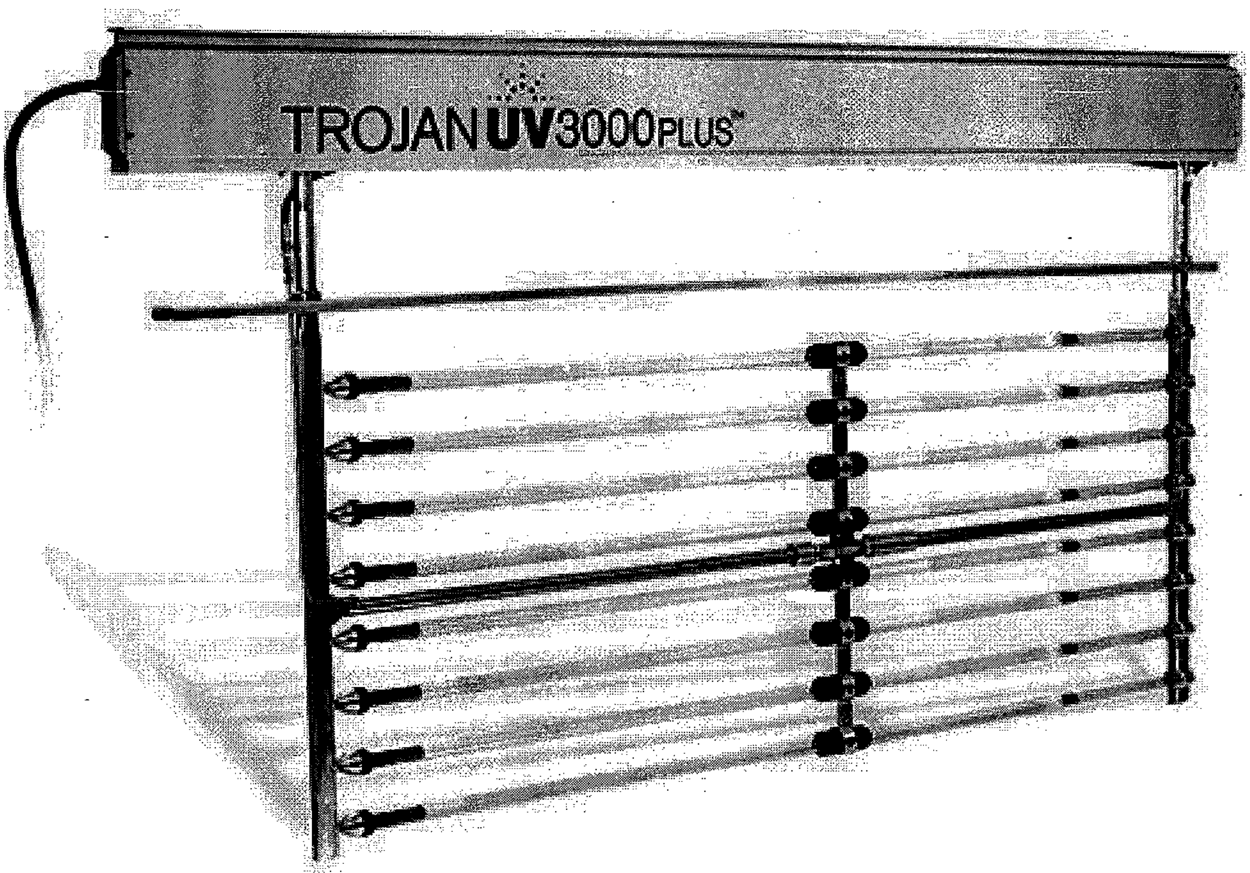
JOB NAME: LANCASTER WRP
JOB LOCATION: CA
DO NOT SCALE DRAWING
AQUA-AEROBIC SYSTEMS, INC.
AQUADISK FILTER
MODEL ADFP-54x126-PC
PILE CLOTH

TROJAN UV3000PLUS™

WASTEWATER DISINFECTION



TROJANUV3000PLUS™



The Reference Standard in UV

Proven, chemical-free disinfection from the industry leader

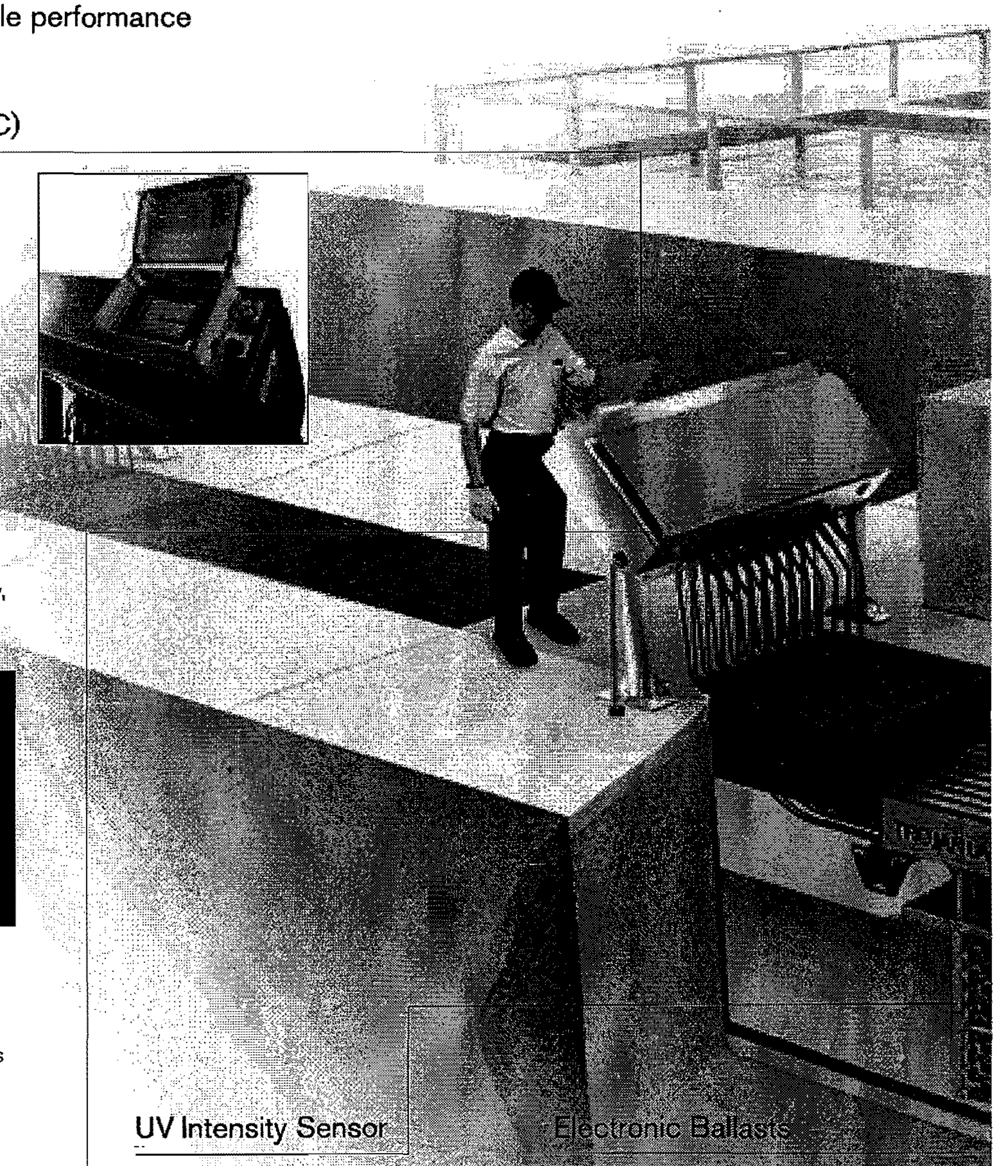
Trojan Technologies is an ISO 9001: 2000 registered company that has set the standard for proven UV technology and ongoing innovation for more than 25 years. With unmatched scientific and technical expertise, and a global network of water treatment specialists, representatives and technicians, Trojan is trusted more than any other firm as the best choice for municipal UV solutions. Trojan has the largest UV installation base – over 4,000 municipal installations worldwide – and almost one in five North American wastewater treatment plants rely on our proven, chemical-free disinfection solutions.

The TrojanUV3000Plus™ is one of the reasons why. This highly flexible system has demonstrated its effective, reliable performance around the world in over 400 installations. It is well suited to wastewater disinfection applications with a wide range of flow rates, including challenging effluent such as combined sewer overflows, primary and tertiary wastewater reclamation and reuse.

Following a review with Plant Operators and Engineers, the proven infrastructure of the TrojanUV3000Plus™ has been refined to make it even more operator-friendly. The result is more dependable performance, simplified maintenance, and maximized UV lamp output at end-of-lamp life. It also incorporates innovative features to reduce O&M costs, including variable output electronic ballasts and Trojan's revolutionary ActiClean™ system – the industry's only chemical/mechanical sleeve cleaning system.

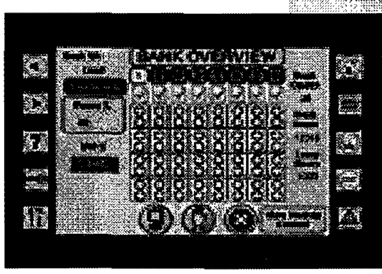
TROJAN UV3000PLUS™

Designed for efficient, reliable performance



System Control Center (SCC)

The SCC monitors and controls all UV functions, including dose pacing – the automatic, flow-based program that ensures proper disinfection levels while conserving power and extending lamp life. The microprocessor-based SCC is integrated onto one Power Distribution Center, and features a user-friendly, touch-screen HMI display with weatherproof cover, and Modbus Ethernet SCADA connectivity. For systems treating larger flows, or where more sophisticated control is desired, a PLC-based System Control Center is available. It features a separate wall-mount panel with colour, touch-screen HMI, Ethernet/IP SCADA connectivity, automatic slide/sluice gate control for multiple channels, and integrated Flash memory trend logging (flow, power, UVT, dose).



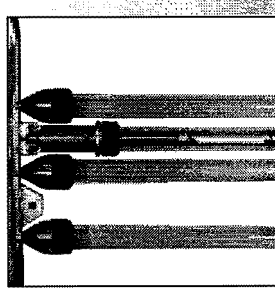
Alarms

Extensive alarm reporting system ensures fast, accurate diagnosing of system process and maintenance alarms. Programmable control software can generate unique alarms for individual applications.

Power Distribution Center (PDC)

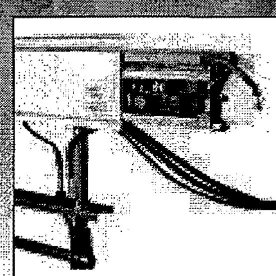
The PDC powers each bank of modules. Its ergonomic, angled design provides easy access to module power cables and hoses for the ActiClean™ cleaning system. The robust stainless steel enclosure is mounted across the channel, with module fuses and interlock relays visually aligned with module receptacles for fast diagnostics. Modules are individually overload protected for safety. Like all TrojanUV3000Plus™ components, the PDC can be installed outdoors and requires no shelter or HVAC.

UV Intensity Sensor



The UV intensity sensor continually monitors UV lamp output. The ActiClean™ system automatically cleans the sensor sleeve every time lamp sleeves are cleaned.

Electronic Ballasts



The variable-output (60 - 100% power) electronic ballast is mounted in its own TYPE 6P (IP67) rated enclosure within the module frame. Features "quick connect" electrical connections. Cooling is by convection.

ActiCleanTM Cleaning System

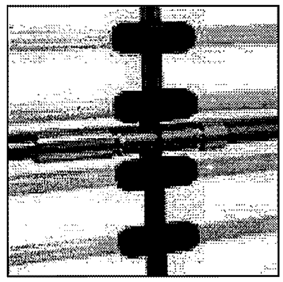
The system consists of two components:

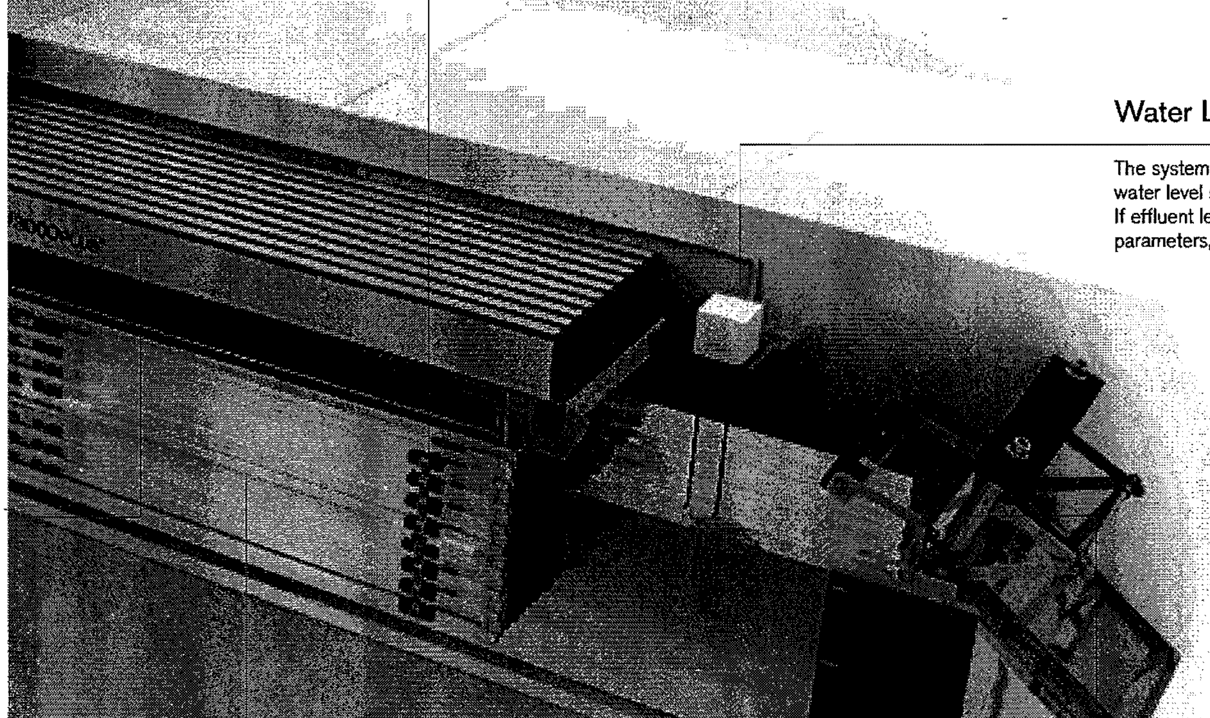
1. Hydraulic System Center (HSC)

The HSC actuates the ActiCleanTM cleaning system, and is mounted close to the channel in a stainless steel enclosure. It contains the pump, valves and ancillary equipment required to operate the cleaning system, and links to the extend/retract hoses of the module wiper drives via a manifold located on the underside of the PDC.

2. ActiCleanTM Wiper Assembly

A submersible wiper drive on each UV module drives the wiper carriage assembly along the module. Attached wiper canisters surround the quartz sleeves, and are filled with Trojan's ActiCleanTM Gel. The gel uses food grade ingredients and contacts the lamp sleeves between the two wiper seals. Cleaning takes place while the lamps are submerged and while they are operating.





Water Level Sensor

The system includes an electrode low water level sensor for each channel. If effluent levels fall below defined parameters, an alarm will be activated.

UV Modules

UV lamps are mounted on modules installed in open channels. The lamps are enclosed in quartz sleeves, and positioned horizontally and parallel to water flow. A bank is made up of multiple modules placed in parallel. All ballast and lamp wiring runs inside the module frame.

Water Level Controller

A fixed weir, motorized weir gate, or Automatic Level Control gate (shown), is required in the channel to maintain the appropriate water level over the lamps. Trojan engineers will work with you to select the appropriate level control device for your application.

Key Benefits
TrojanUV3000Plus™

Increased operator, community and environmental safety.
The TrojanUV3000Plus™ uses environmentally-friendly ultraviolet light – the safest alternative for wastewater disinfection. No disinfection by-products are created, and no chemicals must be transported, stored or handled.

Well suited to changing regulations. Trojan UV systems do not have any negative impact on receiving waters and do not produce disinfection by-products, making them a strategic, long-term choice as regulations become increasingly stringent.

Most efficient UV system available versus competitive low-pressure, high-output (LPHO) or amalgam lamp-based systems.

Reduces operating costs by as much as 30% per year. Long-lasting amalgam lamps and variable-output ballasts optimize UV output to meet wastewater conditions and maximize system efficiency versus competitive UV systems.

Proven disinfection based on actual dose delivery testing (bioassay validation), and over 400 TrojanUV3000Plus™ installations worldwide. Real-world, field performance data eliminates sizing assumptions resulting from theoretical dose calculations.

Dual-action sleeve cleaning system improves performance and reduces labor costs. Automatic ActiClean™ chemical/mechanical cleaning system maintains sleeve transmittance of at least 95%, and works online – eliminating the need to remove modules from the channel.

Reduced installation costs. The compact TrojanUV3000Plus™ can be retrofitted into existing chlorine contact tanks, and comes pre-tested, pre-assembled and pre-wired to minimize installation costs.

Outdoor installation flexibility. The entire TrojanUV3000Plus™ system can be installed outdoors, eliminating the need and costs of a building, shelter, and HVAC for ballast cooling.

Guaranteed performance and comprehensive warranty. Trojan systems include a Lifetime Performance Guarantee, the best lamp warranty in the industry, and use lamps from multiple approved suppliers. Ask for details.

ActiClean™ Dual-Action, Automatic Cleaning System

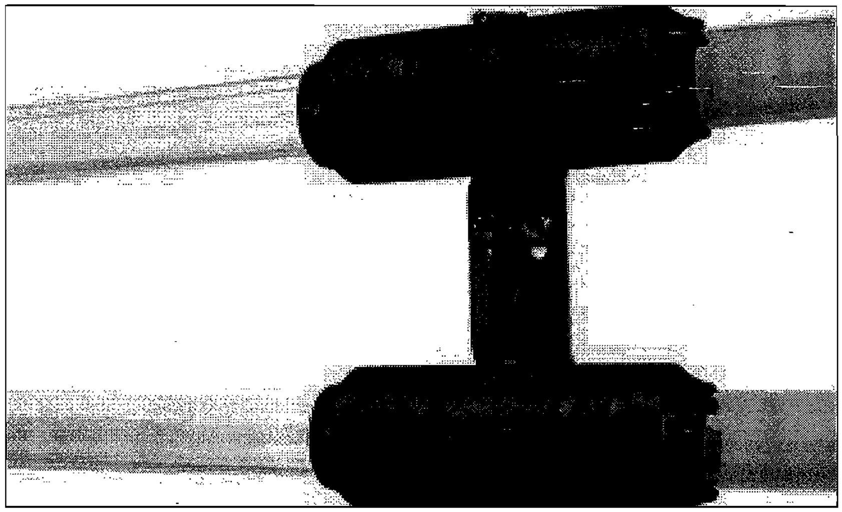
Chemical/mechanical cleaning system eliminates sleeve fouling

Benefits:

- Cleans 50% more effectively than mechanical wiping alone
- Improves lamp performance for more reliable dose delivery
- Elimination of fouling factor reduces equipment sizing requirements and power consumption
- Automatic, online cleaning reduces O&M costs associated with manual cleaning
- Combination of chemical and mechanical cleaning action removes deposits on quartz lamp and sensor sleeves much more effectively than mechanical wiping alone
- Innovative wiper design incorporates a small quantity of ActiClean™ Gel for superior, dual-action cleaning
- Cleans automatically while the lamps are disinfecting. There's no need to shut down the system, remove or bypass lamp modules for routine cleaning
- Proven in hundreds of systems around the world, including use in plants where heavy fouling had previously prohibited the use of UV disinfection technology
- ActiClean™ can be added to an installed TrojanUV3000Plus™ not originally equipped with a cleaning system

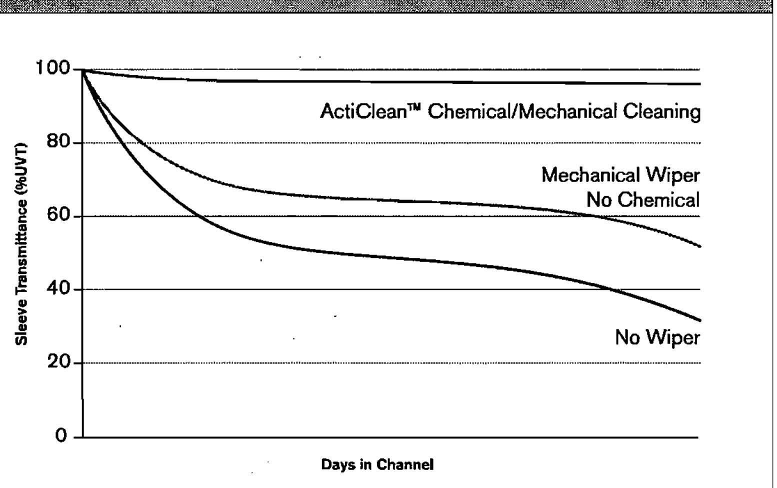
ActiClean™ Gel is Safe to Handle

- ActiClean™ Gel is comprised of food-grade ingredients
- Quick connect on cleaning system allows for easy refill of gel solution
- Lubricating action of ActiClean™ Gel maximizes life of wiper seals



The dual-action, chemical/mechanical cleaning with the ActiClean™ system provides superior sleeve cleaning and reduces maintenance costs. Fouling and residue build-up on quartz sleeves reduces system efficiency. ActiClean™ maintains at least 95% transmittance, ensuring sleeves are clean and the system is consistently delivering accurate dosing while reducing power consumption.

Efficacy of Cleaning Technologies to Control Sleeve Fouling

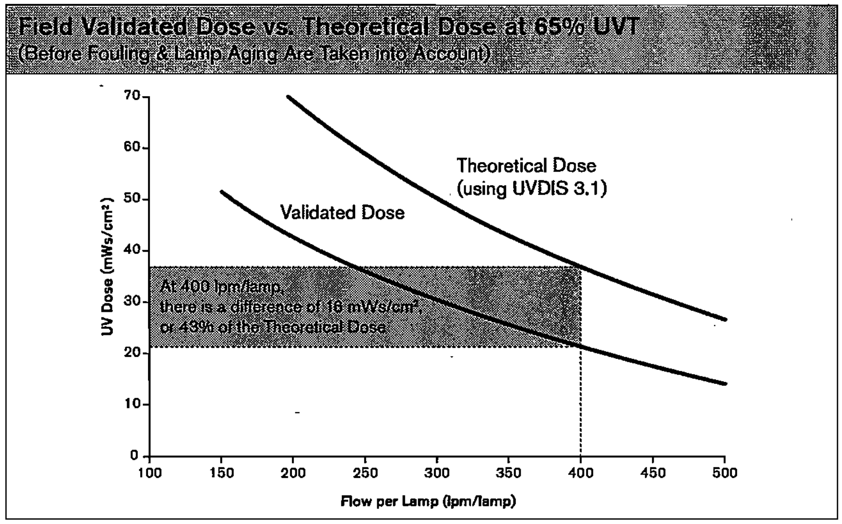


Regulatory-Endorsed Bioassay Validation

Real-world testing ensures accurate dose delivery

Benefits:

- Performance data is generated from actual field testing over a range of flow rates, effluent quality, and UVTs
- Provides physical verification that system will perform as expected; ensures public and environmental safety
- Provides accurate assessment of equipment sizing needs
- The TrojanUV3000Plus™ has been thoroughly validated through real-world bioassay testing under a wide range of operating conditions
- In-field bioassay testing offers the peace of mind and improved public and environmental safety of verified dose delivery – not theoretical calculations
- The USEPA has endorsed bioassays as the standard for assessment and comparison of UV technologies
- The disinfection performance ratings for the TrojanUV3000Plus™ are proof that what you see is what you actually get



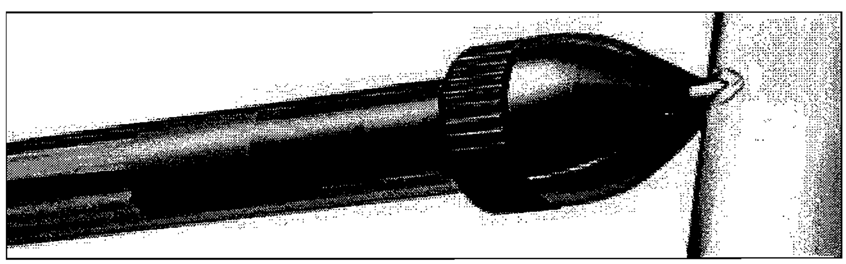
This shows the validated dose of an actual working system and the theoretical dose calculated using UVDIS. Note that the UVDIS 3.1 dose calculation overestimates the system performance.

Amalgam Lamps Require Less Energy

Require fewer lamps and reduce O&M costs

Benefits:

- Draw less energy than competitive high-output systems – only 250 Watts per lamp
- Stable UV output over a wide range of water temperatures
- Fewer lamps are required to deliver the required dose, which reduces O&M costs
- Can treat lower quality wastewater such as primary effluents, combined sewer overflows, and storm water
- Fewer lamps allow systems to be located in compact spaces, reducing installation costs



Trojan's high efficiency amalgam lamps generate stable UV output in a wide range of water temperatures.

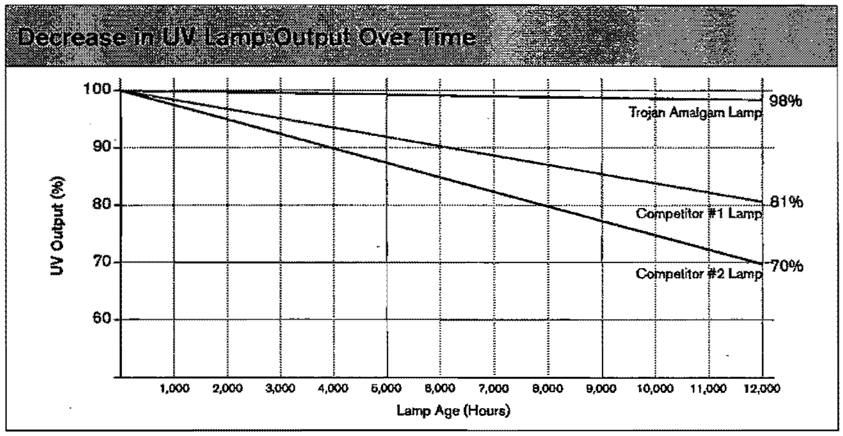
- Trojan's amalgam lamps produce significantly higher UV output than conventional low-output lamps
- Fast and simple lamp changeouts; replacing a 50-lamp system takes less than two hours and requires no tools
- The lamps are sealed inside heavy-duty quartz sleeves by Trojan's multi-seal system, maintaining a watertight barrier around the internal wiring while individually isolating each lamp and the module frame
- Lamps are pre-heated for reliable startup

Amalgam Lamps Maintain Maximum UV Output

Trojan lamps deliver 98% of full UV output after more than one year of use

Benefits:

- Trojan's high efficiency, amalgam lamps deliver the most consistent UV output
- Trojan lamps have 20% less decline in UV output after 12,000 hours of use compared to competitive UV lamps
- Validated performance assures you of reliable dose delivery and prolonged lamp life



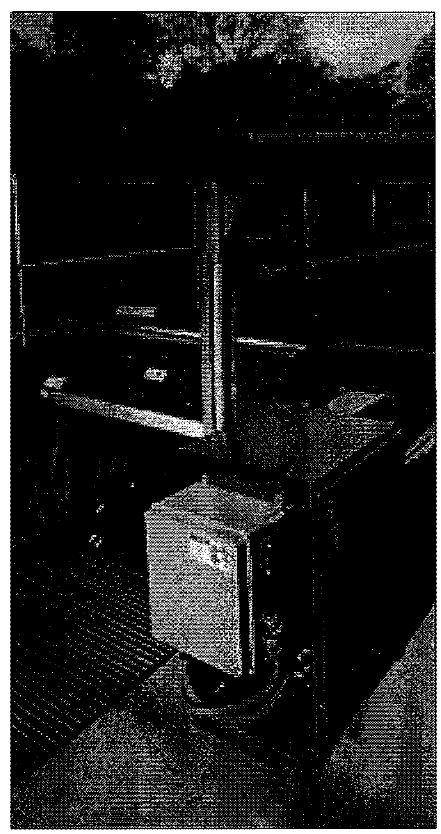
The lamps used on the TrojanUV3000Plus™ system have been independently validated to maintain 98% of original output after 12,000 hours of operation.

Open-Channel Architecture Designed for Outdoor Installation

Cost-effective to install and expand

Benefits:

- Compact, open-channel design allows cost-effective installation in existing effluent channels and chlorine contact chambers
- System can be installed outdoors to reduce capital costs – no building, shelter or HVAC is required
- Gravity-fed design eliminates costs of pressurized vessels, piping and pumps
- Scalable architecture allows precise sizing – reduces capital and O&M costs associated with oversizing
- Modular design is readily expandable to meet new regulatory or capacity requirements
- Trojan's thorough design approach ensures that effluent quality, upstream treatment processes, and O&M needs are addressed in system configurations
- Horizontal lamp mounting delivers optimal hydraulic performance. This arrangement induces turbulence and dispersion, maximizing wastewater exposure to UV output



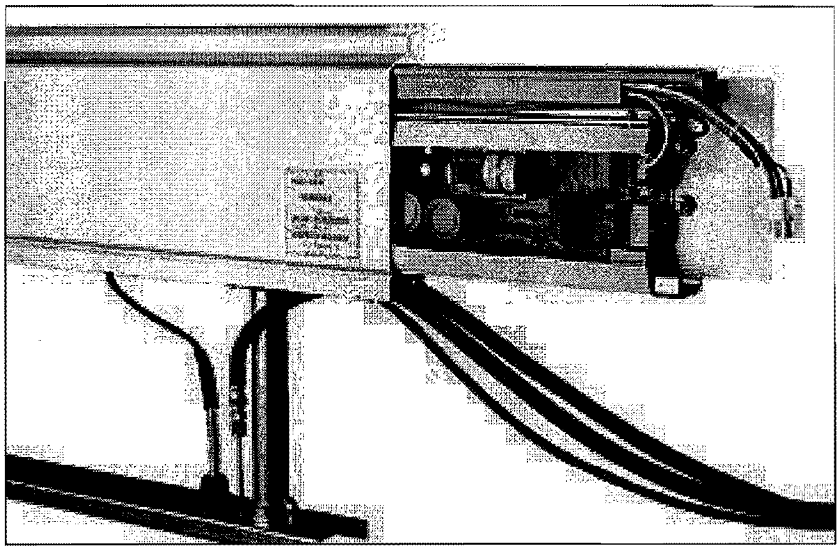
The TrojanUV3000Plus™ system delivers flexibility and cost savings through its simple installation in existing channels and chlorine contact chambers. The system can be situated outdoors with no additional building, shelter or cooling requirements.

Advanced, Self-Contained UV Module

Dramatically reduces footprint size and eliminates costs of air conditioning

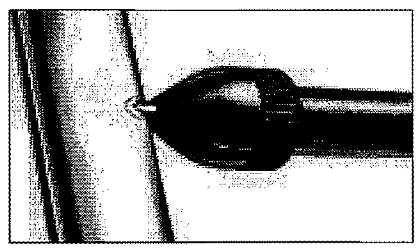
Benefits:

- Lamps are protected in a fully submersible, 316 stainless steel frame
- Waterproof module frame protects cables from effluent, fouling and UV light
- Electronic ballasts are housed right in the module, reducing the system footprint, minimizing installation time and costs, and eliminating the need for separate external cabinets
- Ballast enclosures are rated TYPE 6P (IP67) – air/water tight
- Module leg and lamp connector have a hydrodynamic profile to reduce headloss
- The variable-output, electronic ballast is mounted in an enclosure integrated within the module frame
- Wiring is pre-installed and factory-tested



Module-mounted ballasts allow for compact installation, convection cooling, and protect wires and cables from exposure to effluent and UV light.

- Cooling ballasts by convection eliminates costs associated with air conditioning and forced-air cooling

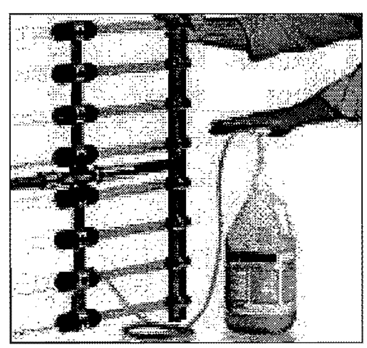


Module leg and lamp connector have a hydrodynamic profile to reduce headloss and potential for debris fouling.

Designed for Easy Maintenance

Trojan UV lamps are easily replaced in minutes without the need for tools.

- TrojanUV3000Plus™ lamps are warranted for 12,000 hours
- Modular design allows for maintenance on one module without disrupting disinfection performance
- Maintenance limited to replacing lamps and cleaning solution
- Automated ActiClean™ cleaning system reduces manual labor associated with cleaning sleeves



Quick connect allows for easy refill of ActiClean™ Gel.

TROJANUV3000PLUS™

System Specifications	
[illegible]	TrojanUV3000Plus
Typical Applications	Wide range of wastewater treatment plants
Lamp Type	High-efficiency Amalgam
Ballast Type	Electronic, variable output (60 to 100% power)
Input Power Per Lamp	250 Watts
Lamp Configuration	Horizontal, parallel flow
Module Configuration	4, 6 or 8 lamps per module
Level Control Device Options	ALC, fixed weir or motorized weir gate
Water Level Sensor	1 electrode low water level sensor per channel
Enclosure Ratings	
Module Frame / Ballast Enclosure	TYPE 6P (IP68) / TYPE 6P (IP67)
All Other Enclosures	TYPE 4X (IP56)
Ballast Cooling Method	Convection, no air conditioning or forced air required
Installation Location	Indoor or outdoor
Sleeve Cleaning System	
ActiClean™ Cleaning System	Optional Automatic Chemical/Mechanical Cleaning System
ActiClean™ Cleaning Gel	Non-corrosive, operator-friendly
Recommended Fouling Factor	1.0
System Control Center	
Controller	Microprocessor or PLC-based
Analog Inputs (Typical)	Flow (4-20 mA) and UVT (4-20 mA)
Discrete Outputs (Typical)	Bank status, common alarms and SCADA communication
Maximum Distance from UV Channel	500 ft. (152 m)
Electrical Requirements	
Power Distribution Center	208Y/120V, 3 phase, 4 wire + GND, 60 Hz (Max 8 modules per PDC) 480Y/277V, 3 phase, 4 wire + GND, 60 Hz 380Y/220V, 3 phase, 4 wire + GND, 50/60 Hz 400Y/230V, 3 phase, 4 wire + GND, 50/60 Hz 415Y/240V, 3 phase, 4 wire + GND, 50/60 Hz
System Control Center (stand alone)	120V, single phase, 2 wire + GND, 60 Hz, 1.8 kVA 220/230/240V, single phase, 2 wire + GND, 50/60 Hz, 1.8kVA
Hydraulic System Center (for ActiClean™)	208V, 3 phase, 3 wire + GND, 60 Hz 380/400/415 V, 3 phase, 3 wire + GND, 50/60 Hz 480V, 3 phase, 3 wire + GND, 60 Hz or 2.5kVA HSC powered from PDC
Water Level Sensor	24VDC powered from PDC

Find out how your wastewater treatment plant can benefit from the TrojanUV3000Plus™ – call us today.

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Trojan Technologies (France): +33 1 6081 0516
Trojan Technologies Espana (Spain): +34 91 564 5757
Trojan Technologies Deutschland GmbH (Germany): +49 6024 634 75 80
Hach/Trojan Technologies (China): 86-10-65150290

Products in this brochure may be covered by one or more of the following patents:
U.S. 4,872,980; 5,006,244; 5,418,370; RE 36,896; 6,342,188; 6,635,613; 6,646,269; 6,663,318; 6,719,491; 6,830,697; 7,018,975
Can. 1,327,877; 2,117,040; 2,239,925
Other patents pending.

MWW-003 (0107) TROW-1040

TROJANUV
WATER CONFIDENCE™

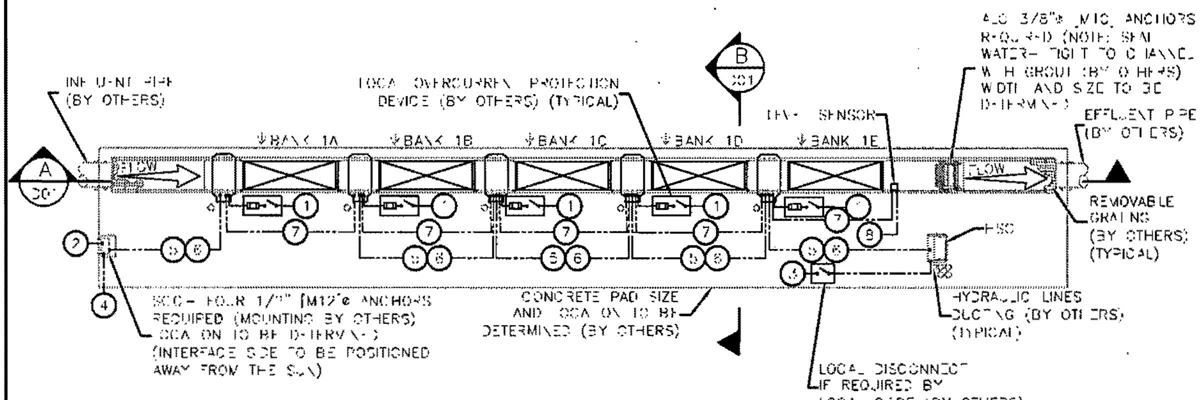
TROJAN UV3000™ PLUS

EQUIPMENT INTERCONNECTIONS

No.	DESCRIPTION	FROM	TO
1	POWER DISTRIBUTION CENTER (PDC) POWER SUPPLY 480Y/277V, 3 PHASE, 4 WIRE + GROUND XX KVA/PDC POWER DRAW XX AMPS MAXIMUM CURRENT/PHASE	DISTRIBUTION PANEL (DP) (BY OTHERS) (NOT SHOWN)	PDC
2	SYSTEM CONTROL CENTER (SCC) POWER SUPPLY 120V, 1 PHASE, 2 WIRE + GROUND, XX AMPS	DP (BY OTHERS) (NOT SHOWN)	SCC
3	HYDRAULIC SYSTEM CENTER (HSC) POWER SUPPLY 480V, 3 PHASE, 3 WIRE + GROUND, 20 AMPS	DP (BY OTHERS) (NOT SHOWN)	HSC
4	FLOW METER 4-20 mA, DC ANALOG INPUT (BY OTHERS)	FLOW METER PANEL (NOT SHOWN)	SCC
5	GROUND LINK 14 AWG TYPE TW- STRANDED	SCC	HSC AND PDC's (DAISY CHAINED)
6	SERIAL COMMUNICATION LINK (RS485) 1 SHIELDED TWISTED PAIR (24 AWG)	SCC	HSC AND PDC's (DAISY CHAINED)
7	LOW WATER LEVEL DETECTION 2- 14 AWG TYPE TW- STRANDED	LEVEL SENSOR	PDC (DAISY CHAINED)
8	LEVEL SENSOR POWER SUPPLY 120V, 1 PHASE, 2 WIRE + GROUND, 5 AMPS	DP (NOT SHOWN) (BY OTHERS)	LEVEL SENSOR

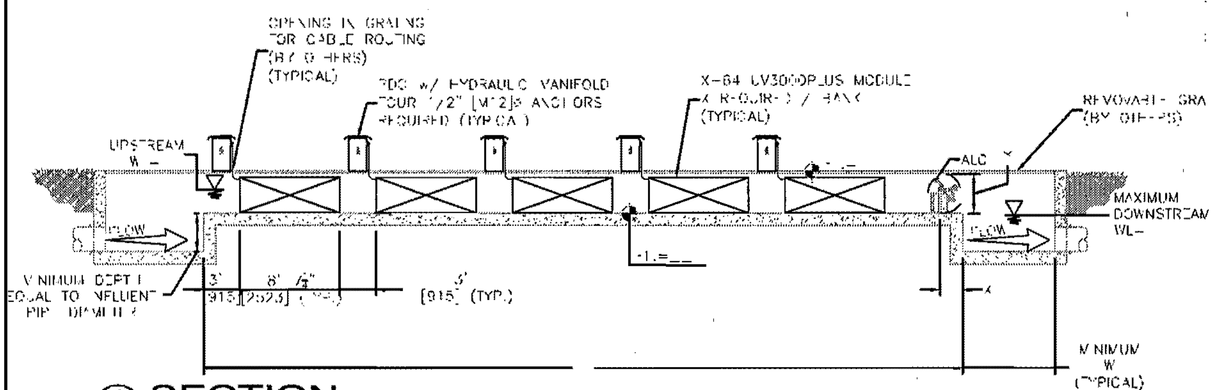
NOTES:

: DO NOT SLOPE CHANNEL FLOOR.
: CHANNEL WIDTH & DEPTH MUST BE KEPT WITHIN A TOLERANCE OF ... OR 1/4" [6mm].
: ANCHOR BOLTS ARE NOT SUPPLIED BY TROJAN TECHNOLOGIES INC.
: SYSTEM CONDUIT, WIRING, DISTRIBUTION PANELS & INTERCONNECTIONS BY OTHERS.
: ELECTRICAL REQUIREMENTS SHOWN ARE TO SUPPLY TROJAN UV EQUIPMENT ONLY. ELECTRICAL INRUSH FACTOR TO BE ADDED AS PER LOCAL CODE.
: REMOVABLE GRATING SECTIONS SHALL BE EASILY REMOVED BY ONE PERSON. MAXIMUM WEIGHT OF THE SECTIONS SHALL BE IN ACCORDANCE WITH REQUIREMENTS OF THE APPLICABLE JURISDICTION.
: CONTRACTOR TO REVIEW ALL TROJAN TECHNOLOGIES INC. INSTALLATION INSTRUCTIONS PRIOR TO EQUIPMENT INSTALLATION.
: TOLERANCE AT ALC IS CHANNEL WIDTH ... [25mm].



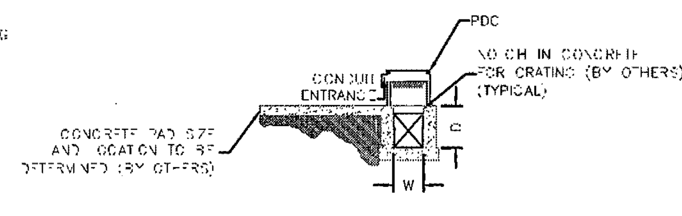
PLAN VIEW

SCALE: NOT TO SCALE



(A/D01) SECTION

SCALE: NOT TO SCALE



(B/D01) SECTION

SCALE: NOT TO SCALE

NOTE: SCC AND GRATING NOT SHOWN FOR CLARITY

DESIGN CRITERIA		
	PEAK FLOW	MGD / m³ / DAY
	UV TRANSMITTANCE AT 253.7 nm	%
	SUSPENDED SOLIDS	mg / l
	DISINFECTION STANDARD	FC / 100ml

TROJAN UV

DESCRIPTION: LAYOUT, UV3000PLUS 1 CHANNEL 5 BANKS ALC WITH ACTICLEAN		STANDARD DRAWING NO. 3M0549	
DRAWN BY: CMS/LAK	DATE: 05FE01	REFERENCE NO. N/A	
CHECKED BY: CMS	DATE: 05FE04		
APPROVED BY: CAR	DATE: 05FE04	DWG. NO. D01	REV. B
SCALE (8½x11): NOT TO SCALE	LOG NUMBER: N/A		