Lake Hancock
Outfall Treatment Project (H014)

Technical Memorandum
Alternative Treatment Technologies Evaluations

FINAL

Prepared for

Southwest Florida Water Management District

Prepared by

PARSONS WATER & INFRASTRUCTURE INC.

In Association With
Wetland Solutions, Inc.
Environmental Research & Design, Inc.

PARSONS

August 2007

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TECHNICAL MEMORANDUM ALTERNATIVE TREATMENT TECHNOLOGIES EVALUATIONS

FINAL

Prepared for:

Southwest Florida Water Management District
August 2007

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This memorandum submitted to the Southwest Florida Water Management District, entitled: Lake Hancock Outfall Treatment Project (H014) TECHNICAL MEMORANDUM ALTERNATIVE TREATMENT TECHNOLOGIES EVALUATIONS-FINAL was prepared under the supervision and direction of the undersigned, who are licensed professional engineers in the State of Florida.

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TABLE OF CONTENTS

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TABLE OF CONTENTS

APPENDICES

LIST OF TABLES

LIST OF TABLES – Cont'd

LIST OF TABLES – Cont'd

LIST OF TABLES – Cont'd

LIST OF FIGURES

LIST OF FIGURES – Cont'd

LIST OF FIGURES – Cont'd

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SECTION 1

INTRODUCTION

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SECTION 1.0
INTRODUCTION

1.1 BACKGROUND

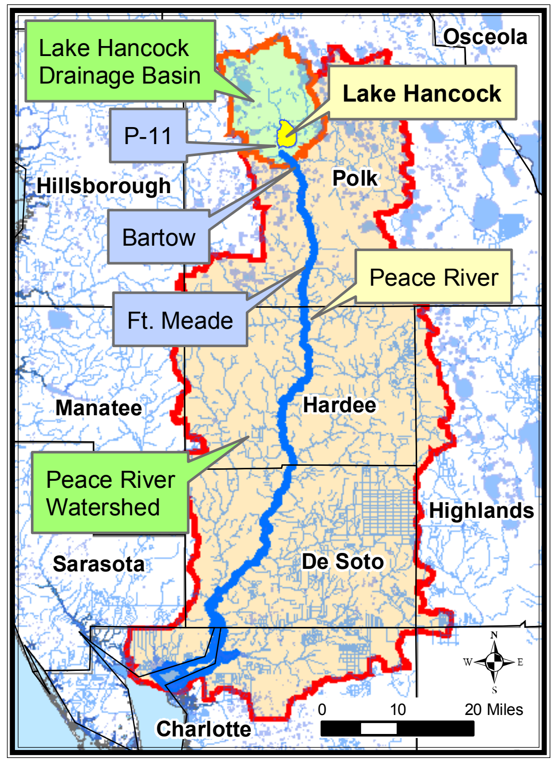

Figure 1.1-1 Charlotte Harbor and Peace River Watershed

The Southwest Florida Water Management District has identified the Lake Hancock Outfall Treatment Project as one of several projects that are critical in the District's strategies for meeting minimum flows in the Upper Peace River, improving water quality in the Peace River, and protecting Charlotte Harbor, an estuary of national significance. The goal of the Lake Hancock Outfall Treatment Project is to improve water quality discharging from Lake Hancock through Saddle Creek to the Peace River. In Water Year 2003 the Saddle Creek drainage basin, one of nine sub-basins in the Peace River Watershed (see Figure 1.1-1), comprised approximately 6 percent of the total flow of the Peace River, yet contributed approximately 13 percent of the watershed's total annual nitrogen load. Nitrogen has been identified as the primary target nutrient in restoring water quality in the Peace River and preventing degradation of Charlotte Harbor, a Surface Water Improvement and Management (SWIM) priority water body. The Peace River ecosystem routinely suffers from algae blooms during periods of low flows and warm weather. These events not only affect the fish and wildlife associated directly with the river and estuary, but also affect the region's largest potable surface water supply system, operated by the Peace River/Manasota Regional Water Supply Authority. Many of the basins along the Peace River, including Lake Hancock, have been identified by the Florida Department of Environmental Protection as impaired under the Clean Water Act, requiring that Total Maximum Daily Loads be established. Furthermore, nitrogen loads have been predicted to increase significantly over the next 20 years as a result of growth. Water quality treatment of discharges from Lake Hancock has been identified by the District as the most cost effective means of reducing nitrogen loads to the Peace River and Charlotte Harbor (SWFWMD, 2006).

Lake Hancock is an approximately 4,500-acre, shallow, hypereutrophic lake, located southeast of the City of Lakeland and north of the City of Bartow within west-central Polk County in central Florida, which receives runoff from a 135 square mile watershed (see Figure 1.1-2). This drainage basin combined with the Peace Creek Canal drainage basin constitutes the headwaters of the Peace River which flows into Charlotte Harbor, Florida's second largest open-water

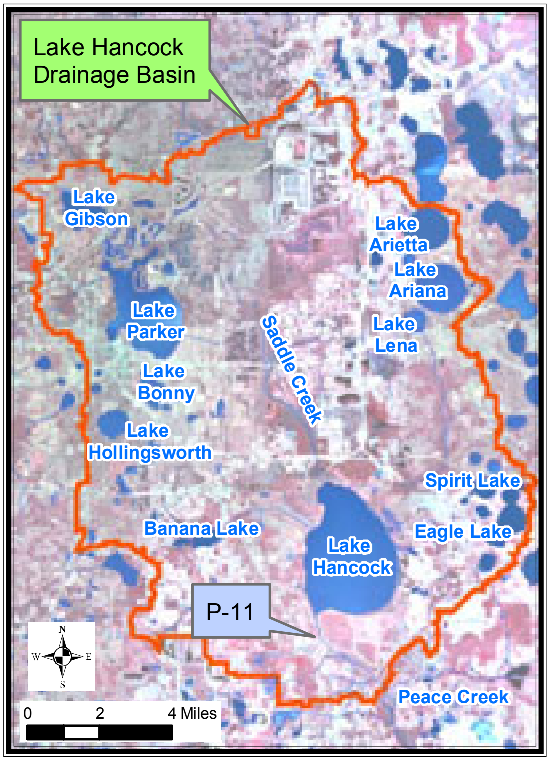

Figure 1.1-2 Lake Hancock Watershed Location Map

estuary. There are three primary tributary drainage systems that contribute to Lake Hancock and comprise approximately 81% of the total watershed (BCI, 2004). Saddle Creek originates east of the City of Lakeland and flows southward into the northern end of Lake Hancock. The headwaters of Lake Lena Run are located in the City of Auburndale and enter the lake on its northeast side. The Banana Lake Overflow Canal receives the discharge from Banana Lake near Highland City and enters the lake on its west side. A minor tributary system, the Eagle Lake system, enters Lake Hancock on the southeast side. There are numerous significant lakes and storage features contained within the contributing watersheds of these tributary systems, including Lake Parker, Lake Gibson, Lake Bonny, Lake Hollingsworth, Banana Lake, Lake Arietta, Lake Ariana, Lake Lena, Spirit Lake, Eagle Lake, and Millsite Lake. In all, lakes comprise approximately 20 square miles of the total watershed drainage area. In addition, significant portions of the Lake Hancock watershed have been mined, creating remnant overburden spoil piles, clay settling areas, and depressions. Land surface elevations in the watershed range from highs of around 265 ft NGVD along the ridgelines to 98 ft NGVD near the lake outfall to Saddle Creek at its southwest corner.

Lake Hancock is the third largest lake in Polk County and is publicly-owned up to the ordinary high water line. The average lake depth is 5 feet, and a muck layer ranging in thickness from 1 to 5 feet covers its bottom. Bottom elevations generally range from 92 to 96 ft NGVD. The lake is within the jurisdiction of the Southwest Florida Water Management District (SWFWMD), which controls the lake level and discharge by the operation of a control structure, Structure P-11, located approximately 0.7 miles downstream from the lake on the lower reach of Saddle Creek (Figure 1.1-3). Constructed in 1962 to regulate discharges into the Peace River for flood control purposes, Structure P-11 is a steel sheet pile weir with two 20’x7’ radial gates sitting on a concrete spillway. The gates and weir overflow at elevation 98.7 ft NGVD, and the spillway invert is at 91.7 ft NGVD.

The SWFWMD operates P-11 according to an operation schedule and lake management levels that were adopted in 1980 to provide guidance for management of seasonal lake level fluctuations. The “Ten-Year Flood Guidance Level” for the lake, 102.4 ft NGVD, approximates an expected level of flooding and is an advisory level for use as a discretionary guideline for lakeshore development. The “Minimum Flood Level” or “High Level” is defined as the maximum level that the lake would achieve by normal operation of the control structure; it is 99.0 ft NGVD for Lake Hancock. The “Maximum Desirable Level”, 98.5 ft NGVD, is an

Figure 1.1-3 Structure P-11 on Lower Saddle Creek

optimal desirable lake elevation based on existing development on the shoreline and floodplain. The "Low Management Level" of 96.0 ft NGVD is the normal yearly low level used as a guide to operate the P-11 control structure. Finally, the "Extreme Low Management Level", 94.0 ft NGVD, is a drought year low level used to operate P-11.

Since the construction of P-11 in 1962, lake levels have been relatively stable, generally fluctuating between elevation 96.0 and 99.0 ft NGVD. The median lake level of Lake Hancock over the period of record is approximately 97.9 ft NGVD. The lowest recorded lake level of 94.4 ft NGVD occurred as the result of a sinkhole that formed near the center of the lake in 1968 and drained it for a period of seven months. The maximum observed lake level of 101.88 ft NGVD was recorded in 1960 after Hurricane Donna passed through the region. This elevation was approached during the 2004 hurricane season, when three separate hurricanes that passed through central Florida resulted in a maximum lake elevation of 101.55 ft NGVD in mid-September 2004. The United States Geological Survey (USGS), who has monitored lake levels and flows at P-11 on a regular basis since 1963, reports peak flows in Saddle Creek at that time far in excess of any previously recorded (greater than 1350 cfs).

Lake Hancock has a long history of water quality problems dating back to the 1950's, when concern arose over the industrial, mining, and agricultural activities that had degraded the lake and the Peace River. Domestic and industrial wastewater treatment plant effluent discharges from the Cities of Lakeland, Auburndale, and Winter Haven into tributaries of Lake Hancock resulted in high nutrient concentrations in the lake. Hypereutrophic conditions developed, characterized by excessive growth of persistent blue-green algal blooms, poor water clarity, widely diurnal fluctuations in dissolved oxygen, and the accumulation of organic matter in the form of a thick layer of organic sediments on the lake bottom. By the early 1990's, most of the domestic and industrial discharges into the Lake Hancock tributaries had been totally or partially discontinued, but Lake Hancock remains in a highly hypereutrophic state (ERD, 1999).

Over recent years, a number of studies have been conducted to characterize and analyze Lake Hancock water quality and environmental conditions, and to evaluate restoration measures to return the lake to an acceptable state of health and reduce the discharge of poor quality lake water to the Peace River and Charlotte Harbor estuary.

- In 1987, Zellars-Williams Company conducted a feasibility study for lake restoration for the Florida Institute of Phosphate Research (FIPR), entitled "Lake Hancock Restoration Study", in which sediment dredging was recommended.

- In 1991, the Florida Game and Freshwater Fish Commission (FGFFC) developed a proposed lake restoration plan that included a partial lake drawdown to consolidate portions of the organic bottom sediments.
- A plan to mine phosphate ore located under the lake bottom was evaluated in the late 1990's by IMC-Agrico, but was abandoned due to environmental concerns.
- In 1997, the Southwest Florida Water Management District retained Environmental Research and Design, Inc. to develop water and nutrient budgets for Lake Hancock and to evaluate alternative water quality improvement measures to improve the water quality of discharges from the lake.
- In 1999, the Florida Fish and Wildlife Conservation Commission (FFWCC), formerly the FGFFC, developed a revision of its previous lake restoration plan, the "Lake Hancock Habitat Enhancement Plan", which proposed lake drawdown and mechanical excavation of sediments.
- In 2002, Polk County and the Florida Department of Environmental Protection (FDEP) retained Camp Dresser & McKee, Inc. (CDM) to evaluate, singularly and in combination, a large spectrum of alternative lake restoration techniques in its "Lake Hancock Restoration Management Plan", including sediment removal by hydraulic dredging, lake drawdown and mechanical excavation, chemical inactivation of sediment with alum, capping of bottom sediments, recirculating treatment wetlands, wetland treatment of inflows, lake level manipulation, biological control, and habitat restoration.

To this date, there have been no implementation actions taken by any of the various agencies and entities that have conducted these studies and plans.

1.2 PROJECT SCOPE OF WORK

For this current Lake Hancock Outfall Treatment Project, the Southwest Florida Water Management District has retained Parsons Water and Infrastructure, Inc. to assist in the design and construction of a treatment system to reduce the annual total nitrogen load discharging the lake as a means to improve poor water quality in the upper Peace River. This poor water quality from the lake affects the entire river all the way to Charlotte Harbor, an "estuary of national significance" and a State Surface Water Improvement and Management (SWIM) priority water body. This project evolved from a study by the District that compared the relative cost and benefits of improving Lake Hancock water quality conditions versus improving the water quality of the water leaving the lake. Results showed little benefit for in-lake water quality treatment in regard to the high cost of treatment, and the option to treat the lake discharge was selected.

The Lake Hancock Outfall Treatment Project is being conducted concurrently with another District initiative in the watershed, the Lake Hancock Lake Level Modification Project (LLMP). The goal of the LLMP is to store additional water in Lake Hancock by raising the control elevation of the existing P-11 lake outflow structure on the lake and to slowly release the water during the dry weather season to help meet the minimum flow requirements in the Peace River. The District has received a conceptual Environmental Resource Permit for the LLMP to raise the normal operating level of Lake Hancock from its current 98.7 ft NGVD up to 100.0 ft NGVD. When the project is implemented, it will alter the operation of the lake control structure and will directly alter the timing and distribution of lake discharges from that of its historical record. Therefore, the two Lake Hancock projects are being closely coordinated to provide compatible and symbiotic designs.

For the feasibility study, the District has targeted two optional nitrogen load reduction goals. The first goal reduces the nitrogen load by 45 percent of the existing total nitrogen load currently discharging Lake Hancock through the P-11 Structure. The second goal was developed based on expected performance of treating 52 cfs utilizing surface flow wetlands. This flow rate represents the estimated minimum discharge needed to meet Minimum Flow Levels (MFL) in the Peace River downstream at Ft. Meade. MFLs have been proposed by the District for the Upper Peace River as mandated by the State Legislature, through Chapter 373.042, Florida Statutes. The State Legislature also directs, through Chapter 373.0421, that when established MFLs are not being met, the water management districts are responsible for implementing a recovery strategy. The 52 cfs treatment goal corresponds to an estimated annual total nitrogen load reduction of about 27 percent.

The first phase of the project involves Research, Monitoring and Data Acquisition, and a Feasibility Study to investigate and evaluate the various water treatment methods that might be applicable to treat the lake discharge. The treatment technologies that are being considered include flow-through constructed wetlands, other aquatic plant-based technologies, sedimentation ponds, direct filtration systems, dissolved air flotation, sedimentation basins followed by filtration, and microscreen filtration. Following the Feasibility Study, the project will progress with the final design, engineering, and environmental permitting, and preparation of construction documents for the selected treatment system. Construction will commence upon issuance of permits.

The construction of the Lake Hancock Outfall Treatment Project is to be sited on District-owned property near the lake outfall to Saddle Creek. In 2003, the District purchased a 3,535-acre parcel of land formerly known as the Old Florida Plantation property with approximately 4 miles of shoreline bordering the southern and southeastern shore of Lake Hancock at a cost of $30.5 million. In a more recent acquisition, the District purchased a 231-acre parcel known as the Saddle Creek property for $4.9 million located along Saddle Creek just south of Structure P-11 between the creek and U.S. Highway 98. These properties, shown in Figure 1.2-1, provide a range of viable options for the siting of alternative lake outfall treatment facilities.

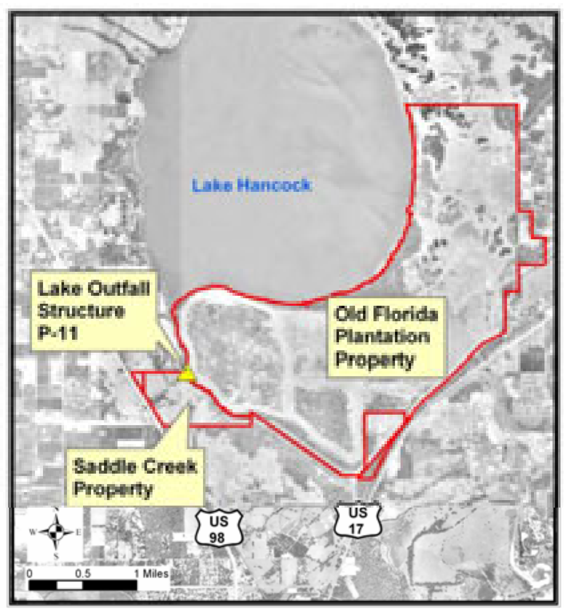

Figure 1.2-1 Saddle Creek & Old Florida Plantation Properties

When the District's Governing Board approved the purchase of the Old Florida Plantation (OFP) it recognized that a number of benefits could result from the acquisition of this property. In addition to a potential site for the Lake Hancock Outfall Treatment project and inundation of low lying areas for the LLMP; the OFP property is a significant and integral piece of a potential, extensive conservation corridor along the Peace River and connecting to the Green Swamp as identified in the District's land acquisition plans. Because the previous owners were only willing to sell the entire parcel rather than the portion of their land that was contained within the District's conservation corridor project, the Board recognized that a portion of the property could be eligible for future surplus. There is an existing Development Order on the OFP property which the District intends to maintain or modify so that the surplus land remains marketable while allowing the construction and maintenance of District projects.

Most of the Old Florida Plantation (OFP) property has at one time or another been mined for phosphates, beginning in the late 1940s and through the 1960s. The property currently consists of reclaimed phosphate land used primarily for cattle grazing. Reclamation activities commenced in the northern portion of the property in the 1980s and the southern portion of the site was substantially completed and approved by FDEP under its non-mandatory program. Reclamation included reforestation, bank stabilization, and impoundment removal. The resulting reclaimed site includes approximately 2672 acres of uplands, 478 acres of wetlands, 309 acres of surface waters (mostly borrow lakes), and 76 acres of roads and easements. The wetlands portions of the property include wetlands located along the shore of Lake Hancock and post-reclamation wetlands approved as part of the non-mandatory reclamation process.

The OFP property can be divided into two distinctly different classifications based on the soils and topography. The northern and eastern areas of the property consist of disturbed native soils of sandy to sandy silt composition that are remnant from the phosphate mining and reclamation activities. These areas have highly irregular topography that generally ranges from a low of 99 ft NGVD around the numerous water features to over 120 ft NGVD. Figure 1.2-2 (enclosed in map pocket at back of report) shows the detailed topography of the property. The southern portion of the OFP property consists of what were the clay settling ponds for the phosphate mining activities. These large, generally flat cells are surrounded by broad, elevated berms. Because of the high clay content of the soils within these cells, they are generally undesirable for development due to the significantly high potential for settlement, both short and long term, when loaded. The topography within the individual clay settling pond cells is relatively level and uniform, as can be seen in Figure 1.2-2, with a general range from approximately 112 to 122 ft NGVD. The OFP surface drainage system is split, with much of the site discharging directly to Lake Hancock, and the remaining areas discharging into Saddle Creek downstream from the P-11 structure.

The Saddle Creek property, located along Saddle Creek just south of Structure P-11 between the creek and U.S. Highway 98, provides a 231-acre site for the proposed Lake Hancock Outfall Treatment Project. This recently acquired property is primarily undeveloped uplands that have been used for cattle grazing. The site is undisturbed (i.e. not mined) with mostly fine sand to sandy soils. The site generally slopes from the northwestern corner at elevation 115 ft NGVD to the southeast at elevation 97 ft NGVD. The eastern half of the property is located within the 100-year floodplain of Lower Saddle Creek, as defined in the FEMA flood insurance rate map at elevation 102 ft NGVD.

Both of these properties have been acquired by the District, and no further acquisitions are envisioned to be required for the Outfall Treatment Project. It is important in the comparison of alternative treatment technologies conceptual plans to consider the relative land requirements of the alternatives. Therefore, the value of the land required for the construction of alternative treatment facilities is included in the economic analyses presented herein based on the original purchase price as a basis to compare all alternatives. An appraisal of the value of the OFP surplus provided by the District's Land Resources Department in February 2006 was used in further evaluating two of alternatives proposed for the OFP site.

1.3 TECHNICAL MEMORANDUM ORGANIZATION

The intent of this memorandum is to describe the applicable technologies and provide conceptual planning level comparative cost estimates for engineering design, equipment and construction, and annual estimated costs for operation and maintenance. In addition to cost comparisons, qualitative criteria are also evaluated to compare ancillary benefits of the different technologies. The Alternative Treatment Technologies Evaluations Technical Memorandum begins in Section 2 with an examination of the hydrologic conditions and historical record of lake discharge to define the range and distribution of flows that are to be treated by the proposed lake outfall treatment systems. In Section 3, there is a discussion and analysis of historical water quality monitoring data to define the expected source water quality for the lake outfall treatment facilities. Bench-scale laboratory testing and analyses were conducted on lake water samples to assess critical factors necessary for the preliminary design and evaluation of alternative treatment technologies. The results of the bench-scale tests are presented in Section 4. In Sections 5 and 6, conceptual designs are presented and discussed for alternative aquatic plant-based and physical treatment technologies, respectively. Each treatment technology conceptual design is discussed and evaluations of operation and maintenance requirements, expected finished water quality, nitrogen removal efficiency, residuals disposal, regulatory requirements, and life cycle costs are presented. In Section 7, the various alternative treatment technologies are compared, and recommendations are presented in Section 8 for further development of the most viable of the alternative treatment systems.

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SECTION 2

HYDROLOGIC ANALYSIS

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SECTION 2.0

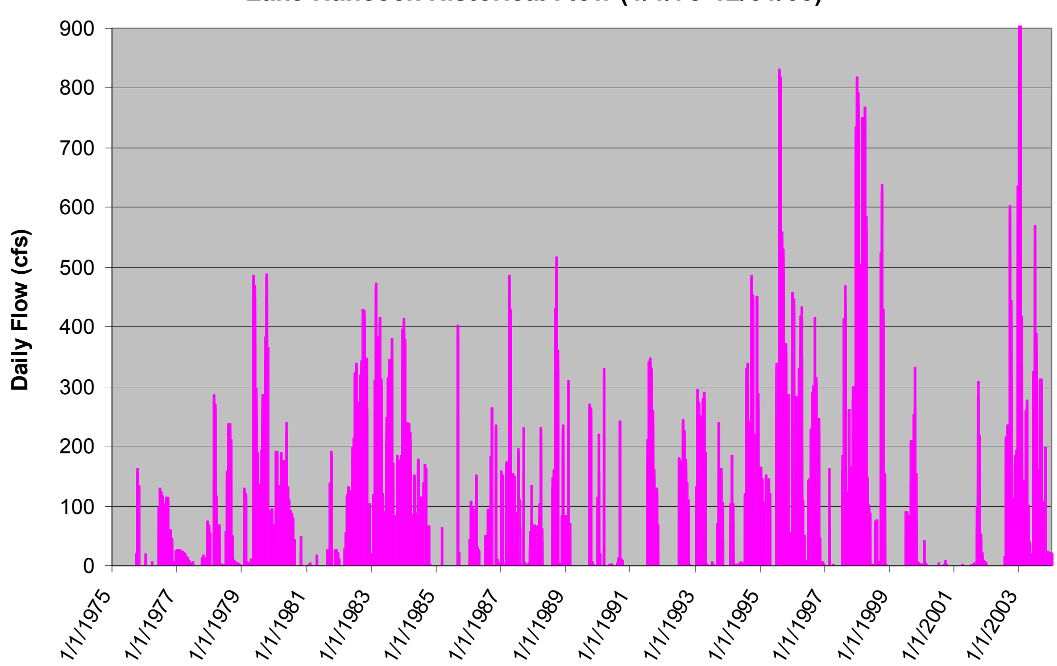
HYDROLOGIC ANALYSIS

2.1 HISTORICAL LAKE HANCOCK DISCHARGE

The design of the Lake Hancock Outfall Treatment Project is directly dependent upon the expected range and distribution of the lake discharge. To evaluate these parameters, Parsons utilized the historical record of flows reported by the U.S. Geological Survey (USGS) at its Saddle Creek streamflow gaging station (Station No. 02294491) located immediately downstream of the lake outfall Structure P-11. The USGS has operated this gaging station since November 1963. It is important to recognize that the flow recorded at this site is entirely regulated by the gated P-11 structure as it is operated by the SWFWMD. As such, there is no firm correlation between the lake elevation and the flow that is discharged downstream to Saddle Creek.

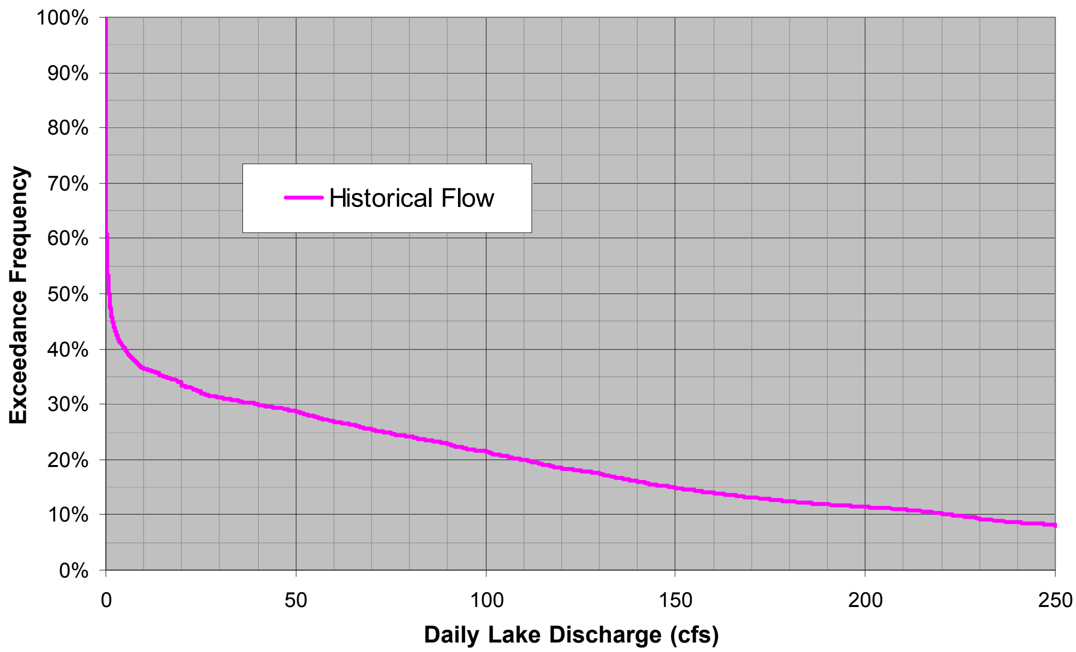
For the purpose of this analysis, Parsons examined the 29-year historical period of Lake Hancock daily discharge from January 1, 1975 through December 31, 2003. Figure 2.1-1 presents a plot of the daily flows over this 29-year period. The average daily flow was 62.6 cubic feet per second (cfs), but the lake discharge varied over an extreme range on both a daily, seasonal, and yearly basis. The maximum daily flow of 936 cfs was recorded in January 2003. There are many days over the period of record when the lake did not discharge at all. In fact, zero flow was recorded for nearly 26% of the days. In addition, the median flow value for the period of

Figure 2.1-1
Lake Hancock Daily Discharge
Lake Hancock Historical Flow (1/1/75-12/31/03)



record is 0.87 cfs, meaning that daily flow was less than this value for half of the days. Figure 2.1-2 presents a Lake Hancock daily flow exceedance frequency plot for the 1975-2003 history of daily flows that illustrates graphically how biased the daily lake discharge frequency is towards the extreme low flow regime. This would present a significant challenge to the design of a lake outfall treatment system, since it means the treatment facilities would be essentially idle during these extreme low flow days.

Figure 2.1-2
Lake Hancock Daily Discharge Exceedance Frequency
Historical Flow (1/1/75-12/31/03)



2.2 LAKE HANCOCK LAKE LEVEL MODIFICATION PROJECT DISCHARGE

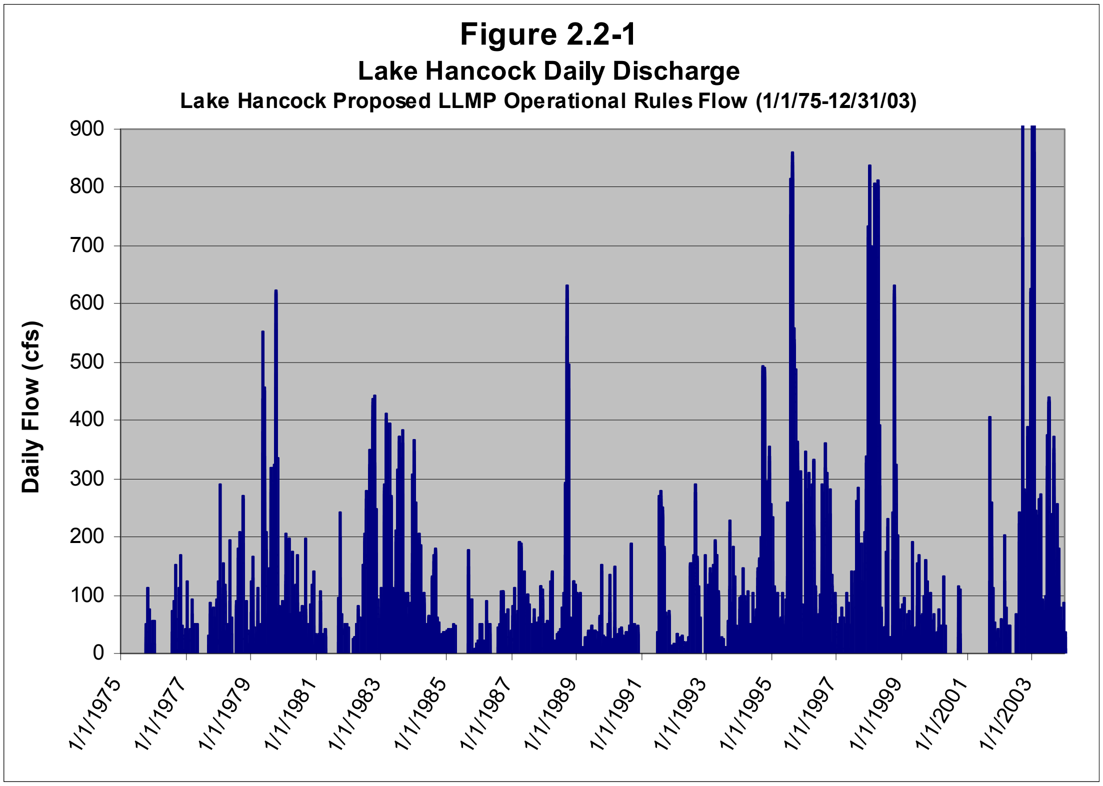
The primary purpose of the proposed Lake Hancock Lake Level Modification Project (LLMP) is to reestablish the minimum flows and levels (MFLs) in the Upper Peace River. The Florida Legislature, through Chapter 373.042, Florida Statutes, mandates that the five water management districts establish minimum flows and levels for all surface watercourses that include lakes and streams, and the minimum groundwater level in an aquifer. The minimum flow is defined as the "limit at which further withdrawals would be significantly harmful to the water resources or ecology of the area". The District has established minimum flow requirements at three locations on the upper Peace River coinciding with the historical USGS streamflow gaging stations near Bartow, Fort Meade, and Zolfo Springs. The adopted minimum low flow values for these sites are 17 cfs, 27 cfs, and 45 cfs respectively (SWFWMD, 2002). The District's goal is to meet or exceed these minimum flows 95 percent of the time on an annual basis, or 348 days per year.

As one of the District's proposed MFL recovery strategies, the proposed LLMP will provide additional storage of surface waters within Lake Hancock that will then be used to maintain minimum flows in the river when required. The LLMP is currently in the planning and feasibility stage until such time as construction is initiated. The project as presently envisioned is subject to changes because of many factors including but not limited to areas of engineering, permitting, the environment, and costs. If implemented, this will benefit the design of the Lake Hancock Outfall Treatment Project, as it will provide a more normalized distribution of daily discharges from the lake, with far fewer days in the extreme low flow range. The current LLMP plan that is being proposed by the District is to capture and store additional surface waters in Lake Hancock by modifying the existing lake outfall Structure P-11. The current Maximum Desirable Level of the lake of 98.5 ft NGVD. Under the LLMP, the proposed maximum Desirable Level will be raised to 100.0 ft NGVD. Above this level, releases would be made to lower the lake. The Low Operating Level for the lake, below which no releases will be made through the P-11 structure regardless of downstream conditions, is proposed to be 97.5 ft NGVD. Releases for meeting the downstream MFL requirements would therefore be made when the lake is between 97.5 and 100.0 ft NGVD.

The proposed operational protocols that will be followed provide that all lake inflows will be "captured" (i.e. stored) within the lake when it is below the Low Operating Level of 97.5 ft NGVD. Between 97.5 and 100.0 ft NGVD, the "capture rate" for lake inflows will be 60%, with the remaining 40% of inflows being released. At no time, however, would lake releases be lowered below the point where downstream minimum flow rates in the upper Peace River are not being met, including any losses to sinkholes along the river course, until the lake reaches the proposed Low Operating Level threshold of 97.5 ft NGVD. Based on an assumed sinkhole loss rate of 25 cfs, the expected minimum required lake release to meet the downstream minimum flow rate will be 52 cfs. This flow rate is of significance if it is determined to be technically or economically infeasible to achieve the full 45% nitrogen load reduction goal. In such a case, the District has adopted an optional minimum goal of providing an optimal level of treatment for all lake discharges up to the 52 cfs minimum flow requirement, as established by the LLMP. In this way, treatment will be provided for the full quantity of water discharged from the lake during dry weather conditions, when the lake will be operated to release stored water to meet the downstream minimum flow requirements. At lake discharges greater than 52 cfs, the excess flow would not receive treatment.

To develop and evaluate alternative operational protocols for the LLMP, the District utilized a water budget spreadsheet model of Lake Hancock for the 1975-2003 historical period. This model incorporated the 29-year period of record of lake levels and discharges at the lake outfall Structure P-11 to generate a history of net lake inflow/outflow (including in-lake losses such as evaporation and seepage) on a daily basis by which alternative operational scenarios were tested. The historical period was modified by removing the historical discharge to the contributing watershed area from the City of Lakeland Wastewater Treatment Plant, which discharged into the Stahl Canal, a tributary to Banana Lake, until April 1987 (BCI, 2004). Between January 1975 and April 1987, this plant discharged an average of 9.9 cfs, over 19% of the total Lake Hancock outflow through Structure P-11 over the same period, and approximately 16% of the 29-year average lake discharge. The historical record of Lake Hancock inflows were reduced by the historical treatment plant point discharge to provide a truer representation of the current watershed hydrology and expected lake inflows. As a result, the average outflow from Lake Hancock for the 1975-2003 period of record was reduced from 62.6 cfs to 58.65 cfs (42,463 ac-ft/yr).

Figure 2.2-1 presents a plot of the Lake Hancock daily flows over the 1975-2003 period that were generated by the District's spreadsheet model to reflect the operational guidelines of the proposed LLMP. By examination of a superposition of the historical record of daily lake discharges in Figure 2.2-2, it can be readily discerned how the proposed LLMP will result in a substantial redistribution of the lake discharges by filling in the periods of historically low flow



(or no flow). This is further demonstrated in Figure 2.2-3 as a comparison of the lake discharge exceedance frequency plots between the historical lake flows and those with the proposed LLMP operational protocols. The frequency of days with lake discharge of 1.0 cfs or less will be reduced from 51% to 34%. Similarly, the frequency of days with flows less than 10 cfs is decreased from 64% to 44%, and those days of less than 20 cfs from 66% to 54% with the implementation of the proposed LLMP operational plan. The median lake discharge is 15.8 cfs for the conditions proposed under the LLMP compared to the 0.87 cfs for the actual historical record of Lake Hancock discharge.

2.3 LAKE HANCOCK OUTFALL TREATMENT CAPACITY AND VOLUME

The design of a water quality treatment facility to provide treatment of Lake Hancock discharge must consider the range of flows that are to be treated to determine the treatment capacity. As can be seen in Figure 2.2-1, the daily discharge from Lake Hancock is predicted to range from zero to over 900 cfs with the implementation of the proposed LLMP operational guidelines. Therefore, in order to provide treatment for all the lake discharge, it would be necessary to provide a treatment facility with a capacity up to the 900 cfs limit. This would be an impractical design, since a large percentage of the treatment facility would remain idle and unused for protracted periods of time.

Figure 2.2-2
Lake Hancock Daily Discharge
Lake Hancock Proposed LLMP Operation vs Historical Flow (1/1/75-12/31/03)

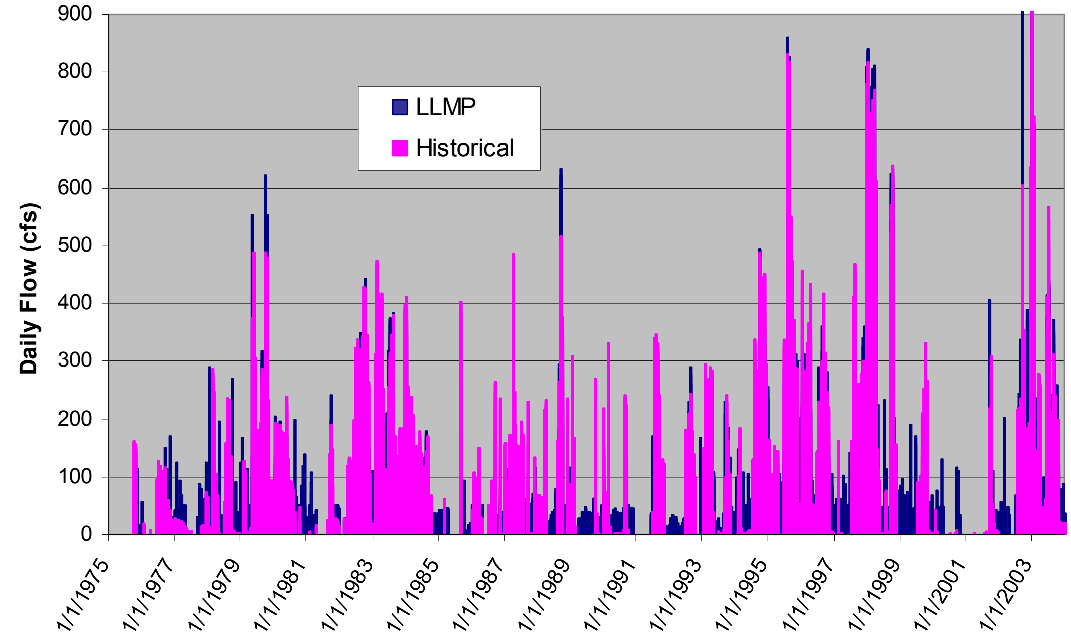
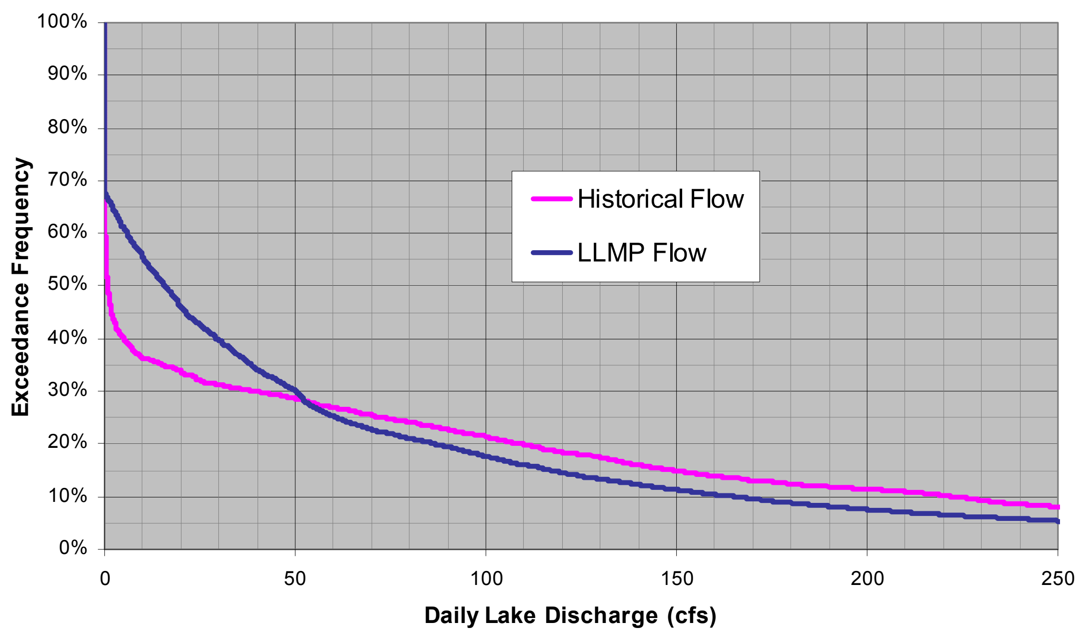


Figure 2.2-3
Lake Hancock Daily Discharge Exceedance Frequency
Lake Hancock Proposed LLMP Operation vs Historical Flow

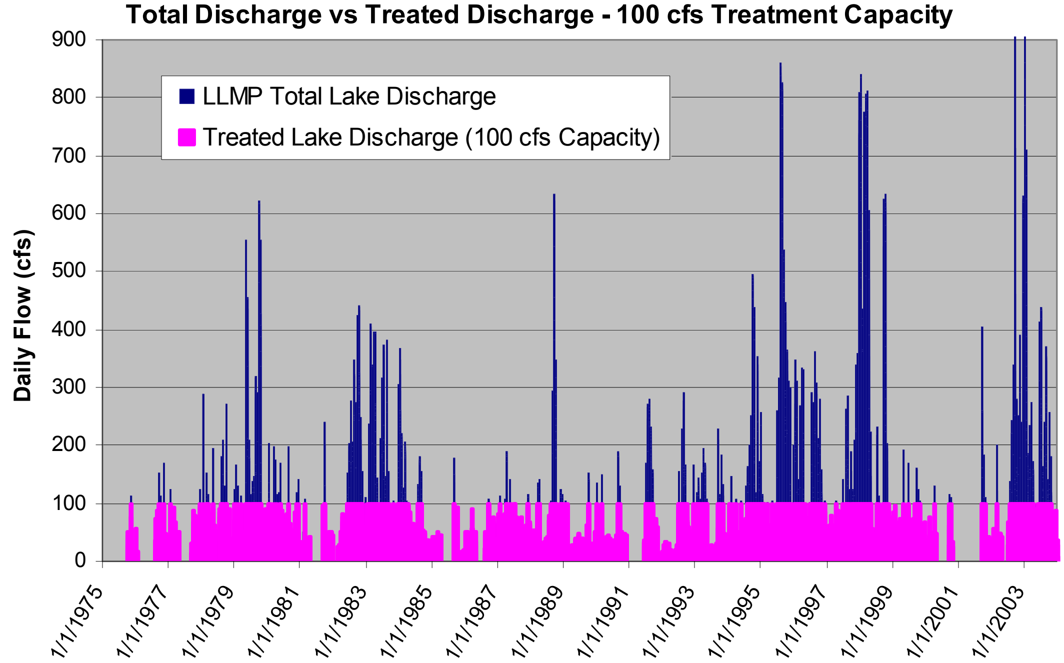


As discussed in Section 1, the stated goal of the Lake Hancock Outfall Treatment Project is to construct a cost effective regional surface water treatment system to reduce total nitrogen loads at a maximum efficiency of 45 percent in the Lake Hancock discharge. For the purpose of this analysis, it is assumed that this goal is an average annual reduction in the total nitrogen load, and the nitrogen load of the lake discharge can be calculated as the product of the annual flow volume and the average total nitrogen concentration in the discharge. Therefore, to achieve the 45% load reduction, the design of the Lake Hancock treatment facility must consider both the percentage of the average annual flow that will be treated, and the efficiency of the facility to reduce the total nitrogen concentration in the lake discharge.

Because it would be impractical to provide a facility large enough treat the entire lake discharge, Parsons developed a means of determining the volume of treated discharge as a function of the capacity of a proposed treatment facility. Using the time series of Lake Hancock daily flows over the 1975-2003 period that were generated by the District's spreadsheet model based on the operational guidelines of the proposed LLMP, an analysis was performed that determined the volume of lake discharge that could be treated for each of a series of presumed treatment capacities.

Figure 2.3-1 provides a visual representation of how this analysis was performed. In this case, for a treatment facility capacity of 100 cfs, the volume of lake discharge that would be treated is represented as the pink area of the plot. The blue area, representing those portions of the total daily lake discharge in excess of the 100 cfs treatment capacity, would be the volume of untreated lake discharge.

Figure 2.3-1
Lake Hancock Proposed LLMP Operation Daily Lake Discharge
Total Discharge vs Treated Discharge - 100 cfs Treatment Capacity



In this case, the average annual treated flow volume would be 33.59 cfs (24,318 ac-ft per year), comprising 57.3% of the total average annual lake discharge of 58.65 cfs. In order to meet the project goal of 45% total nitrogen load reduction, it would be necessary for a treatment facility of 100 cfs capacity to have a treatment efficiency of approximately 78.5% (i.e. 45% divided by 57.3%).

This same methodology was applied to a series of presumed lake discharge treatment facility capacities to generate a plot of potential treatment volume as a function of the treatment facility capacity. Figure 2.3-2 presents the results of the analysis. This plot provides the basis for the determination of the required sizes (i.e. capacities) of the alternative treatment systems that are evaluated in subsequent sections. This figure also illustrates the benefits that will be realized through the implementation of the proposed LLMP, which will effectively elevate the percentage of the annual flow that will be treated for a given size treatment facility above that which could be achieved by the past history of lake operation.

Worthy of consideration in examination of Figure 2.3-2 is the curvature of the graph, which indicates a diminishing of the incremental volume of treated lake flow with the increase in the treatment facility capacity. This is further quantified in Table 2.3-1. This table shows how each additional acre-foot of treated lake discharge becomes more costly in terms of the size (hence capital cost) of the treatment facility that would be necessary to achieve that incremental level of treatment.

Figure 2.3-2
Lake Hancock Potential Treatment Volume vs. Treatment Capacity
Lake Hancock Proposed LLMP Operation vs Historical Flow (1/1/75-12/31/03)

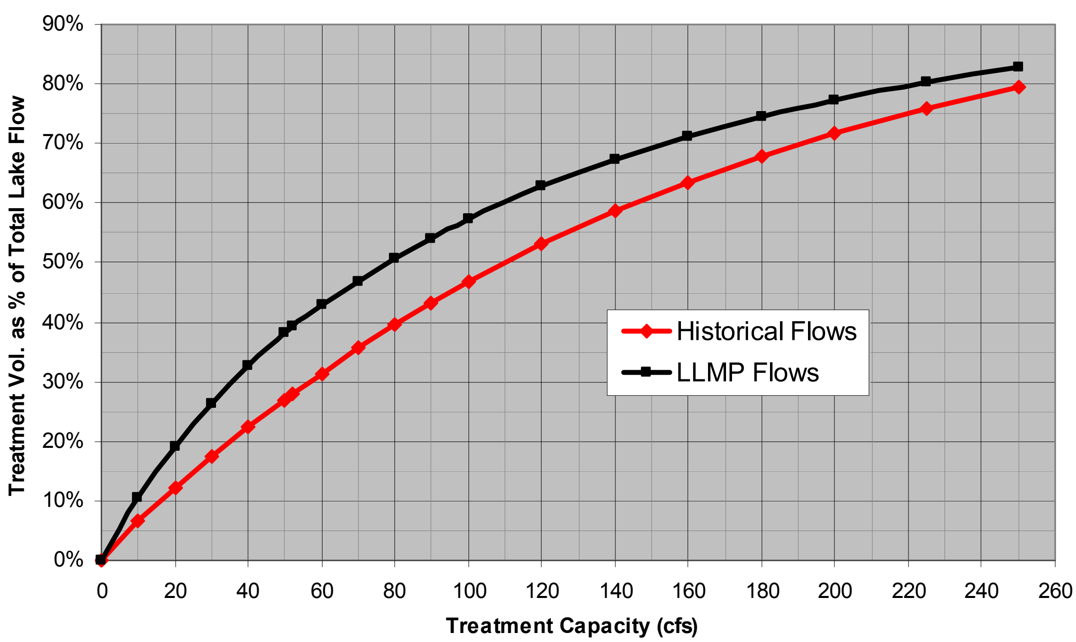
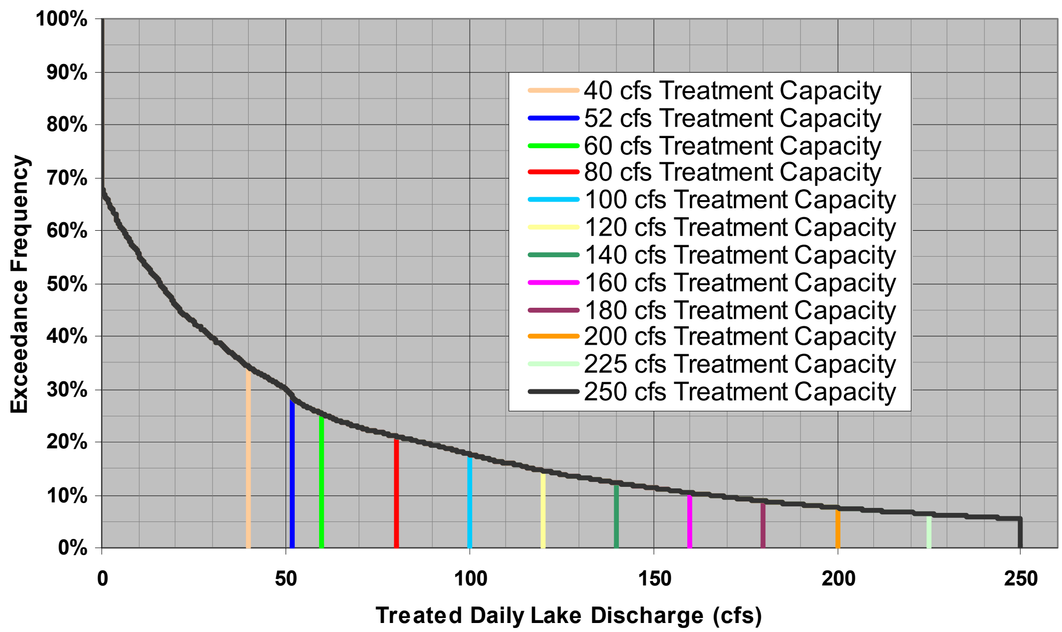


Table 2.3-1 Incremental Increase in Lake Hancock Treatment Volume Related to Increase in Treatment Facility Capacity

Treatment Facility Capacity, cfs	Average Annual Treatment Volume, ac-ft/yr	Increase of Treatment Volume per Unit Treatment Capacity, ac-ft/yr/cfs
20	8,115	-
40	13,880	288.2
60	18,178	214.9
80	21,500	166.1
100	24,317	140.8
120	26,648	116.5
140	28,586	96.9
160	30,228	82.1
180	31,619	69.5
200	32,805	59.3
225	34,063	50.3
250	35,129	42.6

Figure 2.3-3
Lake Hancock Treated Daily Lake Discharge Exceedance Frequency
Lake Hancock Proposed LLMP Operation (1/1/75-12/31/03)



An additional factor in the design of the lake outfall treatment system will be the need to accommodate a wide range of flows and the extreme variability of flows on a daily, seasonal, and yearly basis. Figure 2.3-3 defines the expected frequency of flows that would be treated by a treatment facility of the indicated capacity. The area underneath the plots is an indication of the volume of treated lake discharge, again illustrating the decline in the incremental volume with an increase in capacity. Also of consideration here is the relative percentage of time that a given size facility would be operating at its full capacity, and how that percentage declines with the increase in capacity. For example, a 100-cfs capacity lake outfall treatment facility would be operating at its full capacity an average of 17.7% of the time (65 days/year). Correspondingly, a 200-cfs treatment facility would be operating at full capacity only 7.6% of the time (28 days/year), and one-half of the facility would be unused more than 82% of the time.

These are important issues to consider in the design and selection of a Lake Hancock Outfall Treatment facility to meet the 45% nitrogen load reduction goal, and it therefore became important to reexamine this goal during the course of the alternative treatment technologies evaluation for implementation of the project. At the District's request, Parsons conducted a parallel set of alternative treatment technologies evaluations based on a reduced nitrogen load reduction goal set at providing an optimal level of treatment for the quantity of water discharged from the lake during dry weather conditions, when the lake will be operated to release stored water to meet the downstream minimum flow requirements established by the proposed LLMP. As previously discussed, the proposed LLMP has established that, when losses to sinkholes in the downstream reaches of the Peace River are factored in, the low flow release requirements for Lake Hancock will be up to a maximum of 52 cubic feet per second (cfs). This lake discharge rate was therefore established as an optional goal of the alternative treatment technologies evaluations.

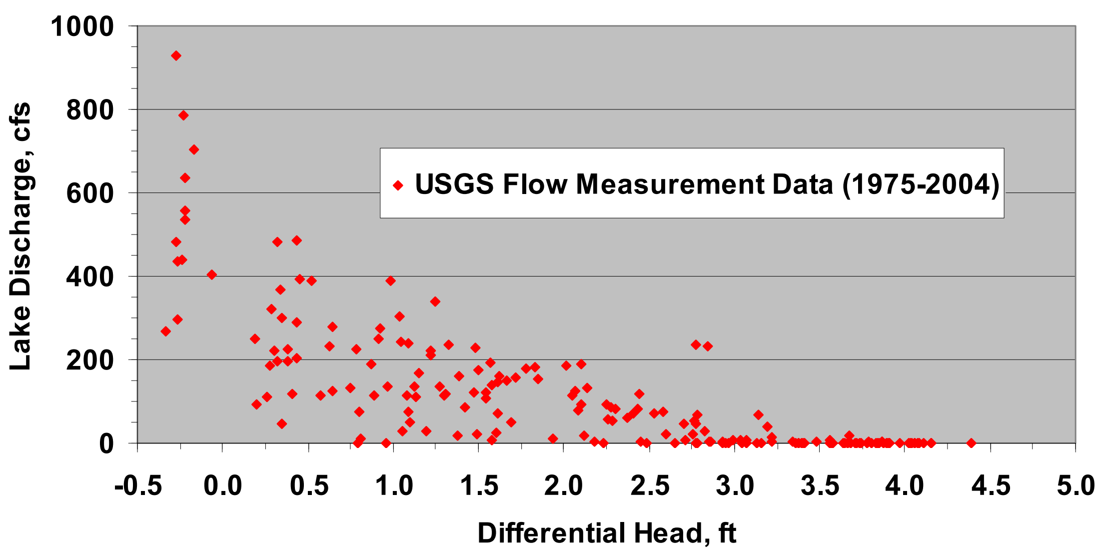
2.4 WATER DELIVERY SYSTEMS

A major consideration in the hydrologic analysis that is the basis for the conceptual design and evaluation of alternative lake outfall treatment systems is the method of water delivery to the treatment facility. As was previously discussed in Section 1, the construction of the Lake Hancock Outfall Treatment Project is to be sited either on a 3535-acre parcel of land formerly known as the Old Florida Plantation (OFP) property bordering the southern and southeastern shore of Lake Hancock, or on a 231-acre parcel known as the Saddle Creek property located along Saddle Creek just south of Structure P-11 between the creek and U.S. 98. These properties are shown in Figure 1.2-1.

There are two options for the feeding of lake water to these sites, either gravity flow or pumping. The normal range of lake levels proposed by the LLMP operational plan will be from elevation 97.5 ft NGVD to 100.0 ft NGVD. The Old Florida Plantation property, for the most part, lies above elevation 104 ft NGVD and those areas lower than 102 ft NGVD are isolated depressions or fringe lake shoreline areas. It would therefore require an extensive amount of excavation to lower a potential treatment facility site on the OFP property to the point where gravity flow would be possible.

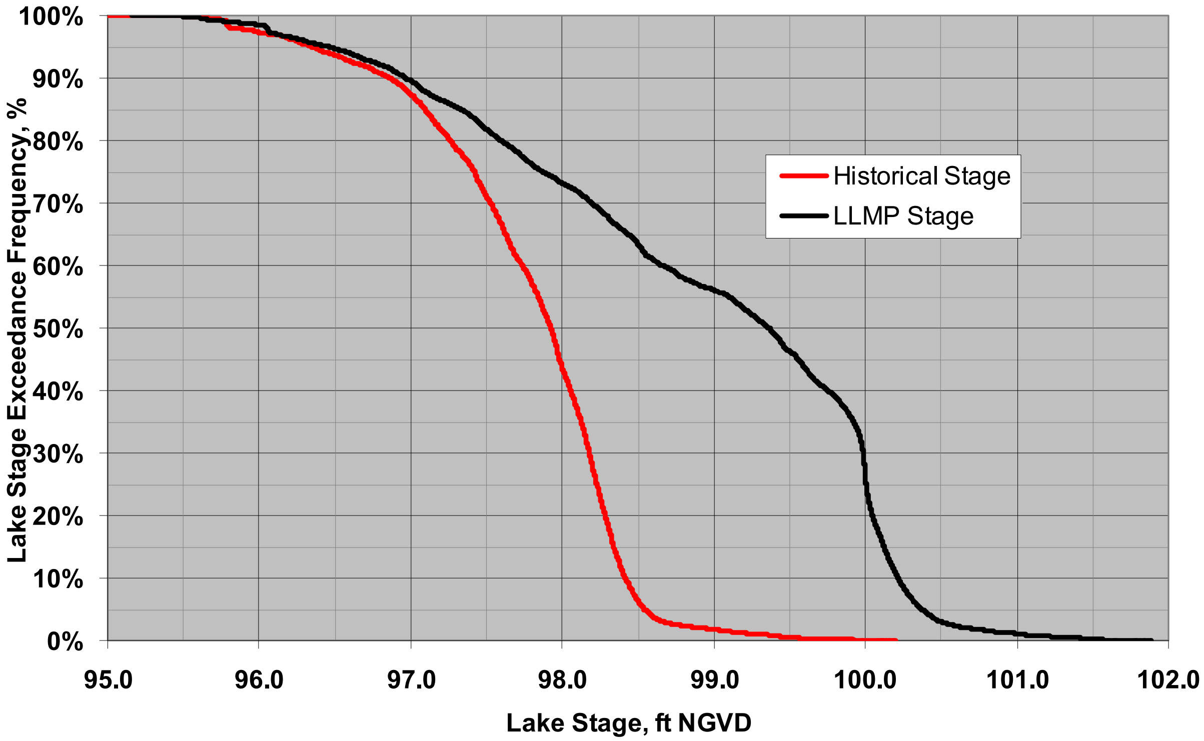
While the Saddle Creek property is substantially lower, with site topography ranging from 97 ft NGVD to 114 ft NGVD, the lower portions of this property, where excavation to provide gravity flow may be feasible, are located within the 100-year floodplain of Saddle Creek, with a 100-year flood elevation of approximately 102.7 ft NGVD according to the proposed LLMP plan. At the same time, the proposed LLMP plan predicts a 100-year flood elevation of approximately 103.2 ft NGVD for Lake Hancock. This indicates that there is only a small differential hydraulic head between the lake level upstream of the P-11 structure and the downstream water level in Saddle Creek during high flow conditions. This condition means that there is a limited head to feed water by gravity through a treatment facility. As an illustration of this, Figure 2.4-1 presents a plot of the differential head between Lake Hancock and Lower Saddle Creek versus lake discharge over the past 30 years. These data were derived from USGS measured flow data in the creek used as the basis for its rating curve at gaging station no. 02294491. This figure shows that, although the lake is frequently over two feet higher than the downstream water elevation in Saddle Creek, the flow conditions are usually in the lower range (less than 100 cfs) when this occurs. Conversely, the differential head is much less during the higher flow conditions (greater than 100 cfs). At these times, differential head is frequently less than one foot, which is inadequate to allow for gravity flow through the proposed lake outfall treatment facility.

Figure 2.4-1
Lake Hancock / Lower Saddle Creek
Differential Head Versus Flow



With the implementation of the proposed Lake Level Modification Program, the future regulation of Lake Hancock levels will alter the historical differential head relationship. Design parameters for the new Lake Hancock outfall structure have not been evaluated as yet in the LLMP, so conclusions as to the impact to differential head are preliminary. Based on proposed LLMP lake budget analyses provided by the District, the projected changes are illustrated in the comparison of lake stage frequency plots in Figure 2.4-2. Because the lake would be maintained at a minimum elevation of 97.5 ft NGVD or higher (no lake releases will occur below this elevation), the anticipated changes due to the proposed LLMP would be most evident in the range of lake elevations between 98.0 and 100.0 ft NGVD. As seen in Figure 2.4-2, there would be an increase in the available differential head of 1.5 to 1.8 feet for lake stages greater than 98.0 ft NGVD. However, the relative increase in differential head diminishes to the point of no change in the lower lake stages (less than 98.0 ft NGVD).

Figure 2.4-2
Lake Hancock Stage Frequency
Lake Hancock Proposed LLMP Operation vs Historical Lake Stage (1/1/75-12/31/03)



While it is conclusive that gravity flow is not a practical means of flow delivery for a treatment facility sited on the Old Florida Plantation property, detailed hydraulic modeling and design is required to definitively evaluate its viability for the Saddle Creek property. Given the variability of available head and a constantly varying head-discharge relationship, pumping was adopted for both sites in the evaluation of alternative lake outfall treatment systems. A pump station would lift water from Lake Hancock, upstream of P-11, and deliver it to the treatment facility, which would then treat and discharge the effluent to Lower Saddle Creek downstream of P-11.

SECTION 3

WATER QUALITY CHARACTERIZATION

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SECTION 3.0

WATER QUALITY CHARACTERIZATION

Concerns over water quality problems in Lake Hancock and the Peace River date back to the early 1950s when the Florida State Board of Health conducted an investigation of water quality in Lake Hancock and the entire Peace River Basin prompted by severe industrial abuse of the river system. Domestic and industrial wastewater treatment plant effluent was discharged from the cities of Lakeland and Winter Haven into tributaries which ultimately reach Lake Hancock. As a result of the treatment plant discharges, Lake Hancock began to develop high nutrient concentrations and high levels of pathogenic bacteria. The growth of water hyacinths began to accelerate in the nutrient-rich water. Accumulation of organic matter within the lake began to occur as a result of hypereutrophic conditions within the lake, and ongoing herbicide treatments to control water hyacinths. The rapid accumulation of organic matter in the lake was noted in 1969 by the Florida Game and Freshwater Fish Commission, which recommended immediate restoration measures for Lake Hancock that included deepening portions of the lake. The organic material on the bottom of Lake Hancock has accumulated to depths as much as 5.5 ft thick, with an estimated 18,000,000 cubic yards of muck on the bottom of the lake. Water quality discharges from Lake Hancock have caused water quality impacts in the Peace River system as far south as Charlotte Harbor. Based on monitoring data collected by Polk County since 1984, Lake Hancock has consistently exhibited hypereutrophic conditions during the past 20 years, with calculated trophic state index (TSI) values ranging from 70 to over 100.

3.1 CURRENT WATER QUALITY CHARACTERISTICS

As a part of this Lake Hancock Outfall Treatment Project study, a 12-month surface water monitoring program was initiated in March 2004. Between March 2004 and February 2005, surface water monitoring was performed at three locations shown in Figure 3.1-1 on March 5, April 15, May 18, July 9, July 20, August 6, August 20, September 9, September 28, and October 25, November 30, December 28, 2004, January 31 and February 28, 2005. Surface water monitoring was not performed during June 2004 due to massive floating vegetation islands blocking access to the lake. During each surface water monitoring event, physical-chemical profiles were collected in Lake Hancock at each of the three surface water monitoring sites. Water samples were collected at a depth of 0.5 m and analyzed for a variety of physical and chemical characteristics.

Diurnal monitoring was performed at Site 2 in Lake Hancock and at Structure P-11 on April 15-16, July 20-21, October 25-26, 2004, and January 31-Febuary 1, 2005. During each diurnal monitoring event, physical-chemical profiles were collected at Site 2 and at Structure P-11 approximately every 2 hours for 24 hours. Water samples were collected at a depth of 0.5 m from the top of the water column and from 0.5 m above the lake bottom approximately every 6 hours for 24 hours.

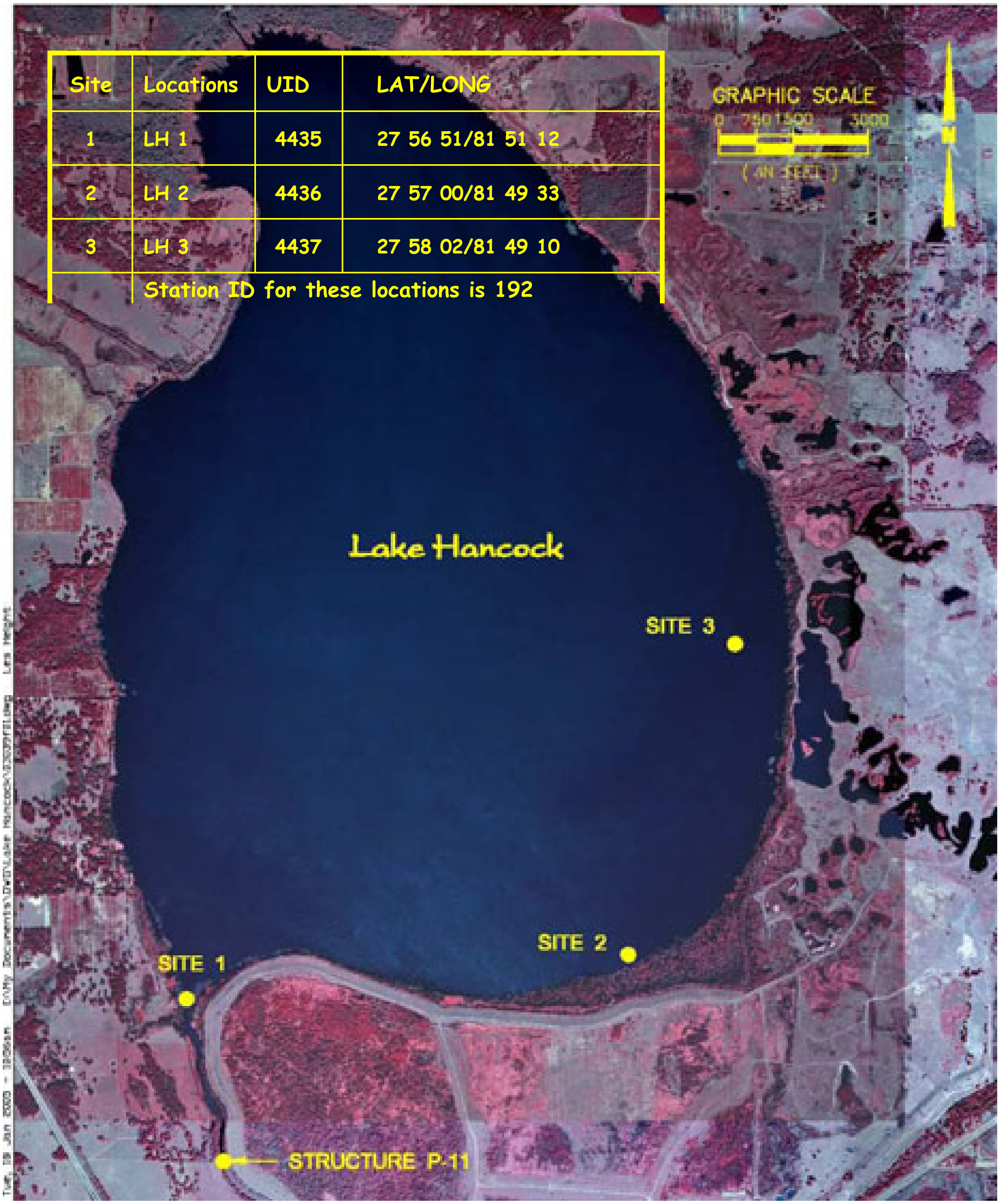

Figure 3.1-1 Surface Water Monitoring Locations for the Lake Hancock Outfall Treatment Project

3.1.1 Field Measurements

Physical-chemical profiles collected in Lake Hancock were found to be relatively similar between each of the three monitoring locations during the monitoring period. Although specific measurements of temperature, pH, and dissolved oxygen vary slightly between the three monitoring sites, the same general trends of increasing or decreasing values with increasing water depth were observed at each individual monitoring site on a specific monitoring date. A complete listing of physical-chemical profiles collected in Lake Hancock from March 2004-Febuary 2005 is given in Appendix A of the of the separate report, *"Physical and Chemical Characterization of Lake Hancock Surface Water"* (ERD, 2005). A summary of field-measured values of pH, conductivity, temperature, dissolved oxygen, oxidation-reduction potential (ORP), and Secchi disk depth is given in Table 3.1-1. Values summarized in this table reflect measurements performed at a depth of 0.5 m at each of the three monitoring sites indicated on Figure 3.1-1. Elevated pH and dissolved oxygen values are a result of excessive biological activity near the water surface, resulting in low Secchi disk depth values.

Table 3.1-1 Summary of Field Measured Characteristics in Lake Hancock from March 2004 to February 2005[1]

Parameter[2]	Units	Mean Value	Minimum Value	Maximum Value
pH	s.u.	8.94	6.65	10.21
Specific Conductivity	µmho/cm	212	169	256
Temperature	EC	24.64	16.46	31.52
Dissolved Oxygen	mg/L	11.4	2.5	> 20
ORP	mV	691	386	691
Secchi Disk Depth	m	0.19	0.09	0.38

1. n = 42 samples
2. Measured at depth = 0.5 m

Diurnal field monitoring performed in Lake Hancock at Site 2 and at Structure P-11 in April, July, October 2004, and January 2005. During these monitoring events the mean daily discharge at the P-11 structure were noted as: 0.01, 61, 249 and 2.1 cfs respectively, and at higher flows appear to have impacted the characteristics of samples collected upstream of P-11. Variations in temperature, pH, dissolved oxygen, dissolved oxygen percent saturation, turbidity, total suspended solids, color, ammonia, particulate nitrogen, Soluble Reactive Phosphorus, chlorophyll-a, BOD and COD were observed with respect to time to at least some degree during all diurnal monitoring events. Diurnal variations can be attributed to photosynthesis, aerobic respiration, and wind driven resuspension. In addition to diurnal changes, variability as a function of stratification within the water column seems to be a function of discharge from the lake, particularly at the Structure P-11 sampling site. Complete data sets for field monitoring of diurnal events is given in Appendix C of the separate report, *"Physical and Chemical Characterization of Lake Hancock Surface Water"* (ERD, 2005)

3.1.2 Laboratory Parameters

Surface water samples were collected at each of the three monitoring sites in Lake Hancock during each of the 14 monitoring events from March 2004-February 2005. A complete listing of laboratory analyses performed on surface water samples collected at each of the three monitoring sites is given in Appendix B and D of the separate report, *"Physical and Chemical Characterization of Lake Hancock Surface Water"* (ERD, 2005). A summary of laboratory measured mean water quality characteristics in Lake Hancock from March 2004-Febuary 2005 is given in Table 3.1-2. Values listed in this table represent the mean of all sites on all sampling dates. Similar to the trends exhibited by field measured parameters in Table 3.1-1, a high degree of variability is also apparent in laboratory measured characteristics. Differences between minimum and maximum values for many parameters, such as ammonia, NO_x, dissolved organic nitrogen, particulate nitrogen, soluble reactive phosphorus (SRP), dissolved organic phosphorus, particulate phosphorus, turbidity, total suspended solids, BOD, color, chlorophyll-a, and COD were 1-2 orders of magnitude. Extreme variability in water quality parameters is a common characteristic of hypereutrophic systems.

The dominant nitrogen species in Lake Hancock is particulate nitrogen, representing nitrogen incorporated into algal biomass. On an average basis, particulate nitrogen represents approximately 67% of the total nitrogen measured in the lake. Dissolved organic nitrogen comprises approximately 27% of the total nitrogen within the lake. Total nitrogen concentrations in Lake Hancock ranged from 1414-7090 μg/L, with an overall mean of 3865 μg/L. Values in this range are extremely elevated compared with total nitrogen concentrations typically observed in urban lake systems. A large portion of the total nitrogen measured in the lake during periods of sustained wind activity is particulate matter that has been resuspended into the water column as a result of wind activity.

Mean orthophosphorus concentrations in Lake Hancock were found to be highly variable, ranging from <1 to 374 μg/L. The overall mean soluble reactive phosphorus concentration of 106 μg/L is elevated in value and suggests an abundance of inorganic phosphorus species within the lake, particularly in comparison to the scarcity of inorganic nitrogen species. The dominant phosphorus species in Lake Hancock is clearly particulate phosphorus, which comprises more than 71% of the total phosphorus in the lake. Particulate phosphorus observed in the lake is a result of excess algal biomass along with resuspended sediment material. Total phosphorus concentrations in Lake Hancock were found to be highly variable, ranging from 113-716 μg/L. The overall mean total phosphorus concentration of 447 μg/L is extremely elevated and places Lake Hancock in the 95-99 percentile for lake systems within the State of Florida with respect to total phosphorus concentrations.

Table 3.1-2 Mean Laboratory Measured Water Quality Characteristics in Lake Hancock from March 2004 to February 2005[1]

Parameter[2]	Units	Mean Value	Minimum Value	Maximum Value
Alkalinity	mg/L	62.7	33.7	82.8
NH_3-N	µg/L	174	10	1613
$NO_2 + NO_3$-N	µg/L	25	< 5	237
Diss. Organic N	µg/L	1052	115	3237
Particulate N	µg/L	2614	192	5544
Total Nitrogen	µg/L	3865	1414	7090
SRP	µg/L	106	<1	374
Diss. Organic P	µg/L	20	4	62
Particulate P	µg/L	321	90	691
Total Phosphorus	µg/L	447	113	716
Turbidity	NTU	38.9	9.6	122
TSS	mg/L	63.5	13.3	164
BOD	mg/L	13.9	4.6	27.4
Color	Pt-Co	89	38	267
Chlorophyll-a	mg/m^3	339	60.7	800
Calcium	mg/L	22.6	16.5	27.9
Chloride	mg/L	15.5	3.9	22.9
COD	mg/L	129	51	294

1. n = 42 samples
2. Field measured at a depth of 0.5 m

Measured turbidity levels in Lake Hancock were found to be extremely variable between the individual monitoring dates. Mean values for turbidity at the three monitoring sites ranged from a low of 9.6 NTU to a high of 122 NTU, with an overall mean of 38.9 NTU. These turbidity values are significantly greater than measurements typically observed in urban lake systems. The increased turbidity in Lake Hancock is a direct result of the tremendous amount of algal biomass within the lake along with the resuspended inorganic sediment particles. The overall mean turbidity value of 38.9 NTU is substantially greater than the Class III surface water criterion for turbidity of 29 NTU, outlined in Chapter 62-302 of the Florida Administrative Code (FAC).

Levels of total suspended solids (TSS) in Lake Hancock also appear to be extremely elevated as well as highly variable, with measured concentrations ranging from 13.3-164 mg/L. The overall mean TSS concentration of 63.5 mg/L is extremely elevated for an urban lake system, which typically has TSS concentrations less than 10 mg/L. The elevated TSS levels observed in Lake Hancock are also a direct result of algal biomass and resuspended sediment matter.

Similar to the trends observed for turbidity and TSS, measured concentrations of BOD in Lake Hancock also appear to be extremely elevated as well as highly variable, with measured concentrations ranging from 4.6-27.4 mg/L. The overall mean BOD value of 13.9 mg/L is extremely elevated for an urban lake system and represents a continuous oxygen demand within the lake which must be continuously satisfied. Typical BOD concentrations in the range of values measured in Lake Hancock can quickly create oxygen depletion in the water column when algal production and primary productivity become more restricted.

In general, extremely elevated chlorophyll-a levels were observed in Lake Hancock on each of the individual monitoring dates. Mean whole lake chlorophyll-a values ranged from 60.7-800 mg/m^3, with an overall mean of 339 mg/m^3. The tremendous rate of algal production within Lake Hancock depends upon continuous nutrient inputs which are necessary to support and sustain the high rate of algal growth.

3.1.3 Water Quality Characteristics at Structure P-11

As indicated previously, surface water monitoring was performed on April 15-16, July 20-21, October 25-26, 2004 and January 31-Febuary 1, 2005 at Structure P-11. On these dates, diurnal monitoring was performed which included physical-chemical profiles approximately every 2 hours, and collection of a water sample 0.5 m from the water surface and 0.5 m from the creek bottom approximately every 6 hours. A summary of measured water quality characteristics at Structure P-11 on these four monitoring events is provided in Table 3.1-3. In general, water quality characteristics at Structure P-11 were found to be relatively similar to water quality characteristics measured in Lake Hancock. Extremely elevated values were observed for dissolved organic nitrogen, particulate nitrogen, total nitrogen, SRP, particulate phosphorus, total phosphorus, turbidity, TSS, BOD, and chlorophyll-a.

A summary of measured total nitrogen concentrations in Lake Hancock and at Structure P-11 from March 2004-Febuary 2005 is provided in Table 3.1-4. During this period, total nitrogen was measured at three locations in Lake Hancock during 14 surface water monitoring events. Total nitrogen was measured at Structure P-11 on four monitoring events at the top and bottom of the water column. The mean total nitrogen concentration in Lake Hancock on April 15, July 20, October 25, 2004 and January 30, 2005 of 3970 μg/L was slightly lower than the mean total nitrogen concentration measured near the top of the water column on the same dates at Structure P-11 of 4196 μg/L. This is approximately half of the 13% difference in total nitrogen concentration observed between these sites measured during October 1998-July 1999. (ERD, 1999).

A summary of measured particulate nitrogen concentrations in Lake Hancock and at Structure P-11 from March 2004 to January 2005 is provided in Table 3.1-5. The mean in-lake particulate nitrogen concentration of 2734 μg/L is approximately 6% higher than the mean particulate nitrogen concentration measured at Structure P-11 on the same dates. Again, this is contrary to the 18% difference measured during October 1998-July 1999 (ERD, 1999).

Table 3.1-3 Summary of Measured Water Quality Characteristics at Structure P-11 on April 15-16, July 20-21, October 25-26, 2004 and January 30-31,2005 [1]

Parameter	Units	Mean Value	Minimum Value	Maximum Value
pH[2]	s.u.	8.61	7.29	9.67
Specific Conductivity[2]	µmho/cm	199	154	236
Temperature[2]	EC	22.98	16.79	27.52
Dissolved Oxygen[2]	mg/L	8.53	1.05	17.82
ORP[2]	mV	605	545	678
Alkalinity[2]	mg/L	--	--	--
NH_3-N[3]	µg/L	136	51	235
$NO_2 + NO_3$-N[3]	µg/L	69	<5	296
Diss. Organic N[3]	µg/L	1417	837	2216
Particulate N[3]	µg/L	2574	784	5001
Total Nitrogen[3]	µg/L	4196	2180	6801
SRP[3]	µg/L	104	1	375
Diss. Organic P[3]	µg/L	22	17	31
Particulate P[3]	µg/L	888	164	2226
Total Phosphorus[3]	µg/L	1034	567	2333
Turbidity[3]	NTU	60.4	13.6	138
TSS[3]	mg/L	87	17	212
BOD[3]	mg/L	16.2	7.3	26
Color[3]	Pt-Co	76	37	147
Chlorophyll-a[3]	mg/m^3	334	123	530
Calcium[3]	mg/L	23.5	17.8	28.5
Chloride[3]	mg/L	17.3	11.8	22.5
COD[3]	mg/L	135	60	300

1. Sample depth = 0.5 m
2. n=46
3. n=17

Table 3.1-4 Summary of Measured Total Nitrogen Concentrations in Lake Hancock/P-11 Structure from 3/04 to 10/04[1]

ERD Sampling Date	Total Nitrogen Concentration (µg/L)			
	Lake Hancock			P-11
	Site 1	Site 2	Site 3	
3/5/2004	3633	2769	2497	--
4/15/2004	3743	3479	4429	3587
5/18/2004	4978	5605	5591	--
7/9/2004	7090	6398	5488	--
7/20/2004	4236	4715	5293	6525
8/6/2004	4777	4436	5050	--
8/20/2004	2386	2291	1414	--
9/9/2004	2457	2393	3518	--
9/28/2004	3958	3737	2372	--
10/25/2004	2448	2892	2309	2526
11/30/2004	4249	3169	3028	
12/28/2004	3923	3873	3642	
1/31/2005	4803	4687	4600	4297
2/28/2005	4463	3319	2194	
Mean: All Dates	4082	3840	3673	--
Mean: 4/15/04, 07/20/04, 10/25/04,1/31/05	3808	3943	4158	4234
Mean: 3 sites	3970			

1. Sample depth = 0.5 m

Table 3.1-5 Summary of Measured Particulate Nitrogen Concentrations in Lake Hancock/ P-11 Structure from 3/04 to 10/04[1]

ERD Sampling Date	Particulate Nitrogen Concentration (µg/L)			
	Lake Hancock			P-11
	Site 1	Site 2	Site 3	
3/5/2004	2423	1570	1469	--
4/15/2004	2393	1693	2404	1428
5/18/2004	3513	3922	4384	--
7/9/2004	5544	4713	4265	--
7/20/2004	3282	3288	3918	4786
8/6/2004	3752	3358	3945	--
8/20/2004	1590	1511	680	--
9/9/2004	1595	1515	2731	--
9/28/2004	1734	192	555	--
10/25/2004	1612	1668	1418	1373
11/30/2004	3144	2007	1887	
12/28/2004	2921	2900	2683	
1/31/2005	3951	3580	3595	2995
2/28/2005	3335	2396	759	
Mean: All Dates	2914	2451	2478	--
Mean: 4/15/04, 07/20/04, 10/25/04, 1/31/05	2810	2557	2834	2646
Mean: 3 sites	2734			

1. Sample depth = 0.5 m

It is important to note that the slightly higher total nitrogen and particulate nitrogen concentrations measured at Structure P-11 are based on only four data points. In addition, the higher mean total nitrogen and particulate nitrogen concentrations at Structure P-11 are a result of higher total nitrogen and particulate nitrogen concentrations on only one date (July 20, 2004). This monitoring event occurred during significant flows through Saddle Creek and Structure P-11. These values may not be representative of average annual values.

Similar summaries for measured total phosphorus and particulate phosphorus concentrations in Lake Hancock and at Structure P-11 are provided in Tables 3.1-6 and 3.1-7, respectively. Total phosphorus and particulate phosphorus concentrations were significantly higher at Structure P-11 than Lake Hancock during the April 15 and July 20, 2004 monitoring events. The overall mean total phosphorus and particulate phosphorus concentrations at Structure P-11 are more than twice the mean concentrations in Lake Hancock. The approximately two-fold increase in total phosphorus and particulate phosphorus concentrations measured at Structure P-11 during 2004 are contrary to the 6% total phosphorus and 4% particulate phosphorus reduction from the lake to Structure P-11 observed by ERD during monitoring performed in 1998-1999.

Table 3.1-6 Summary of Measured Total Phosphorus Concentrations in Lake Hancock/P-11 Structure from 3/04 to 2/05[1]

ERD Sampling Date	Total Phosphorus Concentration (µg/L)			
	Lake Hancock			P-11
	Site 1	Site 2	Site 3	
3/5/2004	268	197	237	--
4/15/2004	691	500	592	1004
5/18/2004	474	534	578	--
7/9/2004	716	627	509	--
7/20/2004	569	442	420	1848
8/6/2004	375	382	377	--
8/20/2004	153	133	113	--
9/9/2004	294	287	279	--
9/28/2004	655	630	497	--
10/25/2004	595	612	584	641
11/30/2004	518	530	468	
12/28/2004	536	555	529	
1/31/2005	544	545	530	650
2/28/2005	287	259	217	
Mean: All Dates	477	464	438	--
Mean: 4/15/04, 07/20/04, 10/25/04,1/31/05	600	525	532	1036
Mean: 3 sites	552			

1. Sample depth = 0.5 m

Table 3.1-7 Summary of Measured Particulate Phosphorus Concentrations in Lake Hancock/P-11 Structure from 3/04 to 2/05[1]

ERD Sampling Date	Particulate Phosphorus Concentration (µg/L)			
	Lake Hancock			P-11
	Site 1	Site 2	Site 3	
3/5/2004	250	180	219	--
4/15/2004	670	481	569	981
5/18/2004	458	519	562	--
7/9/2004	691	604	489	--
7/20/2004	562	434	412	1778
8/6/2004	356	367	362	--
8/20/2004	140	121	97	--
9/9/2004	146	163	126	--
9/28/2004	236	217	90	--
10/25/2004	192	206	192	222
11/30/2004	291	326	194	
12/28/2004	211	237	183	
1/31/2005	475	470	397	549
2/28/2005	268	240	91	
Mean: All Dates	361	337	290	--
Mean: 4/15/04, 07/20/04, 10/25/04,1/31/05	475	398	393	883
Mean: 3 sites	422			

1. Sample depth = 0.5 m

3.1.4 Total Nitrogen Particulate Size Distribution

To further quantify the size of the particulate forms of total nitrogen, samples collected on April 23, July 26, December 23, 2004 and February 2, 2005 at Site 2 and P-11 were filtered through 180, 140, 100, 60, 30, and 11 um filters and total nitrogen was measured on the resulting filtrate. Table 3.1-8 summarizes the results. Based on results, approximately 90 percent of the total nitrogen is associated with particles that are less than 11 µm in size.

Table 3.1-8 Summary of Particulate Nitrogen Size Distribution Measured in Lake Hancock and at Structure P-11

Lake Hancock (Site 2)

Particle Size	Range		Average	
	Min	Max	Mean	Percent Finer (%)
>180 um	16	100	55	0
140 um	8	63	31	99.00
100 um	20	79	53	98.43
60 um	46	156	102	97.46
30 um	49	186	98	95.60
11 um	200	1027	433	93.81
< 11 um	1,885	6961	4,708	85.91
Total	---	--	**5,480**	

Structure P-11

Particle Size	Range		Average	
	Min	Max	Mean	Percent Finer (%)
>180 um	37	107	62	0
140 um	4	26	15	98.98
100 um	10	70	35	98.74
60 um	13	177	104	98.16
30 um	93	188	107	96.45
11 um	175	808	488	94.70
< 11 um	2,664	10509	5,279	**86.68**
Total	---	--	**6,090**	

3.2 HISTORICAL WATER QUALITY CHARACTERISTICS

The primary objective of the Lake Hancock Outfall Treatment Project is to reduce total nitrogen loads discharging from Lake Hancock through Structure P-11 to Lower Saddle Creek. As a matter of comparison, historic total nitrogen and total phosphorus concentration data for Lake Hancock and Structure P-11 were obtained from Polk County, STORET, and the U.S. Geological Survey (USGS). In all, 182 total nitrogen data points and 203 total phosphorus data points were obtained for Lake Hancock. A total of 78 total nitrogen and total phosphorus concentration data points was obtained for Structure P-11. A complete listing of the total nitrogen and total phosphorus data is presented in Appendix A. A summary of mean total nitrogen and mean total phosphorus concentrations measured in Lake Hancock and at Structure

P-11 from various data sources is provided in Table 3.2-1. In Lake Hancock, the overall mean total nitrogen concentration is 5.530 mg/L and the mean total phosphorus concentrations is 0.603 mg/L. At Structure P-11, the overall mean total nitrogen concentration is 5.240 mg/L and the mean total phosphorus concentration is 0.654 mg/L.

Table 3.2-1 Summary of Mean Total Nitrogen and Mean Total Phosphorus Concentrations Measured in Lake Hancock and at Structure P-11

Lake Hancock

Parameter	Data Source				
	ERD	Polk County	Storet	USGS	All
Number of TN Samples	19	46	66	51	182
Mean TN Value (mg/L)	4.867	5.449	5.556	5.816	5.530
Number of TP Samples	19	46	87	51	203
Mean TP Value (mg/L)	0.471	0.555	0.604	0.691	0.603

Structure P-11

Parameter	Data Source				
	ERD	Polk County	Storet	USGS	All
Number of TN Samples	12	0	15	51	78
Mean TN Value (mg/L)	4.861	--	3.586	5.816	5.240
Number of TP Samples	12	0	15	51	78
Mean TP Value (mg/L)	0.748	--	0.455	0.691	0.654

A statistical comparison of total nitrogen and total phosphorus concentrations measured in Lake Hancock by various sources is provided in Figure 3.2-1 in the form of whisker box plots. The bottom line of the box portion of each plot represents the lower quartile, with 25% of the data points lying beneath the box. The upper line of the box represents the 75% upper quartile, with 25% of the data lying above the box. The horizontal line drawn within each box represents the median value where 50% of the data lies above and below this value. For the lines drawn above and below the box, these encompass between 5% and 95% of the data, respectively.

For total nitrogen, little variation is observed in the 25-75% quartiles and median values for any data sources shown in Figure 3.2-1. For total phosphorus, the median values are similar, but the ERD 25-75% quartiles are slightly lower than other data sources. In general, the box and whisker plots show the data collected during this study (i.e., ERD) to be in relative good agreement with the other data sources.

A similar plot of total nitrogen and total phosphorus concentrations measured at Structure P-11 from various data sources is provided in Figure 3.2-2. The total nitrogen and total phosphorus concentrations measured at Structure P-11 by various sources are more variable than the data gathered in Lake Hancock. This is particularly true of the total nitrogen data.

Based on careful review of the data, a **total nitrogen concentration of 5.530 mg/L** and a **total phosphorus concentration of 0.603 mg/L** were selected as representative of the mean annual water quality characteristics discharging from Lake Hancock to Lower Saddle Creek and used as a basis for calculating treatment efficiency and load reductions by the different treatment alternatives evaluated.

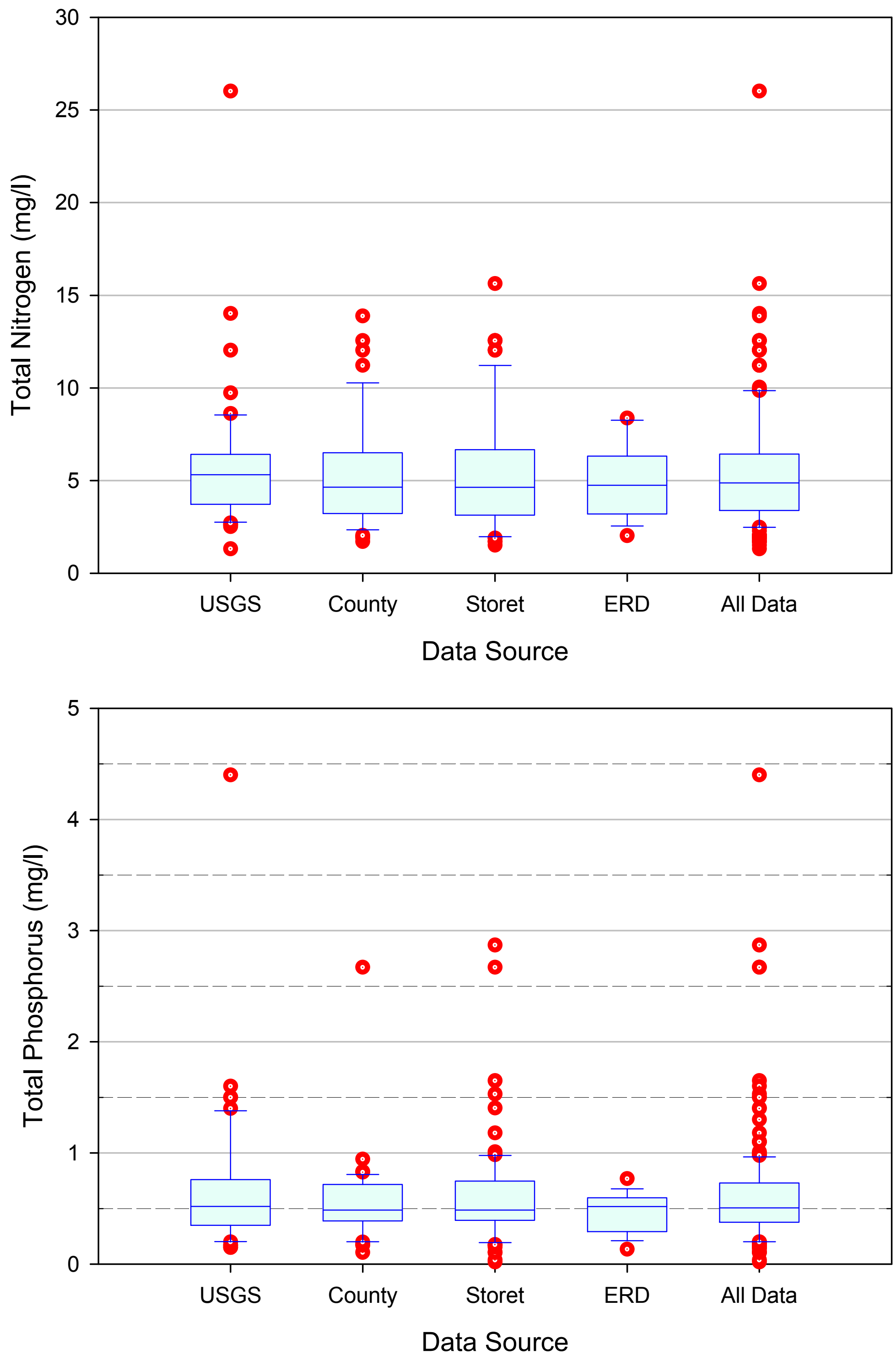

Figure 3.2-1 Summary of Total Nitrogen and Total Phosphorus Concentrations Measured in Lake Hancock from Various Data Sources.

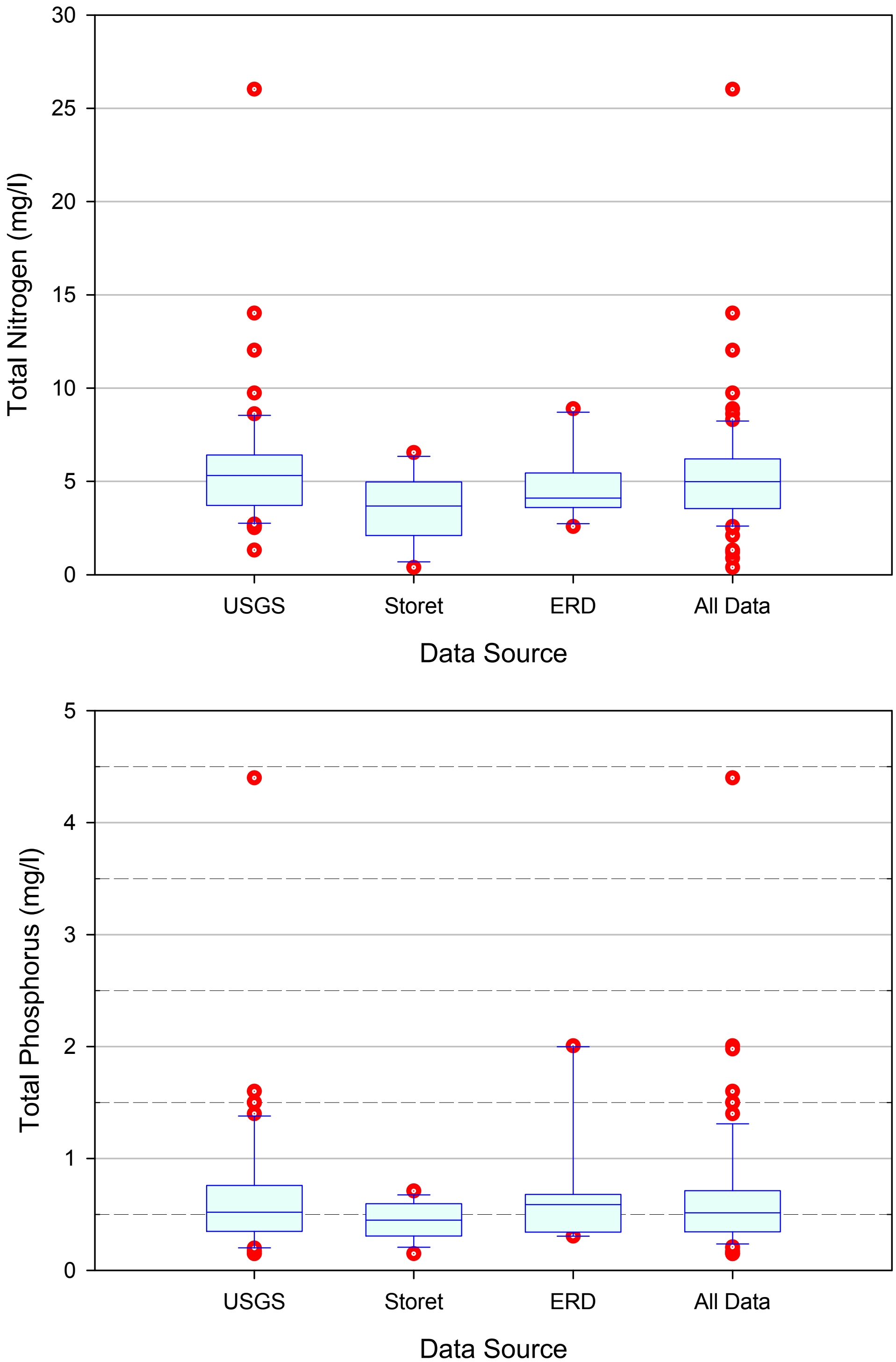

Figure 3.2-2 Summary of Total Nitrogen and Total Phosphorus Concentrations Measured at Structure P-11 from Various Data Sources

SECTION 4

BENCH-SCALE TESTING RESULTS

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SECTION 4.0

BENCH-SCALE TESTING RESULTS

4.1 LABORATORY TESTING PROCEDURES

Laboratory jar tests were conducted to determine the effects of chemical coagulation followed by sedimentation on the reduction of total nitrogen on water samples collected from Site 2 in Lake Hancock and immediately upstream of Structure P-11 on September 17, September 28, and October 25, 2004. The collected water samples were dosed with liquid aluminum sulfate (alum) at doses of 2.5, 5, and 7.5 mg Al/liter. Water samples were also treated with 5 mg Al/liter (Hyperion 1090) and 5 mg Al/liter (Hyperion 4090). Hyperion 1090 and Hyperion 4090 are polyaluminum chloride blends with a proprietary polymer manufactured by General Chemical.

Laboratory testing at each of the chemical doses was conducted individually using a sample volume of approximately 2 liters for each test. To begin a test, the appropriate volume of coagulant was added to a 2-liter water sample and vigorously mixed for approximately 60 seconds. Each sample was then divided into four sub-samples and treated as follows:

1. Five minutes (t = 5 min) following mixing, a portion of the supernatant was decanted from the sample and filtered through a 0.45-micron membrane filter and measured for inorganics, nutrients, suspended solids, and dissolved aluminum. Filtering the sample through the 0.45-micron membrane filter was used in this case, to represent the water quality that might result from direct filtration by sand media filters.

2. Three hours (t = 3 hrs) following mixing and settling, a second portion of the supernatant was decanted from the sample and measured for inorganics, nutrients, suspended solids, and dissolved aluminum. The results of this sample represent the water quality that might be expected from settling in a sedimentation pond or basin with a 3-hour detention time. Filtering of the supernatant was not performed at this time interval.

3. Twenty-four hours (t = 24 hrs) following mixing and settling, a third portion of the supernatant was decanted from the sample and analyzed for inorganics, nutrients, suspended solids, and dissolved aluminum. The results of this sample represent the water quality that might be expected from settling in a sedimentation pond with a 24-hour detention time.

4. Twenty four hours (t = 24 hrs) following mixing and settling, a fourth portion of the supernatant was decanted and filtered through a 0.45-micron membrane filter and measured for inorganics, nutrients, suspended solids, and dissolved aluminum. Filtering the sample through the 0.45-micron membrane filter was used in this case, to represent the water quality that might result from filtration by sand media filters.

4.2 RESULTS OF LABORATORY JAR TESTING

Results of laboratory jar tests are presented in Appendix B. Chemical coagulation of water samples resulted in reductions for virtually all measured parameters for all water samples.

Substantial reductions were observed for particulate nitrogen, soluble reactive phosphorus (SRP), particulate phosphorus, total phosphorus, turbidity, total suspended solids (TSS), biochemical oxygen demand (BOD), and chlorophyll-a. A summary of total nitrogen removal efficiencies is provided in Table 4.2-1.

At Lake Hancock Site 2, all four treatment techniques, including settling for 3 hours, settling for 24 hours, settling 5 minutes and filtering, and settling 24 hours and filtering, provided similar total nitrogen removal efficiencies. At an alum dose of 2.5 mg Al/liter, total nitrogen removal efficiencies ranged from 2-69%. At an alum dose of 5 mg Al/liter, total nitrogen removal efficiencies ranged from 3-75%, and at an alum dose of 7.5 mg Al/liter, total nitrogen removal efficiencies ranged from 3-81%. Total nitrogen removal efficiencies for Hyperion 1090 and 4090 (5 mg Al/liter) ranged from <0-76% and <0-77%, respectively.

Table 4.2-1 Summary of Total Nitrogen Removal Efficiencies for Jar Tests Performed on Water Samples Collected in Lake Hancock from 9/04 to 10/04

Site	Sample Description	Sample Collection Date	Raw Water Particulate N: Total N	Treatment				
				Alum			5 mg Al/liter 1090	5 mg Al/liter 4090
				2.5 mg Al/liter	5.0 mg Al/liter	7.5 mg Al/liter		
2	Settled 3 hours	9/17/04	0.75	55	73	77	72	71
		9/28/04	0.11	2	9	8	1	0
		10/25/04	0.34	17	30	46	30	31
	Settled 24 hours	9/17/04	0.75	66	75	78	76	77
		9/28/04	0.11	12	13	16	9	9
		10/25/04	0.34	21	32	47	38	38
	Settled 5 minutes/ filtered	9/17/04	0.75	68	73	77	72	71
		9/28/04	0.11	10	13	19	< 0	< 0
		10/25/04	0.34	30	39	57	12	30
	Settled 24 hours/filtered	9/17/04	0.75	69	73	81	78	75
		9/28/04	0.11	2	3	3	< 0	1
		10/25/04	0.34	19	23	35	18	15
P-11	Settled 3 hours	9/17/04	0.66	35	64	69	64	62
		9/28/04	0.27	18	26	30	30	15
		10/25/04	0.77	45	53	60	55	55
	Settled 24 hours	9/17/04	0.66	59	62	68	60	64
		9/28/04	0.27	21	22	25	28	27
		10/25/04	0.77	59	68	78	55	64
	Settled 5 minutes/ filtered	9/17/04	0.66	64	65	71	62	65
		9/28/04	0.27	16	25	41	30	15
		10/25/04	0.77	24	24	26	18	24
	Settled 24 hours/filtered	9/17/04	0.66	40	44	50	33	52
		9/28/04	0.27	22	20	21	24	24
		10/25/04	0.77	53	69	81	66	64

At Structure P-11, all four treatment techniques provided similar results with the exception of the September 17, 2004 water sample. There is no explanation for this anomaly. At alum doses of 2.5 mg Al/liter, 5 mg Al/liter, and 7.5 mg Al/liter, total nitrogen removal efficiencies ranged from 16-64%, 20-69%, and 21-81%, respectively. Total nitrogen removal efficiencies for Hyperion 1090 and 4090 (5 mg Al/liter) ranged from 18-66% and 15-65%, respectively.

Based on the results provided in Table 4.2-1, it is clear that total nitrogen removal efficiencies are directly related to the raw water particulate nitrogen to total nitrogen ratio (PN:TN). This was expected since coagulation is known to remove primarily particulate forms of nitrogen. On September 17, 2004, the raw water sample from Site 2 had a PN:TN of 0.75, which resulted in 77-81% total nitrogen removal efficiencies for the various treatment methods at an alum dose of 7.5 mg Al/liter. Conversely, the raw water sample from Site 2 on September 28, 2004 had a PN:TN of 0.11, with total nitrogen removal efficiencies ranging from 3-19% at an alum dose of 7.5 mg Al/liter. This was also observed for the October 25, 2004 sample, with a PN:TN of 0.34. The resulting total nitrogen removal efficiencies at an alum dose of 7.5 mg Al/liter were in the range of the ratio for this sample, with the exception of the sample settled for 5 minutes and filtered which had a 57% total nitrogen removal efficiency. These trends were also apparent for jar tests conducted on water collected upstream of Structure P-11.

In general, at a dose of 7.5 mg Al/liter of alum, the total nitrogen removal efficiency was slightly higher than the PN:TN. Removal efficiencies after 24 hours of settling were approximately the same as the removal efficiencies after 3 hours of settling. Thus, a vast majority of floc settles within 3 hours of coagulant addition. The use of Hyperion 1090 and Hyperion 4090 provided no significant improvement in total nitrogen removal efficiency. These proprietary chemicals are approximately twice the cost of liquid aluminum sulfate on an aluminum mass basis and thus based on costs alone, would not justify their use in this case.

A summary of particulate nitrogen removal efficiencies is provided in Table 4.2-2. Alum addition at a dose of 7.5 mg Al/liter provided significantly higher removal efficiencies than an alum dose of 2.5 or 5.0 mg Al/liter. At Site 2, a mean particulate nitrogen removal efficiency of 81% was observed with an alum dose of 7.5 mg Al/liter after 3 hours of settling. The removal efficiency was lower than the sample settled for 24 hours because of the lower raw water concentration measured on 9/28/04. When additional water samples were analyzed, the mean particulate nitrogen removal efficiency at 3 hours was expected to be similar to the mean particulate nitrogen removal efficiency at 24 hours. The mean removal efficiency at that alum dose increased to 92% after settling for 24 hours. At Structure P-11, an alum dose of 7.5 mg Al/liter achieved a mean particulate removal efficiency of 89% after 3 hours of settling and 90% after 24 hours of settling. At Structure P-11, settling of alum floc was apparently complete within 3 hours. The addition of Hyperion 1090 or Hyperion 4090 did not significantly improve particulate nitrogen removal efficiencies and thus, based on costs alone, would not justify their use in this case.

As shown in Tables 4.2-1 and 4.2-2, the addition of alum at a dose of 7.5 mg Al/liter and settling for 3 hours and settling for 5 minutes followed by filtration were effective in reducing particulate nitrogen at both Lake Hancock Site 2 and at Structure P-11. Removal efficiencies for total nitrogen and particulate nitrogen were not significantly higher for samples settled 24 hours or for samples treated with Hyperion 1090 or Hyperion 4090. Based on these test results, an **alum dose of 7.5 mg Al/liter** is recommended for developing conceptual designs using chemical coagulation.

Table 4.2-2 Summary of Particulate Nitrogen Removal Efficiencies for Laboratory Jar Tests Conducted on Water Samples Collected in Lake Hancock from 9/04 to 10/04

Site	Sample Description	Sample Collection Date	Treatment				
			Alum			5 mg Al/ liter 1090	5 mg Al/ liter 4090
			2.5 mg Al/liter	5.0 mg Al/liter	7.5 mg Al/liter		
2	Settled 3 hours	9/17/04	74	97	98	97	96
		9/28/04	< 0	15	54	< 0	< 0
		10/25/04	41	63	92	94	60
		Mean	38	58	81	64	52
	Settled 24 hours	9/17/04	89	96	99	97	99
		9/28/04	25	69	89	88	9
		10/25/04	21	62	88	47	51
		Mean	55	76	92	77	53
P-11	Settled 3 hours	9/17/04	57	92	98	94	93
		9/28/04	56	81	87	94	93
		10/25/04	75	77	81	81	82
		Mean	63	83	89	90	89
	Settled 24 hours	9/17/04	93	94	98	91	91
		9/28/04	76	78	84	92	92
		10/25/04	74	80	87	66	78
		Mean	81	84	90	83	87

SECTION 5

AQUATIC PLANT-BASED TREATMENT TECHNOLOGIES

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SECTION 5.0

AQUATIC PLANT-BASED TREATMENT TECHNOLOGIES

The use of aquatic plant-based systems for water quality improvement has been an expanding field since the 1970s. There are over 600 such systems currently operating in the U.S. and over 6,000 in the world (IAWQ 2000; Cooper 1999; Vymazal et al., 1998; Haberl et al., 1997; Kadlec and Knight, 1996; Kadlec and Brix, 1995; Bavor and Mitchell, 1994; Cooper and Findlater, 1990; Hammer, 1989, Reddy et al., 1987). Many of these systems provide final treatment of municipal wastewaters prior to surface discharge. Other wastewater categories are also routinely treated with aquatic plant-based systems, including wastewaters from industrial, storm water, and agricultural sources. Considerable information concerning the design and operational performance of these systems has been accumulating over the last 30 years and has led to the development of a rapidly growing literature database available to those who are interested in applying this technology for water quality improvement. Under Florida's Everglades Construction Project alone, over 40,000 acres of large-scale aquatic plant-based treatment systems have been built (Wetland Solutions, 2004a). These systems continue to provide a wealth of performance results, operational information, and costing data.

5.1 APPLICABLE TREATMENT TECHNOLOGIES

The water-cleansing ability of aquatic plant-based systems was discovered in research conducted in natural wetlands receiving pollutant discharges. Many of the early engineered wetland projects utilized naturally occurring swamps and marshes for tertiary treatment of pre-treated wastewaters. Natural wetlands are still used for wastewater treatment in geographical regions where they are abundant. However, many newer applications of the aquatic plant-based treatment technology utilize constructed systems. Environmental engineers and scientists have learned how to replicate the attributes of natural wetlands into constructed systems through control of water depth, flow gradients, and water control structures. Aquatic plant-based systems can also incorporate a variety of wetland plant species that provide desired benefits for water treatment and/or wildlife habitat.

As a result of the physical, biological, and chemical processes that take place in a vegetated aquatic environment, many pollutants in the water are transformed or inactivated (Figure 5.1-1). The low flow rate of the water, and the resulting long residence time (compared to conventional treatment methods); results in the settling and trapping of solids and associated pollutants, the sorption of soluble chemicals on soils and in microbial and plant tissues, and the transformation of reduced forms of carbon and nitrogen to harmless gases.

Soluble pollutants and the trapped particulates are degraded by heterotrophic bacteria and fungi within the wetland environment. High production of reduced carbon by wetland plants feeds this decomposition flywheel, effectively reducing concentrations of nearly all biodegradable organics, metals, and nutrients.

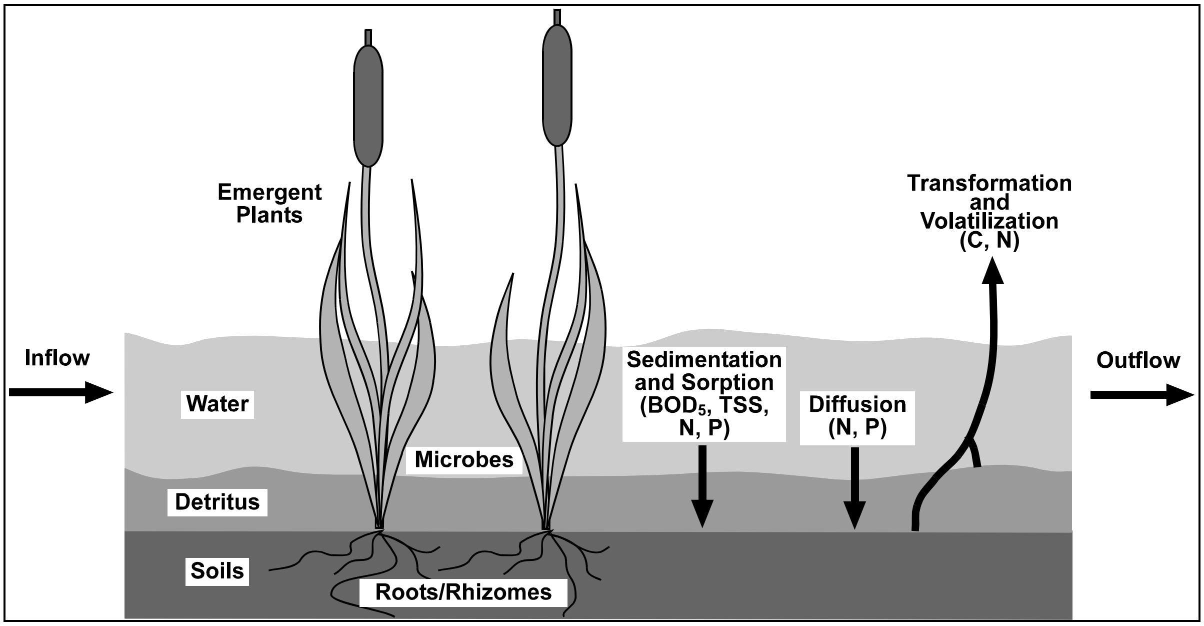


Figure 5.1-1 Aquatic Plant-Based System Processes Include Sedimentation, Chemical Sorption, and Microbial Transformations of Pollutants

In addition to their effectiveness for removal of soluble and particulate forms of organic carbon, treatment wetlands microbially transform nitrogen and thereby reduce discharge concentrations of this nutrient. Nitrogen takes several dominant forms in wetland and aquatic environments. The most common nitrogen species are organic nitrogen (Org-N), ammonia nitrogen (NH_4-N), and nitrate nitrogen (NO_3-N). Nitrite nitrogen (NO_2-N) is rarely detectable because it is rapidly transformed to NO_3-N. The sum of all nitrogen species is commonly reported as total nitrogen (TN).

A variety of nitrogen transformation processes occur in wetlands. The dominant transformations that occur in treatment wetlands are ammonification of Org-N to NH_4-N, nitrification of NH_4-N to NO_2-N and NO_3-N, and denitrification of NO_3-N to nitrogen gas (N_2). Other important transformations include fixation of atmospheric nitrogen and volatilization of dissolved NH_4-N.

Kadlec and Knight (1996) reported that the global median Org-N background concentration in wetlands ranges from about 1 to 1.5 milligrams per liter (mg/L). If Org-N exceeds background levels, then a net TN reduction requires that Org-N is first mineralized, and then subsequent removal of NH_4-N and NO_3-N occurs. Typical un-impacted wetlands exhibit NH_4-N and NO_3-N concentrations that are below normal analytical detection levels (Kadlec and Knight, 1996).

Phosphorus is sequestered in treatment wetlands through the production of chemically bound forms and their accretion in sediments over time. Metals and other insoluble pollutants are also slowly accumulated in the wetland sediments. Removal of these biogeochemically conservative elements is dependent upon the overall plant productivity of the wetland and the rate of formation of new wetland sediments. Accumulation of metals in wetland sediments does not create an environmental hazard as long as water concentrations are within regulatory standards.

Pretreatment to prevent the creation of hazards to wildlife can be an essential component of treatment wetland design.

Operational aquatic plant-based treatment systems range in complexity from natural marshes and forested wetlands to constructed wetlands (surface flow and subsurface flow) with limited operation and maintenance requirements, to intensively-managed installations such as water hyacinth and algal-based systems. Figure 5.1-2 shows the general types of aquatic plant-based treatment systems. The two categories of aquatic plant-based treatment systems that have the most relevance to the water quality conditions at Lake Hancock are surface-flow constructed wetlands and managed aquatic plant systems (MAPS). These types of systems are further described below.

5.1.1 Surface-Flow Constructed Wetlands

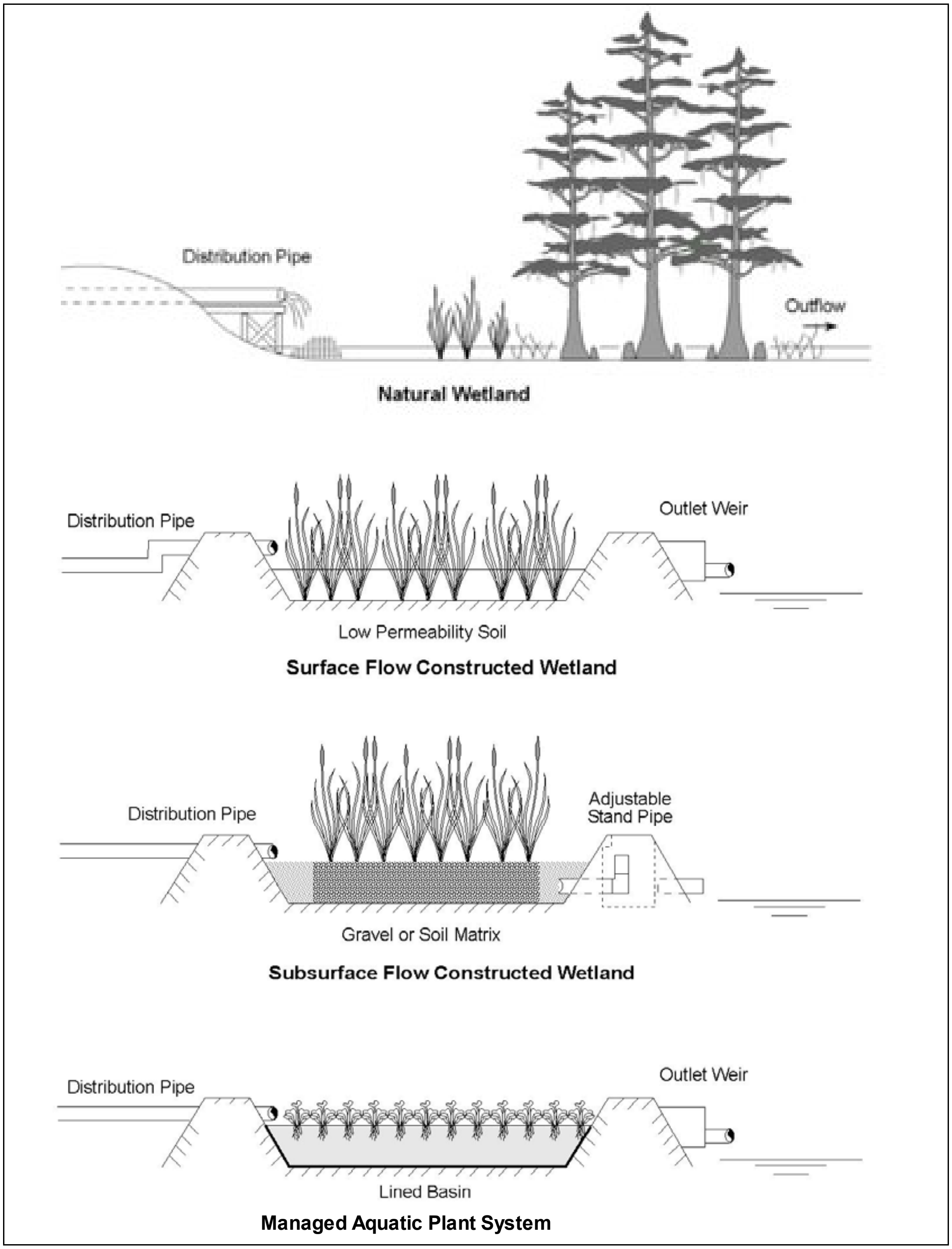
The term "surface-flow constructed wetlands" encompasses a wide range of possible configurations with varying design features and levels of complexity. In general, the term refers to a man-made wetland that is designed to operate with the water surface elevation above the wetland substrate grade. Applied water moves in a sheet flow fashion from inlet to outlet. The wetland vegetation is most often dominated by rooted emergent plants, but in some cases (either by design or by natural colonization), may include varying fractions of submerged aquatic vegetation (SAV), floating aquatic plants (FAP), or attached periphytic algae. In the literature, the term "surface flow constructed wetlands" is interchangeable with other terms such as treatment wetlands, constructed wetlands, stormwater treatment areas (STAs), marsh flow-ways, and filter marshes.

5.1.2 Managed Aquatic Plant Systems

MAPS use floating macrophyte plants in shallow to deep lagoons to treat various pollutants. Research with MAPS began in the 1970s and focused on reducing concentrations of biochemical oxygen demand (BOD), total suspended solids (TSS), nutrients (nitrogen and phosphorus), and metals that occur in municipal wastewaters. Because this technology was found to be well-suited for plant harvesting, MAPS have been used for enhanced nutrient removal.

Much of the research with MAPS focused on water hyacinths (*Eichhornia crassipes*) as the principal plant species. Other plant species that have been used in MAPS include pennywort (*Hydrocotyle* spp.) and duckweed (*Lemna* spp.). In recent years, MAPS research has expanded to the treatment of surface waters, and has focused on the use of filamentous algal species. Management concepts and harvesting systems have become increasingly complex, leading to the development of several patented processes for nutrient removal systems.

Figure 5.1-2 Types of Aquatic Plant-Based Treatment Systems (adapted from Kadlec and Knight, 1996)



5.2 SURFACE FLOW CONSTRUCTED WETLANDS CONCEPTUAL PLAN

This section presents a conceptual design and cost estimate for a surface flow constructed wetland that will receive water from Lake Hancock. This section includes information regarding operations and maintenance activities, estimated finished water quality, residuals management, and permitting requirements. The conceptual plan provided in this document builds upon experience gained and lessons learned at other large-scale surface-flow wetlands in Florida. Table 5.1-1 identifies these large-scale systems.

Table 5.1-1 Large-Scale Surface-Flow Constructed Wetlands in Florida

Site	Location	Area (ac)	Years of Operation
STA-1W	Palm Beach County	6,670	1/93 – present
STA-2	Palm Beach County	6,430	3/00 – present
STA-3/4	Palm Beach County	16,500	10/03 – present
STA-5	Hendry County	4,110	11/98 – present
STA-6	Hendry County	870	12/97 – present
Lakeland WTS	Polk County	1,400	1/87 – present
Orlando Easterly Wetlands	Orange County	1,200	1/88 – present
Blue Heron WTS	Brevard County	264	1/97 – present
Lake Apopka Marsh Flow-Way	Lake County	660	11/03 – present

5.2.1 General Description

The proposed full-scale surface flow constructed wetland system will consist of multiple cells that are separated by earthen levees. Water will be pumped from Lake Hancock to an inlet buffer cell that will be designed to allow for easy removal of settled algal solids, should solids management become necessary. Water will discharge by gravity from the buffer cell into several treatment cells. The final discharge of treated water will occur by gravity over a cascade aeration structure which will return water to Lower Saddle Creek, downstream from the existing P-11 structure, or optionally to Lake Hancock, depending upon operational needs.

In addition to improving water quality, a number of ancillary benefits are provided by surface flow constructed wetlands. These can include increasing wildlife habitat value and providing passive recreational opportunities (hiking and bird watching), educational opportunities (outdoor classrooms and university research), and active recreational opportunities (fishing and hunting).

5.2.2 Conceptual Design

Two annual total nitrogen load reduction targets were considered for treatment utilizing surface flow wetlands. The first load reduction alternative removes a targeted 45% of the existing total nitrogen load currently discharged downstream of the Lake Hancock discharge structure P-11. The proposed surface flow wetland design was selected to achieve the load reduction goal with a single pass through the treatment system. A modeling approach was developed to approximate the effect of operating a constructed wetland at a highly-dynamic loading rate with daily flows ranging from zero to 110 cfs. Based on the model results, up to 2,540 acres of wetlands are estimated to be required to meet this load reduction goal.

The second load reduction target alternative was developed based on the objective of treating up to 52 cfs of flow from Lake Hancock utilizing surface flow wetland technology. This design flow rate represents the estimated maximum discharge from the P-11 structure needed to meet Minimum Flow Levels (MFL) in the Peace River downstream at Ft. Meade as required by Florida Statute. This alternative consists of a buffer cell and three wetland cells totaling 1,095 acres that fit within several existing clay settling ponds in the southwest portion of the District's Old Florida Plantation property. This layout results in an estimated annual total nitrogen load reduction of about 27 percent.

Conceptual Site Layout

Figure 5.2-1 shows the conceptual layout for the 2,540-acre full-scale system located adjacent to Lake Hancock. Figure 5.2-2 shows the conceptual layout for the 1,095-acre system. The following paragraphs provide more detailed descriptions of the proposed layouts. Construction costs and quantities are also provided for both layouts.

The proposed 2,540-acre system consists of 9 individual constructed wetland cells, with a common buffer distribution cell, and two parallel trains, each with four cells in series. Table 5.2-1 summarizes the cell sizes and proposed controlling elevations. The system was compartmentalized for the following reasons:

- Compartmentalization (cells-in-series) forces the collection and redistribution of flow, thereby increasing overall hydraulic and performance efficiency

- Parallel flow paths allow cells to be taken off line for maintenance activities

The 1,095-acre system consists of a buffer cell and three treatment cells that operate in series. The proposed control valves and bypass structures provide flexibility in how water is routed through the system. Table 5.2-2 summarizes the cell sizes and proposed controlling elevations.

DRAWING SCALE: NONE

FIGURE NO. 5.2-1

CONCEPTUAL PLAN
AND
2540 ACRE SURFACE FLOW TREATMENT WETLANDS

LAKE HANCOCK OUTFALL TREATMENT PROJECT
ALTERNATIVE TREATMENT TECHNOLOGIES
SOUTHWEST FLORIDA WATER MANAGEMENT DISTRICT

PARSONS • Tampa, Florida (813) 933-4650

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DRAWING SCALE: NONE

FIGURE NO. 5.2-2

CONCEPTUAL PLAN
1,095 ACRE SURFACE FLOW
TREATMENT WETLANDS

LAKE HANCOCK OUTFALL TREATMENT PROJECT
ALTERNATIVE TREATMENT TECHNOLOGIES
SOUTHWEST FLORIDA WATER MANAGEMENT DISTRICT

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Table 5.2-1 Summary of Design Criteria for a 2,540-Acre Surface-Flow Constructed Wetland

System Details					
Area (ac)					2,540
Average Flow (cfs)					44
Maximum Flow (cfs)					110
Average Hydraulic Loading Rate (cm/d)					1.0
Maximum Hydraulic Loading Rate (cm/d)					2.6
Cell Details					
Cell ID	**Area (ac)**	**Water Surface Elevation (ft)**	**Levee Top Elevation (ft)**	**Deep Zone Bottom Elevation (ft)**	**Maximum Allowable Grade in Cell (ft)**
Buffer	110	113.0	117.0	<108.0	110.0
1	235	112.0	116.0	108.0	111.0
2	220	112.0	116.0	108.0	111.0
3	304	110.0	114.0	106.0	110.0
4	485	110.0	114.0	106.0	110.0
5	308	120.5	124.0	116.0	120.0
6	400	120.5	124.0	116.0	120.0
7	238	118.5	122.0	114.0	118.0
8	240	118.5	122.0	114.0	118.0

Table 5.2-2 Summary of Design Criteria for a 1,095-Acre Surface-Flow Constructed Wetland

System Details					
Area (ac)					1,095
Average Flow (cfs)					29
Maximum Flow (cfs)					52
Average Hydraulic Loading Rate (cm/d)					1.6
Maximum Hydraulic Loading Rate (cm/d)					2.9
Cell Details					
Cell ID	**Area (ac)**	**Water Surface Elevation (ft)**	**Levee Top Elevation (ft)**	**Deep Zone Bottom Elevation (ft)**	**Maximum Allowable Grade in Cell (ft)**
Buffer	62	121.0	125.0	<110.0	113.0
1	374	120.5	124.0	116.0	120.0
2	490	118.5	122.0	114.0	118.0
3	169	115.5	119.0	111.0	115.0

Cell Grading

Because the existing topography is highly variable, level grading within the cell boundaries may not economically practical (See Figures C-1 through C-9 in Appendix C for profiles of existing site). For purposes of this conceptual evaluation, it is assumed that high elevations will be cut down to the maximum grades shown in Tables 5.2-1 and 5.2-2. With the exception of deep zone excavation and the excavation of high areas, no other internal cell grading is proposed. Excavated materials will be distributed on-site to fill low areas within the cells.

Earthwork volumes were calculated using Autodesk Land Development Desktop (LDD) software grid method. A proposed three-dimensional surface was created and compared to the existing digital terrain model provided by the District. Earthwork volumes were reported only for amount of material to be removed above the maximum elevation (i.e. cut volumes only).

Deep Zones

Transverse deep zones are provided for flow distribution and habitat diversity. Deep zones will be constructed at the inlet and outlet of each wetland cell, and at intermediate locations within each cell as needed to approximate "plug-flow" hydraulics. Deep zones will be excavated to a depth of 4 feet below the average wetland grade, with side slopes of 3:1 or flatter. The deep zone bottom will be excavated at a minimum width of 20 feet. Figure 5.2-3 shows a typical profile view through a surface-flow constructed wetland cell with deep zones.

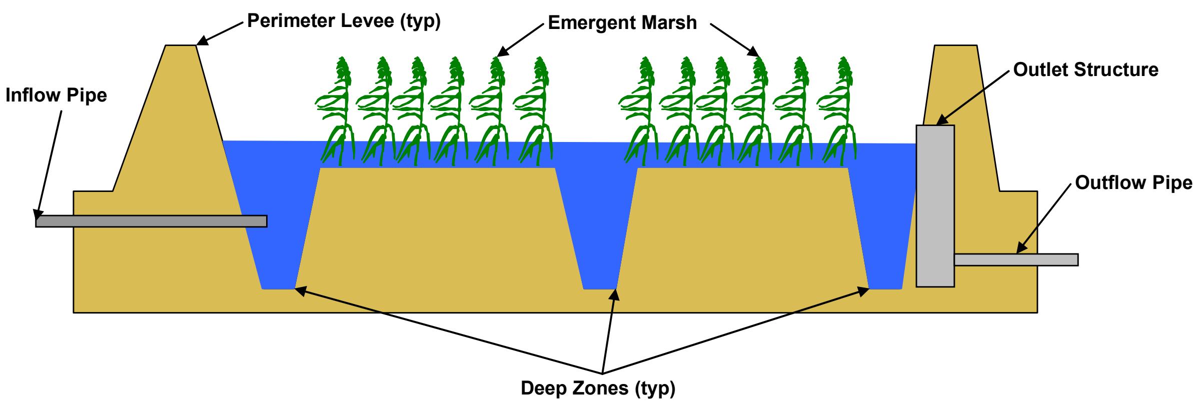

Figure 5.2-3 Typical Surface-Flow Constructed Wetland Profile View (not to scale)

Levees

Levees are needed to compartmentalize the system and provide vehicular access around the site. Perimeter levee tops will have a minimum width of 20 feet and side slopes will be at least 3 to 1 or less for maintenance access. Levee top elevations are established based upon the sum of the design water surface elevation (incorporating peak flow head loss), the storage depth needed for life-cycle sediment accretion, and freeboard for specified storm events. Levee tops will be hardened with limestone base or gravel for vehicle traffic. Intermediate levees may be constructed with reduced top widths and lower top elevations if necessary to balance cut and fill quantities or meet budget limitations.

Emergency overflow notches will be constructed in each levee in the event cell outlet structures become obstructed. Overflows will be directed first to adjacent cells, with the ultimate discharge directed to Lower Saddle Creek.

Design criteria for the buffer cell levees may differ from the treatment cell levees due to the greater depth of impounded water required to provide gravity flow.

Earthwork volumes to bring levees to required elevations was calculated using LDD grid volume techniques. Earthwork volumes were report only for amount of additional fill required to bring levees to required elevation and minimum configuration.

Outlet Canal

For the 2,540-acre alternative, an outlet canal will be constructed to combine the outflows from the two parallel treatment trains and route treated water to the outlet cascade aeration structure.

Multiple outlet structures and culverts are proposed for the 1,095-acre layout to provide operational flexibility. The proposed primary point of discharge is through a new outlet aeration device and channel that will discharge to Lower Saddle Creek southeast of the P-11 structure.

Raw Water Pump Station

For the 2,540-acre layout, a raw water inflow pump station with a variable flow capacity ranging from 5 to 110 cfs will be located on the shore of Lake Hancock, as shown in Figure 5.2-1. Power requirements and projected annual power costs are listed in Appendix C. The pump station will include mechanical bar screens to collect and remove trash and floatable debris, three inlet bays, each provided with its own sluice gate into a common wet well, and constant speed submersible propeller pumps varying in capacity from 5 cfs to 37.5 cfs, achieving a total pumping capacity of 110 cfs with all pumps in operation. Discharge from the pump station will flow through one 64-inch diameter steel transmission main, delivering 110 cfs to the buffer cell. Flow from the buffer cell to the remaining wetland cells will be through gravity structures and culverts.

Because there is a significant change in topography from the northeast section of the site to the southwest, an intermediate pump station will be required to lift water from Cells 3 and 4 to Cells 5 and 6 of the 2,540-acre system. The intermediate pump station will consist of a common wet well that draws from Cells 3 and 4. The discharge side of the pumps will be fitted with valves and flow meters to proportion the inflows to Cells 5 and 6.

For the 1,095-acre layout, the raw water pump station is located on the south shore of Lake Hancock, as shown in Figure 5.2-2. The pump station will consist of multiple pumps with a combined capacity of 52 cfs. The force main from the station to the buffer cell will be outfitted with gate valves to allow direct discharge to each treatment cell.

Water Control Structures

Water control structures will be required at the outlets from each cell. Control structures will consist of cast-in-place or pre-cast concrete boxes fitted with adjustable rectangular weir plates. Weirs will be adjustable by hand wheel operation. Culverts will be used to transfer water from the weir boxes to the downstream cells. Weirs and culverts will be sized to pass the maximum design flows with the minimum practical head loss.

Automated water control structures may be constructed, at an additional cost, to allow for remote operation from the District's control center, if desired.

Cascade Aeration Outlet Structure

Outlet structures will provide for the discharge of treated flow from the outlet canal to Lower Saddle Creek. The structures will consist of a sluice gate and stair-step concrete apron that follows the slope of the perimeter levee. The passive aeration structures will provide an increase in dissolved oxygen concentration prior to final discharge. Gates at the downstream end of the cascade structure will direct treated water to Lower Saddle Creek.

Instrumentation

Instrumentation requirements include flow metering devices and water level recorders. A flow meter will be required on the discharge from the pump station so that accurate measurements of the inflow can be determined. Water level recorders will be required at the upstream and downstream ends of each cell.

Water level and flow meter measurements may be relayed to the District control center through a radio telemetry system, at an additional cost, if desired.

Boat Ramps

Ramps will be provided for airboat access to the interior of each wetland cell. Ramps will be constructed of compacted fill material placed at a 6:1 slope from the levee top to wetland cell grade.

Planting

Options for vegetation establishment within treatment wetland cells include natural recruitment and planting. Natural recruitment is a reasonable approach for full-scale system vegetation establishment, but may not result in adequate plant cover within the constraints of the District's schedule for beginning operations. Some planting or seeding may be required to supplement natural recruitment. Careful scheduling of construction activities will be necessary to allow maximum plant growth prior to the start of routine operations. All levees will be hydro-seeded for erosion protection.

5.2.3 Operation and Maintenance Requirements

This section provides general management information for several operational scenarios, presents a proposed operational monitoring plan, and further describes anticipated maintenance activities.

Wetland operations are relatively simple, and primary system control is possible through operation of only three types of structures:

- Inflow pump station

- Intermediate pump station (2,540-acre wetlands only)
- Cell-to-cell water control structures
- Outflow weirs

The inflow pump station regulates the quantity and timing of lake discharge introduced into the wetland. The outflow weirs regulate the timing and release of water from the wetland as well as the depth of water in the wetland cells subject to hydraulic head loss considerations.

In the case of the 2,540 acre wetland solution, the intermediate pump station will serve to transfer and regulate the flow been the cells 3 and 4 to cells 5 and 6, respectively (Figure 5.2-1).

In addition to these primary management controls, monitoring of system operation and performance is critical to successful operations. Monitoring data need to be validated, summarized, and evaluated on a near real time basis to provide effective system control.

Startup Operations

The operational goal during wetland startup is encouraging the grow-in of wetland vegetation while limiting or avoiding release of dissolved and particulate nutrients downstream. Vegetation grow-in will provide an initial storage for nutrients released from the newly flooded soils. Nutrients initially released from soils will be taken up by the growing plants in their biomass. Some of the released nutrients will also be taken up by microbial populations that will colonize the site under wetter conditions. The intention is to not release any water from the wetland until surface water nutrient concentrations are equal to or below the concentration of nutrients in the source water.

Outlet weir structures will be closed during wetland startup. The goal of this startup period is to fully saturate soils with surface water and to control water depths in the optimal range for wetland plant growth while preventing outflow. Any water requiring discharge from the wetland will be recycled to Lake Hancock.

Normal Operations

Normal operations are defined as operations up to and including the design peak flow pumping rate. Outlet weirs will be set to provide a mean water depth of about two feet at the peak flow pumping rate. Vegetation density and resulting head loss are expected to vary over time and with system maturation, and outlet weirs may need to be adjusted periodically to assure flooding of the entire wetland footprint without excessive water depths in the cells.

Wet Weather Operations

Extreme rainfall events are possible in the vicinity of the Lake Hancock Outfall Treatment Project. The wetland can be operated during these storms without problem. Direct rainfall on the wetland will accumulate, resulting in a higher water stage in the cells. However, outflow through the outlet weirs will quickly equilibrate with this additional inflow due to the power-law effect of stage on outlet weir flow. In addition, emergency overflows will be provided on all levees to allow for levee protection during an unlikely plugging of the outlet weirs.

Dry Weather Operations

During natural drought conditions, it is not anticipated that water will need to be added for hydration of the wetlands. Because of the high clay content in the native soils and the incorporation of deep water zones into each cell, sufficient moisture should be available to maintain vegetation. Observations from field visits to the site indicate that existing vegetation can be sustained without artificial hydration during natural dry periods.

During extended drought conditions, artificial hydration may be necessary to maintain viable vegetation. A benefit of the proposed system configuration is water from the lake can be pumped periodically to hydrate the cells. However, its use must be balanced with LLMP and the need to maintain the minimum flows and levels (MFLs) in the Upper Peace River and losses such as evapotranspiration (ET). Based on an estimated maximum monthly ET rate of 0.25 inches per day, the required pumping rates to hydrate the wetlands are 27 cfs and 12 cfs for the 2,540-acre and 1,095-acre wetlands, respectively. In the case of an extended drought, the following affects on wetland operation could be observed:

- Oxidation of organic-bound nutrients in the wetland soils and dissolution and release of nutrients upon re-flooding
- Loss of wetland plant dominance and competitive replacement by invasive upland plant species with subsequent die-off upon re-flooding
- An inability for the District to meet downstream minimum flow requirements with treated water.

Possible responses include water retention in the wetland, pumping of makeup water to the extent possible, and caution before re-establishing flow-through conditions with discharge to Saddle Creek. Re-establishment of vegetation, which may also require artificial planting of some species, may be needed to recover treatment efficiency.

Monitoring and Performance Optimization

Monitoring and data analysis are key to successful wetland operation. Because of the limited selection of controls available to the wetland operator, and due to the relatively slow response time of a wetland to operational changes, data trends need to be followed to anticipate system performance changes before they become extreme. Routine wetland monitoring for operational control includes the following groups of parameters:

- Inflows and outflows
- Inflow and outflow water quality

- Water stage and depth
- Plant community dominance and health

Table 5.2-3 summarizes the proposed operational monitoring program for the Lake Hancock Outfall Treatment Project. The importance and scope of each of these measurements is briefly described below.

Table 5.2-3 Proposed Operational Monitoring Program for the Lake Hancock Outfall Treatment Project

Parameter	Sampling Frequency
Water Stage and Flows	
Rainfall	Daily
Water Stage	Daily
Water Flow	Daily
Water Quality	
Temperature	Weekly
Dissolved Oxygen	Weekly
pH	Weekly
Conductivity	Weekly
Total Suspended Solids	Weekly
Total Phosphorus (as P)	Weekly
Ortho Phosphorus (as P)	Weekly
Total Nitrogen (as N)	Weekly
Ammonium Nitrogen (as N)	Weekly
Nitrate/Nitrite Nitrogen (as N)	Weekly
Chlorides	Monthly
Alkalinity	Monthly
Sulfate	Monthly
Sodium	Monthly
Hardness	Monthly
Trace metals	Semi-Annually
Agri-chemicals	Semi-Annually
Plant Community Dominance	
Aerial photography	Semi-Annually
Semi-quantitative estimates	Monthly

Inflows and Outflows

Knowledge of water inflows and outflows is imperative for estimating wetland performance. Inflows and outflows will be measured on a continuous basis using the monitoring equipment provided with this project. Inflows will be estimated based on pumping records and flow meter output. Water outflows will be estimated using water stage recorders and applicable weir equations. Water stage will be measured in each cell. Staff gauges will be located at the up and down-gradient ends of each cell. A continuous stage recorder will be installed near the outlet of each cell. These recorders will provide a continuous record of water levels. Knowledge of these levels is critical for assessing the system water balance, estimating nutrient removal performance, and understanding the effects of water depth on wetland plant communities.

Inflow and Outflow Water Quality

Inflows and outflows will be monitored for the following parameters and at the specified sampling intervals:

- Field parameters and nutrients: temperature, pH, dissolved oxygen, specific conductance, total suspended solids, total phosphorus, ortho-phosphorus, total nitrogen, ammonium nitrogen, nitrate/nitrite nitrogen – weekly

- Major water quality parameters: chlorides, sulfate, sodium, hardness, alkalinity – monthly

- Non-routine water quality parameters: trace metals and agri-chemicals – semi-annually

The purpose of this water quality monitoring is to provide a record of performance of the wetland for water quality improvement. Analysis of this system performance will be utilized to make informed operations and maintenance decisions. Sampling for trace metals and agri-chemicals is recommended during the start-up phase, but may not be required long-term if the results are consistently below threshold toxicity limits.

Plant Community Dominance and Health

The wetland plant community is critical for effective wetland performance. For this reason and because of the potential time lag involved in re-establishing the plant community if it is impacted or altered, it is necessary to provide basic monitoring of plant community dominance and health. Two types of basic monitoring are described:

- Aerial photography and interpretation – semi-annual

- Semi-quantitative plant cover estimates - monthly

High altitude color infrared photographs will be used to document plant communities twice each year, during the summer and winter seasons. These photographs will be interpreted and digitized to allow estimation of the aerial extent of dominant plant community types.

A semi-quantitative inventory of plant cover will need to be prepared monthly to provide a written record of plant community dominance. An observer from the external levees can prepare these estimates. Dominant plant associations (emergent, FAV, SAV, filamentous algae, etc.) and plant communities will need to be quantified based on eight cover categories:

- 0 to 1 percent
- 1 to 5 percent
- 5 to 25 percent
- 25 to 50 percent
- 50 to 75 percent
- 75 to 95 percent
- 95 to 99 percent
- 99 to 100 percent

Cover estimates will be made from locations evenly spaced around each of the wetland cells.

Sample Station Locations

Figures 5.2-4 and 5.2-5 identify proposed sampling station locations for the two alternative designs. Twelve water quality stations are proposed for the 2,540-acre layout, and six are proposed for the 1,095-acre layout. Water quality stations should be monitored at the frequencies noted in Table 5.2-3. Water stage and depth will be located at the up and down-gradient ends of each cell. Stations are located at the inlet to the buffer cell, the inlets and outlets from the treatment cells, and at the final discharge locations. Plant community dominance and health will be estimated from the levees at a total of 29 stations for the 2,540-acre layout and 14 stations for the 1,095-acre layout. Vegetation monitoring stations will be evenly spaced around the perimeter of each cell. Some stations can be used for multiple cells.

Data Management and Analysis

Monitoring data should be entered into computer spreadsheets for summary and analysis. Data trend graphs should be updated frequently to assess performance and to allow changes in operation/management of the system including varying loadings to individual cells and changing water depths.

DRAWING SCALE: NONE

FIGURE NO. 5.2-4

PROPOSED MONITORING PLAN
AND
2540 ACRE SURFACE FLOW WETLANDS

LAKE HANCOCK OUTFALL TREATMENT PROJECT
ALTERNATIVE TREATMENT TECHNOLOGIES
SOUTHWEST FLORIDA WATER MANAGEMENT DISTRICT

PARSONS • Tampa, Florida (813) 933-4650

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DRAWING SCALE: NONE

FIGURE NO. 5.2-5

PROPOSED MONITORING PLAN
1095 ACRE SURFACE FLOW WETLANDS

LAKE HANCOCK OUTFALL TREATMENT PROJECT
ALTERNATIVE TREATMENT TECHNOLOGIES
SOUTHWEST FLORIDA WATER MANAGEMENT DISTRICT

PARSONS • Tampa, Florida (813) 933-4650

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Routine Maintenance

Because there are few moving parts in a constructed wetland, most of the routine tasks are related to vegetation management, maintaining site access, and maintaining flows. Routine maintenance is required to keep a wetland system operating efficiently and to keep the site accessible. Regular maintenance activities include the following:

- Site Access. Grassed levees should be mowed several times each growing season to keep the site accessible for monitoring, reduce contact between venomous snakes and pedestrian traffic, and for aesthetic value. Public access features (if included), such as boardwalks, interpretive signage, and observation decks, will require periodic safety inspections and weather-proofing.
- Inspecting water control structures and instrumentation. The inflow pump station intake and discharge points, as well all water level control structures, should be inspected at least weekly to ensure that flows are unobstructed by debris.
- Vegetation management. The wetland vegetation monitoring plan (described above) will identify whether vegetation maintenance is necessary. Herbicide or pesticide application may be necessary if a specific vegetative community is desired. In some cases, replanting or water level modifications may be required to fully retreat zones within the wetland cells.

5.2.4 Expected Finished Water Quality

Historical data from Lake Hancock were reviewed to estimate the average inflow water quality for the Lake Hancock Outfall Treatment Project. The following annual average or long-term inflow water quality concentrations were assumed to be applicable for project design:

- Total Nitrogen – 5.53 mg/L
- Total Phosphorus – 0.603 mg/L
- Biochemical Oxygen Demand – 18.0 mg/L
- Total Suspended Solids – 115 mg/L

Currently, no modeling tool is available for predicting nitrogen removal performance for the dynamic inflows that are expected in this facility. The k-C* steady-state model (Kadlec and Knight, 1996) was adapted to estimate treatment wetland performance for a quarterly time step using the Excel® spreadsheet software. Estimated daily flows at the P-11 structure (1/1975 – 12/2003) were rolled up to quarterly averages and combined with the concentration data summarized above. The resulting quarterly average wetland effluent concentrations and mass removal rates were determined using the k-C* model, and the quarterly values were combined to yield annual average performance estimates. Model parameters and output are provided in Appendix C. The pump station capacity and final area for the 2,540-acre layout were determined through an iterative process. The resulting area and pump station capacity were the minimum required to meet a target annual nitrogen load reduction of 45 percent. For the smaller layout, the pump station capacity was specified as 52 cfs, and the area was determined by maximizing the treatment footprint within the confines of the existing clay settling areas. The purpose for limiting the wetland footprint to the clay settling impoundments was to avoid significant jurisdictional wetlands and Federal Aviation Administration (FAA) permitting issues to the east and north as they relate to close proximity to the City of Bartow Airport and increased presence of water fowl (discussed in further detail later), and to avoid areas with more extreme topographic variation. In both cases, the pump stations were modeled to supply all available flow to the wetland up to the maximum pumping rate. Higher lake discharges were passed directly to Lower Saddle Creek without treatment.

Tables 5.2-4 and 5.2-5 show the estimated long-term average performance for a 2,540-acre and 1095-acre surface flow constructed wetland, respectively. While the total nitrogen load is reduced, the k-C* model predicts that concentrations of NH_4-N and NO_3-N will increase in the wetland outflow compared to the Lake Hancock inlet water. This result is expected as organic nitrogen in the lake water is mineralized to NH_4-N as part of the treatment process.

Table 5.2-4 Estimated Finished Water Quality for a 2,540-acre Surface-Flow Constructed Wetland

Parameter	Inflow Concentration (mg/L)	Estimated Outflow Concentration (mg/L)	Estimated Mass Removal Rate (kg/yr)
BOD	18	2.3	603,000
TSS	115	2.1	4,416,000
TN	5.53	1.58	130,400
NH_4-N	0.015	0.18	--
NO_3-N	0.015	0.11	--
TP	0.603	0.09	17,200

Table 5.2-5 Estimated Finished Water Quality for a 1,095-Acre Surface-Flow Constructed Wetland

Parameter	Inflow Concentration (mg/L)	Estimated Outflow Concentration (mg/L)	Estimated Mass Removal Rate (kg/yr)
BOD	18	2.5	398,400
TSS	115	2.2	2,957,500
TN	5.53	2.0	79,200
NH4-N	0.015	0.32	--
NO3-N	0.015	0.19	--
TP	0.603	0.15	10,600

5.2.5 Residuals Disposal

While treatment wetlands have been proven to provide long-term TN and TP removal over a wide range of loading rates, there are differing opinions as to the need for maintenance, particularly sediment management, during the design life of a wetland. To date, very few wetlands have required any sort of sediment management. In addition, those where sediments were removed did not exhibit any prior degradation in overall performance that implied maintenance was necessary.

Sediment accretion is the net result of several external loading sources and a variety of internal wetland processes such as plant growth, decomposition, and net accretion of residual organic solids or accreted sediments (“biomachine”). These processes are illustrated in Figure 5.2-6. A large body of post-startup data from treatment wetlands worldwide indicates that under most normal circumstances, nutrients such as TN and TP have exponentially declining concentration gradients with travel distance (and time) through a treatment wetland. The first point on this curve is the average wetland inlet concentration and is a function of the type of wastewater or stormwater being treated and the level of pre-treatment, if any. This concentration declines with distance through the wetland, initially faster and at a decreasing rate to a lower average outlet concentration. The actual magnitude of this lower outflow concentration relative to the inlet concentration is indicative of the design and operation of the treatment wetland including the hydraulic loading rate of the wastewater into the wetland, the type of wetland, its area, hydraulic considerations such as degree of mixing, plant communities, and a number of other factors important in treatment wetland design.

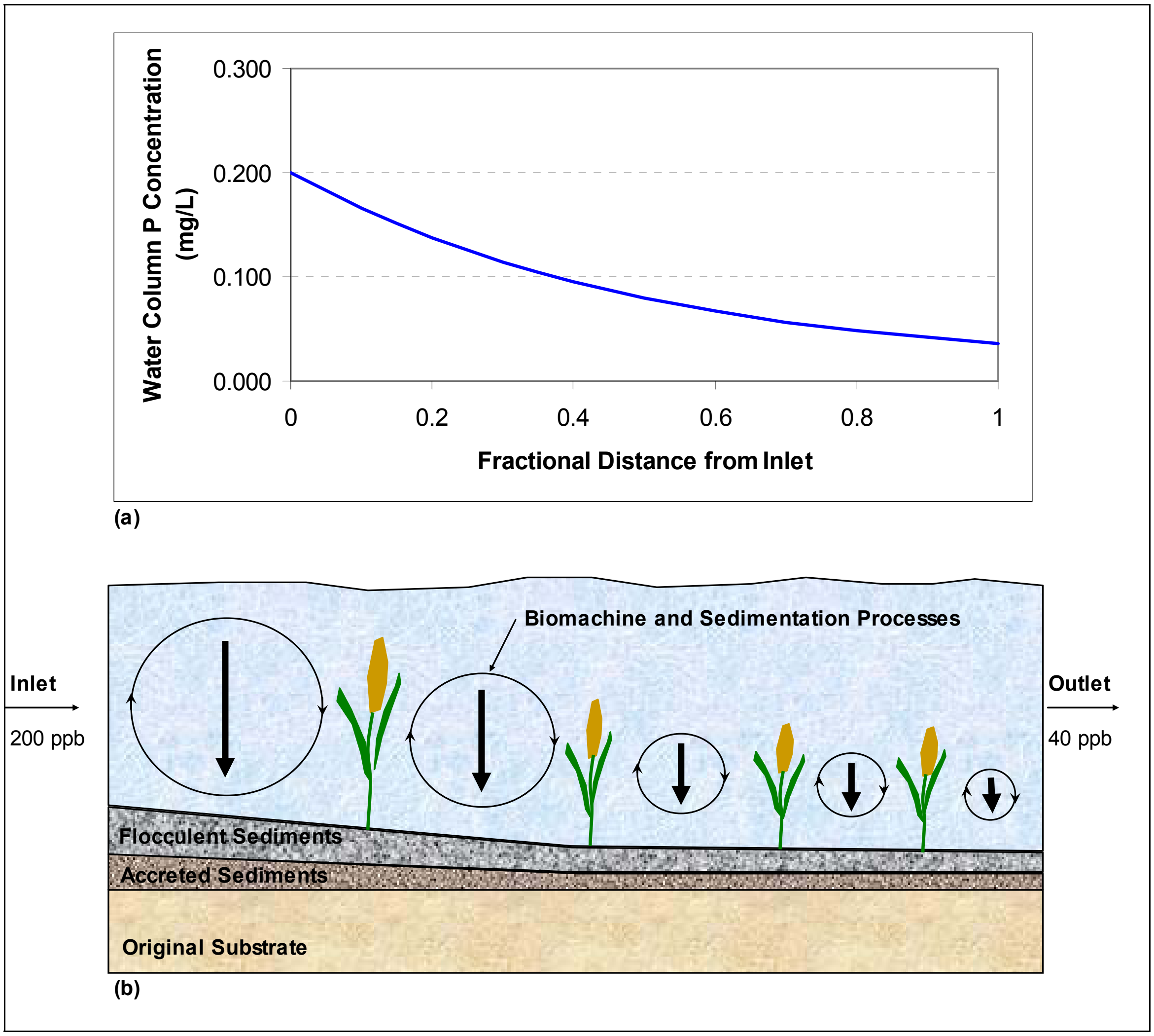

Figure 5.2-6 Steady-State Gradients for Sediment Accretion and Internal Processes (adapted from Kadlec and Knight, 1996). ***Circles represent the relative magnitude of the biomachine along the gradient from inlet to outlet. Downward arrows indicate net settling processes and nutrient fluxes to the sediments. The size of the arrows represents the relative magnitude of sediment processes and nutrient fluxes from inlet to outlet. (scales are exaggerated to illustrate processes)***

Because of their relatively large area compared to most conventional treatment processes, and due to their resulting long hydraulic and solids retention times, treatment wetlands are well-suited for passive nutrient treatment under conditions of variable pollutant inlet loads and concentrations. However, some net sediment accretion occurs in all treatment wetlands. The rate of this accretion is important in treatment wetland design, long-term maintenance, and economics.

The Lake Apopka Marsh Flow-way is a 660-acre system that was designed to treat highly eutrophic lake water and is very similar in concept to the Lake Hancock Outfall Treatment Project. Coveney, *et al*. (2002) measured a median accretion rate of flocculent sediments of 137 mm/yr in the first cell of the marsh, which received influent TSS concentrations ranging from 35 to 190 mg/L. Although this rate is high, the sediment thickness does not reflect compaction that would occur over time. A significant difference between the Apopka wetland and that proposed for Lake Hancock, is the storage area provided in the buffer cell; deep inlet canals; and, in un-filled portions of the treatment cells. The Apopka system has no buffer cell or internal deep zones.

Long-term net sediment accretion rates in treatment wetlands were reviewed by WSI (2004b) and found to range from 2.7 to 14.8 mm/yr, with a median value of 5.6 mm/yr. Typical long-term accretion rates that can be used in treatment wetland design range from 5 to 10 mm/yr, and at these rates, the effects on system life are expected to be minimal (about 30 to 60 years of system life per foot of levee freeboard) with appropriate levee design. An important difference in the case of Lake Hancock is that most of the discharging solids are algal particles primarily comprising of organic carbon, rather than inert matter. Organic carbon is readily oxidized in treatment wetlands, being converted to carbon dioxide, and ultimately lost to the atmosphere. A very small fraction of this organic matter is typically recalcitrant to biological degradation and ultimately forms peat, resulting in typical consolidated sediment accretion rates averaging less than 0.5 cm/yr in most treatment marshes (such as the South Florida Water Management District Stormwater Treatment Areas). In the case of the smaller 1095-acre wetland as an example and assuming that all of the suspended solids are volatile, the incoming solids loading rate is approximately 1,400 $g/m^2/yr$ (60 cfs, 115 mg/L TSS, 1,095 acres). This is a relatively small fraction of the typical internal carbon cycling rate in emergent constructed treatment wetlands of about 3,000 – 5,000 $g/m^2/yr$. With the design features included in the wetland conceptual layouts for Lake Hancock and the relatively low loading rate of organic solids, sediment accretion is not likely to measurably impact performance over the projected 50-year operating life.

5.2.6 Regulatory Requirements

As with all other treatment processes, siting of treatment wetlands is regulated by various state and federal statutes.

In accordance with Chapter 373, Florida Statutes, an Environmental Resource Permit (ERP) is required for any project affecting wetlands or altering surface water flows. In accordance with 62-341.485, Florida Administrative Code (FAC), FDEP may issue general permits to water management districts for environmental restoration or enhancement. It is likely that a treatment wetland would qualify for a noticed general ERP.

Nearly all natural wetlands are considered to be “waters of the U.S.” and “waters of the state” and are afforded legal protection from certain types of activities. This legal protection is codified in federal statutes such as the Clean Water Act and mandates permit requirements for dredging or filling (Section 404).

Section 404 of the CWA mandates that filling activities in "waters of the U.S. will be avoided or minimized, will be mitigated if they are unavoidable, and will be subject to a Section 404 permit. The U.S. Army Corps of Engineers reviews applications for Section 404 permits in Florida. Dredge and fill activities associated with natural treatment wetlands are generally minor and are frequently subject to a Nationwide Permit. This type of permit authorizes activities that are similar in nature and cause only minimal impacts to the environment. The construction of a treatment wetland for the Lake Hancock discharges may be considered self-mitigating and permitted as a Nationwide Permit.

Rare or endangered wildlife attracted to treatment wetlands (as well as to treatment ponds and lagoons) are not exempt from requirements of the Endangered Species Act. Populations of rare and endangered species that are wetland-dependent are often enhanced by treatment wetlands. As shown in Figure 5.2-7, the Bartow Municipal Airport is located approximately 3,800 feet east of the eastern boundary of the District's former Old Florida Plantation (OFP) Property. The Federal Aviation Administration (FAA) has issued an Advisory Circular (AC 150/5200-33) providing guidance on certain land uses that have the potential to attract wildlife that could be potentially hazardous to aircraft utilizing public-use airports. District staff has initiated coordination with airport staff. Concerns expressed by airport representatives focused on compliance with an avigation easement that exists within a 4.5 nautical mile radius from the center of the airport and includes the District's OFP property. The easement pertains to over flight and associated noise by aircraft utilizing the airport. Treatment wetlands as outlined above have minimal open water, and are anticipated to primarily attract wading birds which are low flying rather than water fowl such as ducks and geese that pose bird strike potential to aircraft.

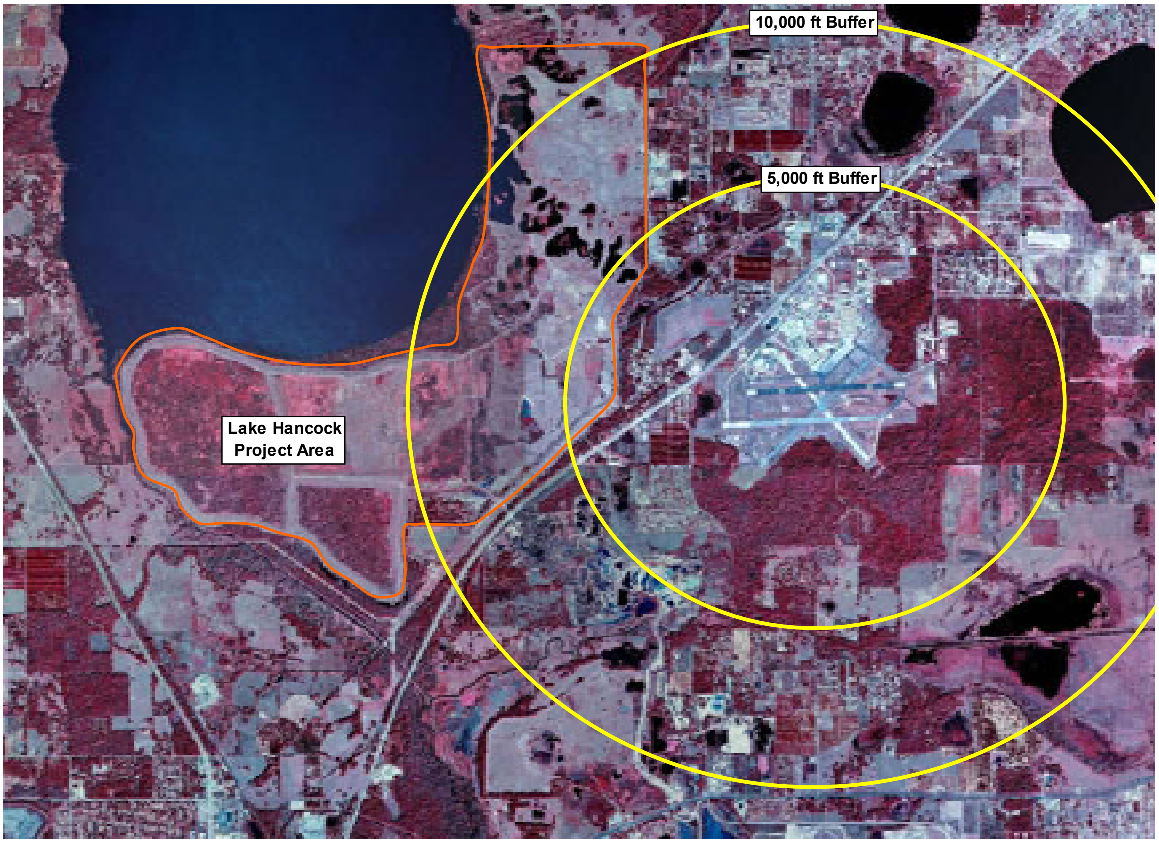

Figure 5.2-7 Proximity of the Bartow Municipal Airport to the Lake Hancock Project Area

Further coordination with the Bartow Municipal Airport will be necessary, but indications are that airport representatives do not believe a wetland treatment system constructed on the District's OFP property would adversely affect the airport's operations (SWFWMD, 2005b, see also Appendix I).

As an integral component in the process of permit compliance, treatment wetlands must be engineered to meet specific water quality goals. Due to their large land area and their open nature, treatment wetlands are not as amenable to human control as some conventional treatment processes. Treatment wetland designers and regulators must be aware of the inherent variability of the water quality leaving the wetlands and size the wetland to consistently achieve the permit limits. As described above under the section on performance, it is also critical to be aware of the natural constraint of the treatment wetland background (lowest achievable) constituent concentration and the potential for natural events such as storms, wind, drought, and pests to affect wetland outflow water quality.

5.2.7 Capital, Operation and Maintenance Costs

Capital cost items for surface-flow constructed wetlands include the following:

- Site preparation such as clearing and grubbing

- Levee construction and cell grading
- Pump station and transmission main
- Water supply and distribution (pump station, internal piping, water control structures, outfall structure)
- Engineering and permitting fees
- Contingencies

Operation and Maintenance costs may include the following:

- Levee maintenance (erosion control, mowing)
- Vegetation management (replanting, herbicide/pesticide treatment)
- Energy costs for pump stations
- Repair of pumps, valves, electrical equipment
- Operational compliance monitoring
- Maintenance of public access features, if equipped (restrooms, trash removal, boardwalk repair and water proofing, etc.)

Detailed assumptions for capital cost items are provided in Appendix C.

Table 5.2-6 summarizes the estimated capital and operations costs for the 2,540-acre surface-flow wetland system described in this report. The total present worth (50 years at 5.625%) for the surface-flow constructed wetland option is approximately $91.5 million. This cost is based upon a preliminary estimate of the earthwork requirements and may change as additional topographic and geotechnical data become available. The greatest percentage (67%) of the present worth cost of this alternative is the initial construction cost, and specifically the earthwork cost. The cost per pound (14,350,500 pounds over 50 years) of nitrogen removal is about $6.38.

Table 5.2-7 summarizes the estimated capital and operations costs for the 1,095-acre surface-flow wetland system described in this report. The total present worth (50 years at 5.625%) for the surface-flow constructed wetland option is approximately $41.8 million. This cost is based upon a very preliminary estimate of the earthwork requirements and may change as additional topographic and geotechnical data become available. About 47% of the present worth cost of this alternative is the initial construction cost. The cost per pound (8,716,000 pounds over 50 years) of nitrogen removal is about $4.79.

Table 5.2-6 Estimated Capital and Operating costs for 2,540-acre Surface-Flow Constructed Wetland needed to achieve 45% total nitrogen load reduction.

System	Capital Costs ($)	Annual Operating Costs ($)	Equipment Replacement Costs[1] ($)
Clearing and Grubbing	$6,295,000		
Earthwork	$30,956,000		
Intake and Inflow Pump Station	$1,913,000	$173,000	$1,263,000
Intermediate Lift Station	1,913,000	$123,000	$1,263,000
Inflow Transmission Main	$463,000	$5,000	$306,000
Intermediate Transmission Main	$232,000	$3,000	$153,000
Water Control Structures	$1,129,000	$12,000	$745,000
Piping	$603,000	$7,000	$398,000
Instrumentation and Telemetry	$181,000	$8,000	$120,000
Operational Monitoring		$200,000	
Routine Operations		$219,000	
Routine Maintenance		$160,000	
General Conditions & Contingency [2]	$11,148,000		
SUBTOTAL	$54,828,000	$ 960,000	$ 4,245,000
Land Acquisition [3]	0		
Engineering [4]	$6,553,000		
Total	$61,380,000	$906,000	$4,245,000
Present Worth Cost [5]	$61,380,000	$26,331,000	$3,821,000
Total Present Worth Cost	$ 91,532,000		
Per Pound Nitrogen Removed [6]	$ 6.38		

1 Replacement of equipment and material items every 20 years.
2 Includes construction contractor contingency of 20%, mobilization/demobilization of 5%, permits of 1%, bonds of 1%, insurance of 1% and sales tax on materials and equipment.
3 Land costs were not included in this present worth analysis. These costs and their influence on the present worth analysis are discussed in further detail in Section 7.0.
4 Estimated as 15% of capital costs.
5 Estimated at an interest rate of 5.625% for a 50-year period. Annual O&M costs were inflated at 3% per year. Salvage of equipment purchased at 40 years estimated at 1/3 the purchase value at the end of 50 years.
6 Listed cost based on estimated per pound nitrogen removed by constructed flow through wetlands over 50 year operating period.

Table 5.2-7 Estimated Capital and Operating Costs for 1,095-Acre Surface-Flow Constructed Wetland needed to achieve 27% total nitrogen load reduction.

System	Capital Costs ($)	Annual Operating Costs ($)	Equipment Replacement Costs[1] ($)
Clearing and Grubbing	$2,714,000		
Earthwork	$6,563000		
Intake and Inflow Pump Station	$1,343000	$210,000	$866,000
Inflow Transmission Main	$2,340,000	$24,000	$1,545,000
Water Control Structures	$591,000	$6,000	$390,000
Piping	$322,000	$4,000	$213,000
Instrumentation and Telemetry	$150,000	$6,000	$99,000
Operational Monitoring		$100,000	
Routine Operations		$219,000	
Routine Maintenance		$100,000	
General Conditions & Contingency [2]	$2,989,000		
SUBTOTAL	$17,009,000	$ 960,000	$ 3,131,000
Land Acquisition [3]	0		
Engineering [4]	$2,552,000		
Total	$19,560,000	$667,000	$3,131,000
Present Worth Cost [5]	$19,560,000	$19,390,000	$2,819,000
Total Present Worth Cost	$ 41,768,000		
Per Pound Nitrogen Removed [6]	$ 4.79		

1 Replacement of equipment and material items every 20 years.
2 Includes construction contractor contingency of 20%, mobilization/demobilization of 5%, permits of 1%, bonds of 1%, insurance of 1% and sales tax on materials and equipment.
3 Land costs were not included in this present worth analysis. These costs and their influence on the present worth analysis are discussed in further detail in Section 7.0.
4 Estimated as 15% of capital costs.
5 Estimated at an interest rate of 5.625% for a 50-year period. Annual O&M costs were inflated at 3% per year. Salvage of equipment purchased at 40 years estimated at 1/3 the purchase value at the end of 50 years.
6 Listed cost based on estimated per pound nitrogen removed by constructed flow through wetlands over 50 year operating period.

5.3 MANAGED AQUATIC PLANT SYSTEMS CONCEPTUAL PLAN – ALTERNATIVE 2 (SINGLE-STAGE WHS™)

This section presents a conceptual design and cost estimate for a managed aquatic plant system (MAPS) that will receive water from Lake Hancock. This section includes information regarding operations and maintenance activities, estimated finished water quality, residuals management, and permitting requirements.

A proposal for a MAPS was solicited from HydroMentia, Inc. of Ocala, Florida as treatment technology alternatives to remove 45 and 27 percent of the annual total nitrogen load from Lake Hancock discharge. HydroMentia provides MAPS technologies and has recent pilot-scale experience treating surface water in the Lake Okeechobee watershed. Initially, Hydromentia provided proposals for a combination Water Hyacinths Scrubber (WHS™) and Algal Turf Scrubber (ATS™) for the 45 percent target (HydroMentia, 2004a), and a stand alone single-stage WHS™ (once through) system for the both the 45 and 27 percent targets (HydroMentia, 2004b). The combination WHS™-ATS™ system was eliminated from further evaluation, as the ATS™ requires a fairly consistent and constant flow-rate to maintain system viability and performance which is not available treating the variable discharge flow from Lake Hancock. Copies of proposals for the WHS™ systems are given in Appendix D, with highlights presented below.

5.3.1 General Description

HydroMentia, Inc. markets a MAPS known as the Water Hyacinth Scrubber (WHS)™. A typical WHS™ system consists of one or more water hyacinth treatment ponds and associated harvesting and solids management equipment. HydroMentia's processes rely on routine harvesting of plant biomass and settled solids as the primary means of nutrient removal.

A single-stage WHS™ system is designed to provide the following benefits:

1. The WHS™ ponds provide a means for attenuating the phytoplankton load through shading, settling and interspecific competition. The high nitrogen load results in high levels of water hyacinth productivity and, accordingly, relatively high rates of removal.

2. The WHS™ conditions the water quality by :

    - Reducing the organic solids loads and facilitating conversion of organic nitrogen to more available forms, largely through lysis (breakdown) of the algal cells associated with the heavy phytoplankton load.

    - Direct plant uptake of the nutrients nitrogen and phosphorus, and the subsequent recovery of these nutrients through crop harvesting and processing into organic fertilizer/compost products. These by-products can then be either removed from the watershed, thereby avoiding extensive storage within the Lake Hancock watershed, or substituted for imported fertilizer products, thereby reducing nutrient imports into the basin.

    - Facilitating other nitrogen removal mechanisms such as nitrification-denitrification, larval emergence, and predatory migration—i.e. a visiting predator feeds upon organisms within the WHS™ then migrates from the site, thereby removing nitrogen (and phosphorus).

    - Reducing biodegradable organic loads, as well as reduction of metals and synthetic organic pollutants.

- Modulating pH fluctuations by transferring primary productivity from phytoplankton to water hyacinths. High pH levels attendant with low alkalinities and high phytoplankton blooms can be deleterious to certain aquatic communities. Within the hyacinth system CO_2 is generated through heterotrophic activity within the root zone and the sediments. This typically reduces pH to between 5.5 and 7.0 and attenuates the diurnal variability of the pH, and eliminates high pH (>9.5) peaks.

- The floating plant mass modulates water temperature by providing insulation, which levels out fluctuations both in the summer and winter.

5.3.2 Conceptual Design

The conceptual design for the MAPs consists of an inlet pump station, influent conveyance channel, WHS™, re-aeration lagoon and discharge channel. Influent enters a series of cells in parallel in which water hyacinths utilize the high nutrient loads present to grow and produce “biomass”. Nitrogen removal is ultimately achieved through harvesting of the water hyacinths and removal of settled solids. Figures 5.3-1 and 5.3-2 show site layouts for the 45 and 27 percent nitrogen load reduction targets, respectively. Components of the system are described below.

Influent Pump Station

An inflow pump station with a variable flow capacity ranging from 5 to 300 cfs will be located on the shore of Lake Hancock as shown in Figures 5.3-1 and 5.3-2. A higher capacity station is needed in this case compared to the other alternatives because of lower removal efficiency reported the WHS™ (see Appendix D). This pump is assumed to provide water for both alternatives; a reduction of maximum flow capacity was not explored in Hydromentia’s proposal for the 27 percent load reduction target. The inflow pump station will include mechanical bar screens to collect and remove trash and floatable debris, three inlet bays, each provided with its own sluice gate leading into a common wet well, and constant speed submersible propeller pumps varying in capacity from 5 to 100 cfs, achieving a total pumping capacity of 300 cfs with all pumps in operation. Discharge from the pump station will flow through two 64 inch diameter transmission mains, delivering 300 cfs to the WHS™ units.

Influent Conveyance Channel

A lined (40 mil HDPE) influent conveyance flume will route water from the inflow pump station to the WHS™ units. Individual 8- to 10-inch laterals will transfer flow from the flume to the four upstream WHS™ units. Control of flow would be through low-pressure in-line valves, such as those manufactured by Pond Dam Piping, LTD.

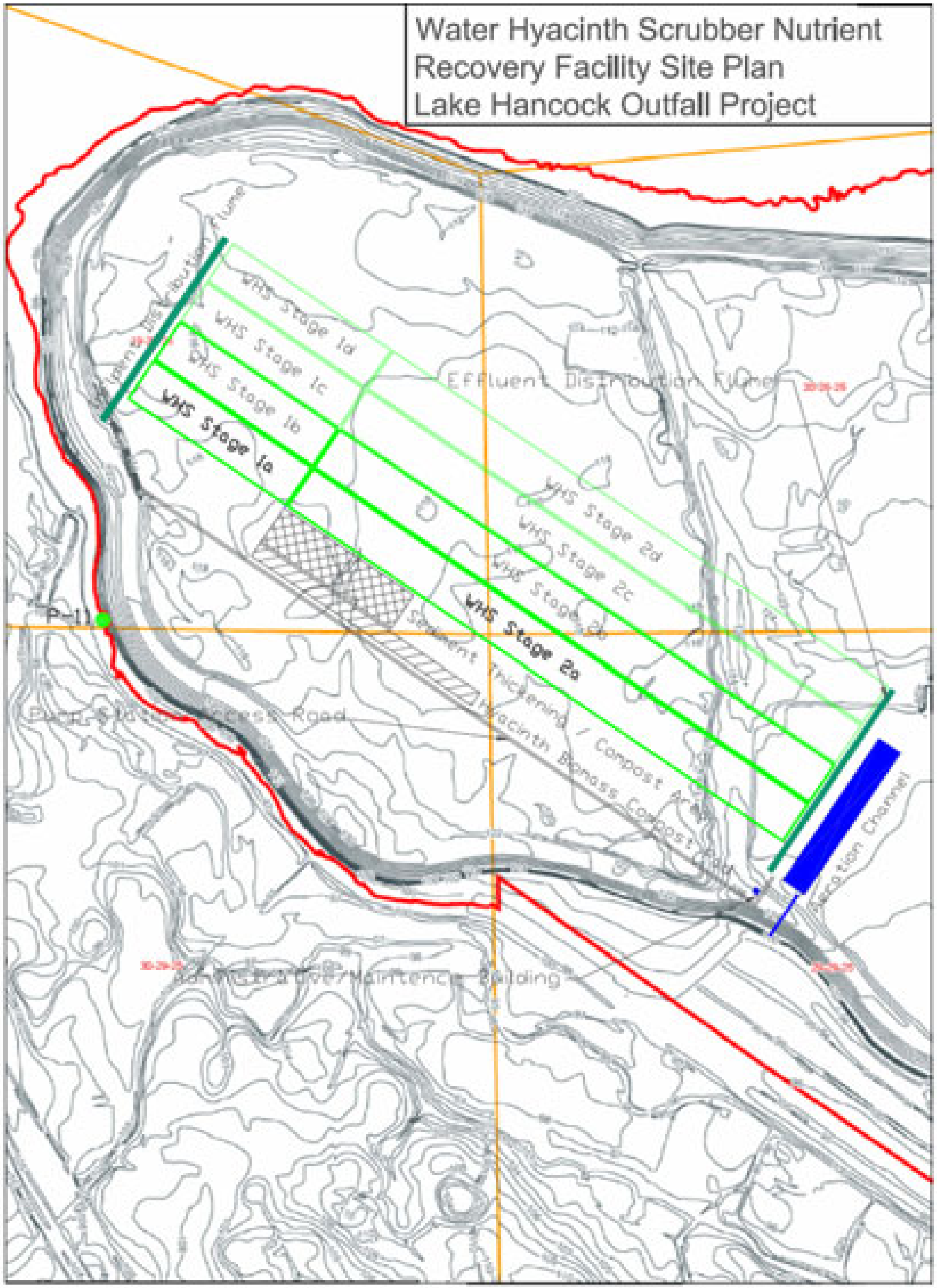

Figure 5.3-1 General Site Layout for 45% TN Reduction, 210 AcreWHS™. Drawing not to scale (HydroMentia, 2005a).

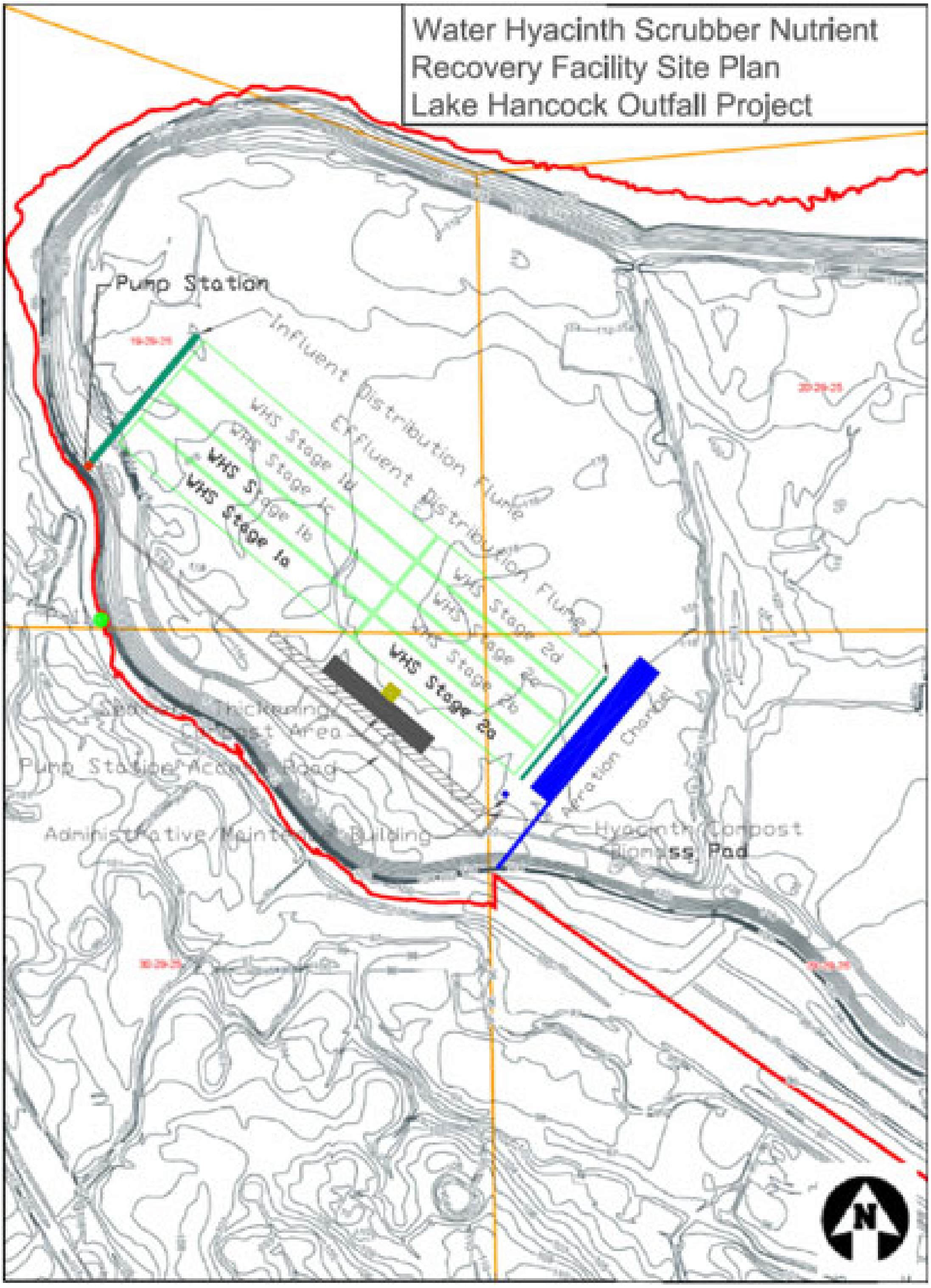

Figure 5.3-2 General Site Layout for 27% TN Reduction Target, 88 Acre WHS™. Drawing not to Scale (HydroMentia, 2005b).

WHS™ Units

Figure 5.3-3 and 5.3-4 show the schematic layout for the proposed WHS alternatives. Detailed descriptions of the WHS Units, water hyacinths and solids recovery methods are given in Appendices D1 and D2 for the 45 and 27 percent load reduction targets, respectively.

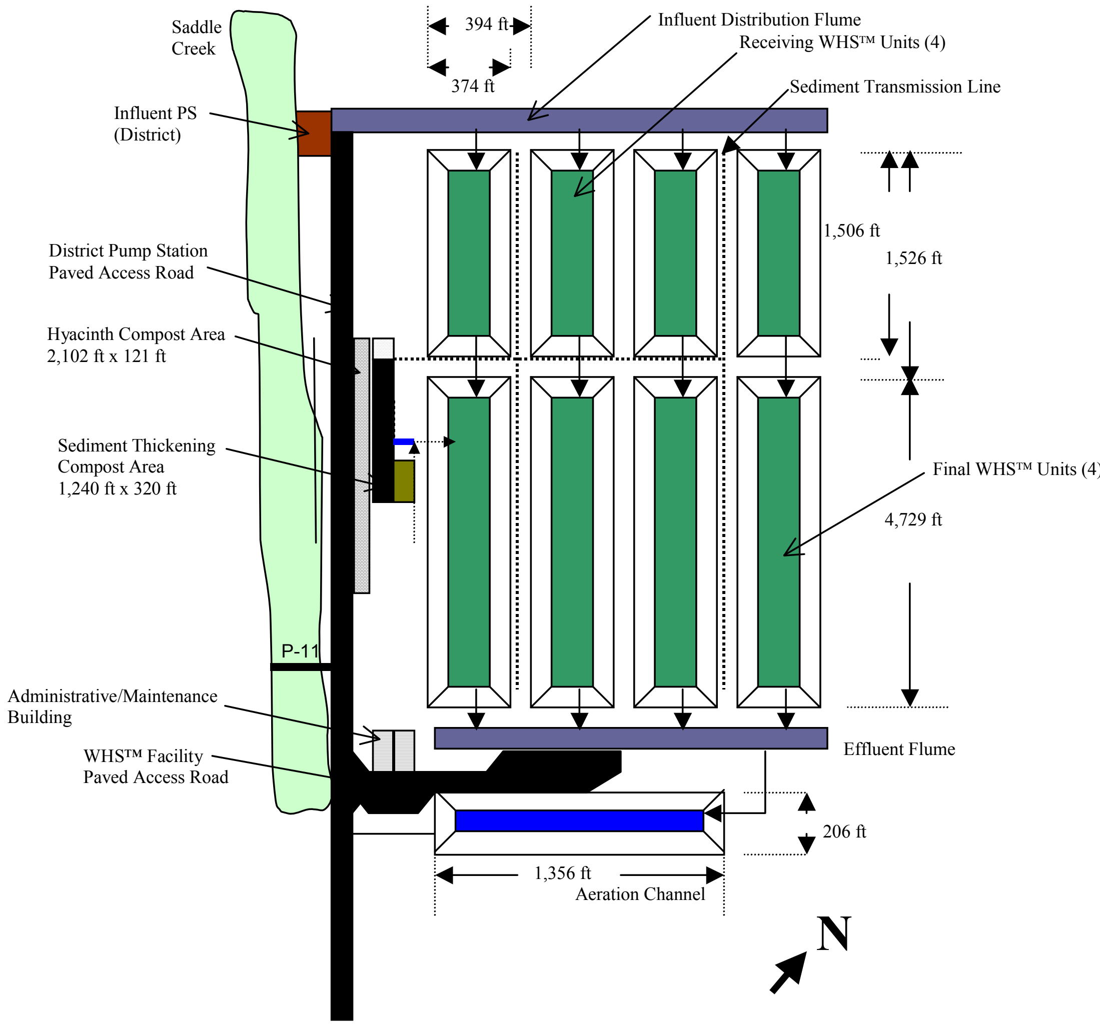

Figure 5.3-3 45% TN Reduction Target, 210 Acre WHS™ (drawing not to scale). Hydromentia, 2005a.

The four WHS™ units (2 in series and 4 in parallel) consist of unlined ponds protected by earth berms. The operation of the WHS units would be segregated into smaller 100 ft long growing units separated by 6" floating booms, which prevents excessive compression of the hyacinth crop, and facilitates healthy production. The initial receiving units will serve to a greater extent to settle and transform the heavy solids loads. Each parallel WHS™ train includes this receiving unit and a larger final unit. Water will be transferred through adjustable overflow weirs, thereby facilitating effective settling within the first unit. Effluent discharge from the final WHS™ units will also be through a series of overflow weirs. The effluent will be directed to the effluent flume, which eventually delivers the flow to the aeration channel. The WHS™ units will be bordered by a 20-ft harvest road to permit access by the integrated harvesting/processing system.

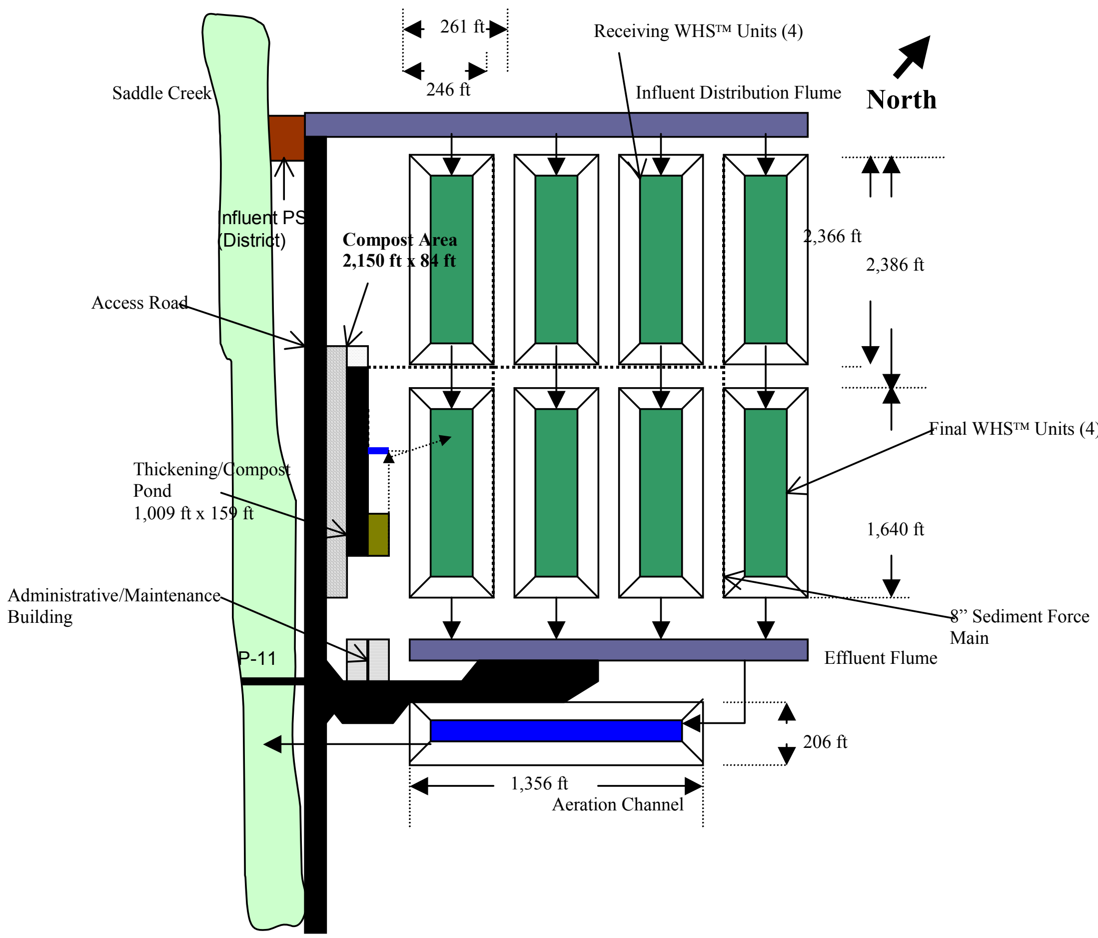

Figure 5.3-4 27% TN Reduction Target, 88 Acre WHS™ (drawing not to scale). Hydromentia, 2005a.

WHS™ Harvesting Equipment

Harvesting of the WHS™ unit will be via HydroMentia's Model 101-G WHS™ harvest grapple used in tandem with a mobile version of a Model 401-P biomass processor, as developed by HydroMentia, to include cross and vertical conveyors as necessary. The use of conveyance flumes in this system is not considered cost effective because of the distances involved. Drive will be by a tractor PTO (John Deere Model 7420 or equivalent). The harvest grapple will transfer harvested biomass (300-450 lbs per grapple) into the processor, and the chopped product will be then delivered into a transfer trailer (Miller Series 5300 or equivalent) which, when loaded, will transfer the chopped biomass to the compost area. The recovered hyacinth biomass, once delivered to the compost area, will be spread into a windrow.

Composting Area

Harvested hyacinths and sediments will be windrow composted in the on-site composting area.

WHS™ Effluent Flume

Flows from the final WHS™ will be delivered to an effluent flume, from which flows will be directed to the final aeration channel.

Final Re-Aeration Lagoon

Because the WHS™ effluent is likely to have a DO concentration below 5 mg/L, post-aeration is required prior to final discharge. The post aeration system includes a lined pond with paddlewheel aerators. The proposed dimensions for the pond are 200 feet wide by 1,350 feet long by 4 feet deep. A workable design would involve 20-10 HP paddlewheels, about 12 feet in length, placed in a staggered manner along the long axis of the pond. The pond would have one foot of freeboard. After aeration, flows will be directed for release into designated receiving waters.

5.3.3 Operation and Maintenance Requirements

The MAPS technologies are much more labor-intensive than conventional treatment wetlands. Daily management of these systems is required because of the level of harvesting that is necessary to maintain optimum biomass growth rates and because there are a number of moving parts that require inspection and maintenance. Additional detail regarding the solids management requirements is provided in Section 5.3.5 (Residuals Disposal). The general management requirements include the following:

- Inspection and adjustment of inflow surgers and outflow weirs
- Daily harvesting of hyacinths.
- Sloughed material and settled solids will need to be removed from the WHS™ ponds. A hydraulic dredge will be required for these sediment removal events.
- HydroMentia notes that micronutrient supplements may be necessary, depending upon the levels in the source water.
- An effluent water quality monitoring program will be required to demonstrate compliance with discharge standards.
- Maintenance of harvesting equipment (rakes, tractors, etc.) and replacement after approximately 20 years.
- Loading and hauling of composted solids to either a landfill or land spreading site.

5.3.4 Expected Finished Water Quality

HydroMentia's estimates of finished water quality are based on output from the proprietary HYADEM model and recent operational experience at their S-154 prototype project in south Florida. Table 5.3-1 summarizes the expected finished water quality for the key parameters of concern (TN, TP, BOD, and TSS), for the single-stage WHS™ alternative 45% and 27% total nitrogen load reduction targets. Data and tools used to derive these estimates are included in Appendices D1 and D2.

5.3.5 Residuals Disposal

Sources of residuals requiring management include harvested hyacinth biomass and accumulated WHS™ sediments. The relative proportions of these are projected in Table 5.3-1. It is intended that all solids sources be managed through windrow composting.

The use of windrow composting to reduce and stabilize organic solids is a well-established process, with numerous large-scale facilities located throughout Florida and the United States. HydroMentia developed and implemented a design mix using the methodology developed by Haug (1993). This strategy was applied to the S-154 WHS™-ATS™ MAPs prototype, and resulted in a stable, high quality organic fertilizer/compost. The composting process results in a reduction of moisture to 40-45%, with a solids reduction of about 25%. The source material, composed of chopped hyacinths, algae and hay, achieved internal temperatures of about 55^{o} C during composting, resulting in a total weight loss of about 88%. The initial composting process lasted approximately 35 days, after which the material was stockpiled and cured for 60 days. This material is very high in nitrogen content (3.21%), which provides for a high quality organic fertilizer.

As a "best-case scenario", the finished product is marketable, and could be sold in bulk, or should market conditions so warrant, as a packaged product. As a "worst case scenario", the processed compost/organic fertilizer would be transported to a landfill for disposal. While HydroMentia is confident that this material could be marketed, the pricing included within the cost estimate includes a \$5.00/ton hauling cost plus a tipping fee of \$20.50/ton for the "worst case".

Sediments pumped from the WHS units on a quarterly basis will be dewatered in a thickening pond and then composted with the addition of processed water hyacinth compost. HydroMentia provides detailed descriptions of the solids handling and estimated solids balances in their proposals which can be found in Appendices D1 and D2.

An FDEP Environmental Resource Permit will be required for the construction of the proposed treatment system.

Local construction permits (City of Bartow, Polk County) may also be required due to the required installation of mechanical harvesting components.

Table 5.3-1 Estimated Effluent Water Quality for MAPS WHS™ System (Hydromentia, 2004b & 2005a)

Parameter	Annual Total Nitrogen Load Reduction Target	
	45 %	27 %
Process Size, Acres	210	88
Maximum Flow Rate, MGD	194	194
Average Flow Rate, MGD	32.12	32.12
Average Hydraulic Retention Time, days	8.52	3.57
Minimum Hydraulic Retention Time, days (@194 MGD)	1.41	0.59
Average Hydraulic Loading Rate cm/day	14.31	6.00
Nitrogen Removal kg/yr	132,108	80,801
Nitrogen Effluent Concentration mg/l	2.56	3.71
Nitrogen Areal Removal Rate g/m^2-yr	155	227
Phosphorus Removal kg/yr	15,138	8,277
Phosphorus Effluent Concentration mg/l	0.262	0.418
Phosphorus Areal Removal Rate g/m^2-yr	17.8	23.2
TSS Areal Loading Rate g/m^2-yr	6,005	14,330
TSS Areal Removal Rate g/m^2-yr	5.404	12,897
TSS Effluent Concentration mg/l	<12	<12
Wet/Dry Biomass Harvest tons/yr	52,756 / 3,429	25,407 / 1,651
WHS™ Wet/Dry Sediment Harvest tons/yr	26.680 / 1,334	11,262 / 563
Wet/Dry Growth tons/yr *	95,260 / 4,763	44,290 / 2,215
Annual Compost Production tons/yr	8,931	2,769
Annual Compost Production cy/yr	14,884	4,602

* See Comment 6, Appendix D2, page 45

5.3.6 Regulatory Requirements

The WHS™ will require an ERP and authorization from the USACE. In addition, the WHS ™ will require an Aquatic Plant Management Permit from the Florida Bureau of Invasive Plant Management for the stocking, harvesting, and disposal of water hyacinths. Water hyacinths are listed as Class I Prohibited Aquatic Plants (FAC 62C-52). The disposal of composted hyacinths may also require a Land Application Permit from the FDEP.

5.3.7 Capital, Operation and Maintenance Costs

The present worth calculation of capital, operation and maintenance costs for the MAPS alternatives are a combination of vendor costs for the proprietary WHS™ and estimated costs for the influent pump station and transmission main. Parsons provided unit costs to HydroMentia for general site, earthwork, and concrete tasks, a worksheet for calculating capital and O&M costs for the influent pump station, and worksheets for calculating present net worth. Detailed assumptions, quantities and cost estimates for the capital, operation and maintenance costs for each of the proposed single-stage WHS™ treatment alternatives is provided in the vendor's proposal included in Appendices D1 and D2. The costs for the influent pump station and transmission main are not included in the proposal and are computed separately (see Appendix D-3).

Capital costs include items such as:

1. Influent pump station and transmission main
2. Influent manifold flume
3. Four parallel WHS™
4. Influent and effluent structures
5. A network of 20-ft wide harvest roads of compacted stone to facilitate management and harvesting of the hyacinth crop
6. Effluent flume
7. Aeration channel
8. Composting pad
9. Paved access road from U.S. 17 to the facility, to include a security gate
10. Harvesting, processing and transport equipment
11. Grassing, erosion control and stormwater management, to include a perimeter swale
12. A perimeter security fence
13. Fuel and material storage facilities
14. Electrical distribution and controls
15. Tools and small engine items as required for system operation and maintenance
16. All elements as deemed necessary to meet applicable health and safety standards
17. Fees, profits and licenses for all proprietary technologies for the subject facility are included in quote

It is assumed that the Single-Stage WHS™ Treatment Facility will be operated by the SWFWMD or its agent with training provided by HydroMentia Inc. The costs included in the estimate below are:

1. All labor required to operate the facility as described, including all components identified above

2. All energy costs, including electricity and fuels as required to operate necessary equipment
3. All costs associated with the management, transport and landfilling of the residual solids as the "worst case" scenario
4. All expendables including chemicals, biological control agents, etc. as may be required to facilitate system performance, and the proper management of these agents
5. All equipment maintenance and replacement of damaged or expended equipment, and maintenance of necessary tools and spare parts to ensure expeditious repair of critical items
6. Contracting costs associated with the removal of sediments from within the WHS™ treatment unit

For each of the reduction targets the vendor submitted a best and worst case cost based on the disposal of compost alternative. The best case being the compost is sold to offset costs, the worst case being that hauling and tipping fees would have to be paid.

Parsons reviewed each of the estimates and added or deleted costs to be consistent with unit costs being applied to the other technologies being considered. All exceptions to the vendor supplied costs are highlighted and footnoted in a revised capital cost calculation worksheet given in Appendix D-3. In particular, HydroMentia assumed that the berms around the WHS cells would be constructed of in situ materials. The site locations indicated in Figure 5.3-1 and 5.3-2 are located in an area of reclaimed phosphatic clays encapsulated by a limited amount of cover material. For purpose of this conceptual level estimate, Parsons used a unit cost for construction of berms based on importing borrow material until such time that geotechnical studies can provide a definitive measure of in situ soil suitability.

Tables 5.3-2 and 5.3-3 present the estimated capital and operations costs for the 210-acre MAPS WHS™ System used in achieving the 45% reduction goal and the 88-acre MAPS WHS™ System used in achieving the 27% reduction goal, respectively. Capital costs range from $16.2 to $21.8 million. An initial technology performance fee required in using this technology which ranges from $291,000 to $445,000 has been included in the capital costs. Annual operations and maintenance (O&M) costs range from $1.1 to $1.9 million per year and include an annual technology performance fee of $89,000 to $146,000 per year. Equipment is estimated to be replaced every 20 years at an estimated cost of $4.1 million (2004 dollars). Using a present worth analysis, the estimated costs were estimated to be $5.42/lb TN to achieve the 45% reduction goal using the 210-acre MAPS WHS™ System and $5.68/lb TN to achieve the 27% reduction goal using the 210-acre MAPS WHS™ System.

Table 5.3-2 Estimated Capital and Operating costs for 210-acre MAPS WHS™ System needed to achieve 45% total nitrogen load reduction (Worst-Case Scenario - Landfill Disposal of Compost/Organic Fertilizer).

System	Capital Costs ($)	Annual Operating Costs ($)	Equipment Replacement Costs[1] ($)
Intake and Inflow Pump Station	$3,732,000	$261,000	$2,463,000
Inflow Transmission Main	$447,000	$5,000	$295,000
Pump Station Access Road	$660,000	$7,000	$436,000
Single Stage WHS Facility [2]	$13,865,000	$1,268,000	$1,104,000
Residuals Disposal		$228,000	
SUBTOTAL	$18,703,000	$ 1,767,000	$ 4,297,000
Land Acquisition [3]	0		
Engineering [4]	$2,806,000		
Technology Performance Fee [5]	$291,000	$146,000	
Total	$21,799,000	$1,913,000	$4,297,000
Present Worth Cost	$21,799,000	$53,165,000	$3,869,000
Total Present Worth Cost [6]	$ 78,832,000		
Per Pound Nitrogen Removed [7]	$ 5.42		

1 Replacement of equipment and material items every 20 years.
2 Equipment annual O& M estimated at 5% of capital costs, consistent with Hydromentia proposal May 2005.
3 Land costs were not included in this present worth analysis. These costs and their influence on the present worth analysis are discussed in further detail in Section 7.0.
4 Estimated as 15% of capital costs.
5 Technology Performance Fee. ($0.50 per lb of nitrogen removed) payable annually during years 1-18, Years 19 and 20 payable in advance based on performance estimate. 3% Inflation rate not applied to Technology Fee.
6 Estimated at an interest rate of 5.625% for a 50-year period. Annual O&M costs were inflated at 3% per year. Salvage of equipment purchased at 40 years estimated at 1/3 the purchase value at the end of 50 years.
7 Listed cost based on estimated per pound nitrogen removed by MAPS-WHS™ system over 50 year operating period.

Table 5.3-3 Estimated Capital and Operating costs for 88-acre MAPS WHS™ System needed to achieve 27% total nitrogen load reduction (Worst-Case Scenario - Landfill Disposal of Compost/Organic Fertilizer).

System	Capital Costs ($)	Annual Operating Costs ($)	Equipment Replacement Costs[1] ($)
Intake and Inflow Pump Station	$3,732,000	$261,000	$2,463,000
Inflow Transmission Main	$447,000	$5,000	$295,000
Pump Station Access Road	$650,000	$7,000	$429,000
Single Stage WHS Facility [2]	$8,844,000	$679,000	$939,000
Residuals Disposal		$71,000	
SUBTOTAL	$13,672,000	$ 1,021,000	$ 4,125,000
Land Acquisition [3]	0		
Engineering [4]	$2,051,000		
Technology Performance Fee [5]	$445,000	$89,000	
Total	$16,167,000	$1,110,000	$4,125,000
Present Worth Cost	$16,167,000	$30,660,000	$3,714,000
Total Present Worth Cost [6]	$ 50,540,000		
Per Pound Nitrogen Removed [7]	$ 5.68		

1 Replacement of equipment and material items every 20 years.
2 Equipment annual O& M estimated at 5% of capital costs, consistent with Hydromentia proposal May 2005.
3 Land costs were not included in this present worth analysis. These costs and their influence on the present worth analysis are discussed in further detail in Section 7.0.
4 Estimated as 15% of capital costs.
5 Technology Performance Fee. ($0.50 per lb of nitrogen removed) payable annually during years 1-18, Years 19 and 20 payable in advance based on performance estimate. 3% Inflation rate not applied to Technology Fee.
6 Estimated at an interest rate of 5.625% for a 50-year period. Annual O&M costs were inflated at 3% per year. Salvage of equipment purchased at 40 years estimated at 1/3 the purchase value at the end of 50 years.
7 Listed cost based on estimated per pound nitrogen removed by MAPS-WHS™ system over 50 year operating period.

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SECTION 6

PHYSICAL TREATMENT TECHNOLOGIES

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SECTION 6.0

PHYSICAL TREATMENT TECHNOLOGIES

6.1 APPLICABLE TREATMENT TECHNOLOGIES

Because nearly 70 percent of the total nitrogen discharging Lake Hancock through Structure P-11 is in the particulate form as discussed in Section 3.0, treatment technologies that employ physical removal of particulate matter should be capable of reducing nitrogen loads by as much as 60 percent. Technologies that were evaluated and discussed herein include:

- Coagulation followed by sedimentation ponds
- Coagulation followed by sedimentation basins
- Coagulation followed by filtration (direct filtration)
- Coagulation followed by sedimentation and filtration (conventional filtration)
- Coagulation followed by dissolved air flotation (DAF)
- Coagulation followed by DAF and filtration, and
- Microscreens

Coagulation followed by sedimentation is a technology commonly used in the treatment of drinking water. A coagulant is added to the raw water to form particles that are large enough to be removed by settling or by filtration. Particles in water typically have negative charges that repel each other preventing them from coming in contact with each other and thereby remaining in suspension. Positively charged metal-ion coagulants (i.e., alum, ferric sulfate and ferric chloride) are added to neutralize these charges allowing the suspended particles to make contact with each other and accumulate. This accumulation, which forms a larger particle known as a "floc", continues to grow and as a result gains enough mass to be removed by physical means either by settling in a sedimentation pond or basin and/or by filtration. The distinction made here between sedimentation "ponds" and sedimentation "basins" refers to the physical structure used for each. In the case of ponds, the generated floc is allowed to settle in large ponds where the accumulated floc or sludge is allowed to condense in the bottom of the pond for several months before being removed using a floating dredge. In the case of basins, the generated floc is allowed to settle in small basins utilizing enhanced technologies such as plate settlers or inclined tubes and allowed to accumulate for a short period of time (i.e., less than a day) before being removed by mechanical means (i.e., horizontal or rotating scraper arms).

Direct filtration (i.e., coagulation followed by filtration) is used primarily for filtering raw waters with low suspended solids and/or turbidity. Because the coagulated water is directly filtered without sedimentation, suspended solids in the raw water and those solids formed from the addition of a coagulant (i.e., aluminum and ferric hydroxides) are loaded directly on the filters. If the suspended solid or turbidity is sufficiently high, then filter cycles can be short requiring frequent cleaning, increased downtime, increased operator attention and increased backwash water waste, all of which can result in increased operating costs. As a rule of thumb, direct filtration is typically limited to waters with turbidities of 25 NTU or less (Tchobanoglous and Schroeder, 1985). Water higher in turbidity may be treated, but pilot testing is needed to define operating requirements and costs. Given the raw water turbidity can vary from 5 to over 100 NTU throughout the year (see Section 2 for review of raw water quality data) coupled with pilot filter results performed by ERD (ERD, 1999) which showed severe clogging requiring frequent backwashing (Telecommunications with Mr. Jeff Herr, ERD, May 6, 2004), direct filtration is not recommended and was not explored further as a treatment technology for Lake Hancock in this memorandum. Thus sedimentation is required after coagulation addition and only those physical treatment alternatives that included sedimentation were considered.

As an alternative of sedimentation, dissolved air flotation (DAF) was also considered. DAF was of particular interest in this case, because most of the particulate nitrogen is assumed to be associated with algae and DAF is typically more efficient at removing algae than conventional sedimentation. DAF is a clarification process that has been used in Europe, especially in Scandinavia and the United Kingdom, for the past 30-years. Like sedimentation, DAF is used in removing flocculated particles. DAF, however, relies on flotation of the flocculated particles rather than gravity settling. Thus, DAF is usually more efficient at removing low-density particles such as bacteria, algae and protozoan cysts (i.e., Giardia lamblia and Cryptosporidium parvuum oocysts). Because of the enhanced efficiency for removing algae, DAF has the potential of being much smaller than conventional sedimentation; hence, a considerable savings in capital costs can be realized. Hydraulic loading rates are typically 10 times higher than those used for sedimentation and detention times are typically much shorter, ranging from 5 to 15 minutes (AWWARF 1992) as compared with sedimentation, which can vary from 1.5 to 3.0 hours (AWWA/ASCE, 1998).

Because of the association of particulate nitrogen with algae, rotating disc microscreens were another treatment technology that was also explored. Microscreens are commonly used to pre-filter algae prior to drinking water treatment plants. Microscreens use woven fabric to filter the water and remove particulate matter, greater than 10-µm in size.

The Saddle Creek property, recently purchased by the District, between U.S. 98 and Lower Saddle Creek adjacent to Structure P-11 (see previous Figure 1.2-1) was selected as the site for the physical treatment systems. Water from Lake Hancock would be pumped via a pump station located on the shore upstream of Structure P-11. Two design capacities were considered. The first was designed to achieve an average annual 45% total nitrogen load reduction and the second was designed to achieve an average annual 27% total nitrogen load reduction for all discharges from Lake Hancock to Lower Saddle Creek. The reason for providing these two designs is discussed in Section 1.0. Per previous information presented in Section 2.0, the average annual lake discharge for design purposes is 42,463 ac-ft or 58.65-cfs (37.9-mgd). As discussed in Section 3.0, a raw water total nitrogen concentration of 5.530 mg/L and a raw water total phosphorus concentration of 0.603 mg/L were used for design.

6.2 SEDIMENTATION PONDS

6.2.1 General Description

The Sedimentation Ponds Alternative involves pumping water from Lake Hancock just upstream of Structure P-11 into multiple sedimentation ponds. Lake water flow rate would be measured in the transmission main extending from the lake water pump station to the sedimentation ponds and an appropriate dose of chemical coagulant would be added to the raw water for removal of total nitrogen and other pollutants. The chemical coagulant would react with the lake water in the transmission main prior to discharge into the multiple sedimentation ponds. The chemical precipitate would settle to the bottom of the sedimentation ponds. The treated water would discharge from the opposite end and return to Lower Saddle Creek downstream of Structure P-11. The chemical precipitate settling on the bottom of the sedimentation ponds would be dredged periodically and pumped into gravity thickeners and mechanical dewatering devices. Dewatered sludge would be pumped to multiple drying beds. The dried residual would need to be periodically removed from the drying areas. The water flow rate metering equipment, chemical injection equipment, chemical storage tanks, and other appurtenances would be placed in an operations and maintenance building located adjacent to the sedimentation ponds.

6.2.2 Design Parameters

As discussed in Section 4.0, the use of aluminum sulfate (alum) at a dose of 7.5 mg Al/liter results in a particulate nitrogen concentration reduction of approximately 89%. The total nitrogen removal efficiency is directly related to the ratio of particulate nitrogen to total nitrogen ratio (PN:TN) in the raw water. From October 1998-July 1999, surface water samples were collected and measured for particulate and total nitrogen at four locations in Lake Hancock and just upstream of Structure P-11. A summary of measured PN:TN for this monitoring period is provided in Table 6.2-1. The mean particulate PN:TN measured at Structure P-11 during this time period was 0.65. From March-October 2004, surface water samples were collected at three locations in Lake Hancock and just upstream of Structure P-11. A summary of measured PN:TN during this period is provided in Table 6.2-2. Through October 2004, the PN:TN at Structure P-11 is 0.69 at the top and bottom of the water column. Based on an average of the PN:TN during these two sampling periods, water discharging at Structure P-11 is expected to have an annual average PN:TN ratio of approximately 0.67.

Assuming that 67% of the total nitrogen is in the form of particulate nitrogen and an alum dose of 7.5 mg Al/liter will remove 89% of the particulate nitrogen, 76% and 46% of the annual lake discharge volume must be treated to achieve a 45% (i.e., 0.89 x 0.67 x 0.76 x 100% = 45%) and 27% (i.e., 0.89 x 0.67 x 0.46 x 100% = 27%) annual mass total nitrogen load reduction, respectively. This efficiency ignores minimal dissolved nitrogen removal which was observed during laboratory jar tests. Based on Figure 2-7, the sedimentation ponds would need to treat a peak design water flow rate of 190 ft^3/sec (cfs) and 68-cfs to achieve an overall 76% and 46% annual treatment volume, respectively. If the average annual lake discharge is 42,500 ac-ft, then the sedimentation ponds would need to treat 32,300 ac-ft/yr and 19,500 ac-ft/yr, respectively. At an alum dose of 7.5 mg Al/liter, approximately 1,450,000 gallons and 878,000 gallons of alum would be required each year to treat 76% and 46% of the average annual flow. This would result in generating approximately 320 ac-ft and 200 ac-ft, respectively, of alum sludge in the bottom of the sedimentation ponds each year. The alum sludge is expected to dry to less than 5% of the initial wet volume in the drying areas. Therefore, the estimated annual dried sludge volume is 16 ac-ft or 26,000 yd^3 and 10 ac-ft or 15,800 yd^3, for each respective design. At a peak design flow rate of 190-cfs and 68-cfs, 16,700 gallons and 10,100 gallons of alum would be required each day, generating approximately 4 ac-ft and 2.4 ac-ft of alum sludge per day, respectively.

Table 6.2-1 Summary of Particulate Nitrogen: Total Nitrogen Ratios (PN:TN) for Lake Hancock Surface Water Samples Collected from 10/98 to 7/99

Surface Water Sample Collection Date	Particulate N : Total N Ratio at Sample Collection Location				
	Site 1	Site 2	Site 3	Site 4	P-11
10/9/98	0.24	0.37	0.46	0.37	--
11/3/98	0.82	0.80	0.75	0.71	--
12/10/98	0.76	0.86	0.77	0.75	--
1/19/99	0.63	0.63	0.54	0.55	--
2/27/99	0.57	0.28	0.38	0.37	0.62
3/17/99	--	--	--	--	0.65
3/26/99	0.82	0.81	0.76	0.71	0.69
4/2/99	--	--	--	--	0.61
4/16/99	--	--	--	--	0.71
4/30/99	--	--	--	--	0.73
5/11/99	0.68	0.60	0.56	0.47	0.52
6/10/99	0.72	0.75	0.78	0.78	0.63
7/1/99	0.84	0.75	0.79	0.78	0.70
Mean	0.68	0.65	0.64	0.61	0.65

For each design, three parallel sedimentation ponds were provided, each capable of providing 3 hours of detention time at ½ the peak flow rate plus 30-days of floc storage. This was done so that two sedimentation ponds could be in-service while the third pond could be off-line for maintenance and dredging. To provide 3 hours of detention time and 30 days of floc storage, each pond was designed at a volume of approximately 80 ac-ft for the 190-cfs peak flow and 30 ac-ft for the 68-cfs peak flow. The ponds were located in an area of lower existing ground surface elevations closer to Lower Saddle Creek. The remaining available site area with higher existing ground surface elevations would be used for sludge drying and storage. Soil excavated for creation of the sedimentation ponds would be used to construct berms around portions of the sedimentation ponds, berms around the proposed sludge drying and storage areas, and for raising the ground surface elevation of areas to be used for sludge drying.

Table 6.2-2 Summary of Particulate Nitrogen: Total Nitrogen Ratios (PN:TN) for Lake Hancock Surface Water Samples Collected from 3/04 to 10/04

Surface Water Sample Collection Date	Particulate N : Total N Ratio at Sample Collection Location			
	Site 1	Site 2	Site 3	P-11
3/5/04	0.67	0.57	0.59	--
4/15/04	0.64	0.49	0.54	0.70 (top) 0.62 (bottom)
5/18/04	0.71	0.70	0.78	--
7/9/04	0.78	0.74	0.78	--
7/20/04	0.77	0.70	0.74	0.75 (top) 0.85 (bottom)
8/6/04	0.79	0.76	0.78	--
8/20/04	0.67	0.66	0.48	--
9/9/04	0.65	0.63	0.78	--
9/28/04	0.44	0.05	0.23	--
10/25/04	0.66	0.58	0.61	0.62 (top) 0.61 (bottom)
Mean	0.68	0.59	0.63	0.69 (top) 0.69 (bottom)

6.2.3 Conceptual Design

The conceptual plan for the 190-cfs and 68-cfs sedimentation ponds alternatives is provided in Figure 6.2-1 and 6.2-2, respectively. Raw water would be withdrawn from Lake Hancock immediately upstream of Structure P-11 and pumped through an inflow transmission main to three sedimentation ponds. Water flow rate would be measured as it passes through the raw water inflow transmission main and alum would be added into the raw water transmission main downstream of the raw water inflow pumps. Inflow control structures with motorized valves would be constructed at the inflow points to each sedimentation pond and used to control the flow rate. The motorized valves would be controlled from the equipment/storage tank building located near the southwest corner of the parcel.

Each of the three sedimentation ponds were designed to treat half of the peak flow up to 95 cfs of raw water. The ponds would have a water surface elevation of approximately 101 ft (NGVD 29), top-of-bank elevation of 103 ft (NGVD 29) and a bottom elevation of 91.5 ft (NGVD 29).

The pond dimensions at top-of-bank are approximately 470-ft x 970-ft at 190-cfs and 170-ft x 350-ft at 68-cfs. The sedimentation ponds would have a 15-ft maintenance berm around the entire perimeter. Treated water from each of the ponds would discharge through a control structure which includes a motorized weir gate to control pond water surface elevation. Controls for the weir gates would be located in the equipment/storage tank building. The outfall weir elevation can be varied from 100-102 ft (NGVD 29). Water from the outfall control structures would discharge through 6-ft x 5-ft concrete box culverts into the outflow canal. The outflow canal would extend approximately 1500 ft to Lower Saddle Creek. The outflow canal has a 5-ft bottom width at elevation 93 ft (NGVD 29) and a 10-ft top width at elevation 103 ft (NGVD 29).

The equipment/chemical storage tank building will house chemical storage tanks, alum feed pumps and control panel, the valve and weir gate control panel, the stormwater flow meter electronics, the alum flow meter, and miscellaneous piping and electrical equipment. Alum will be pumped from the equipment/storage tank building to the point of alum addition through a PVC conduit. An alum dose of 7.5 mg Al/liter will be maintained for all water flow rates up to 190 cfs. A PRGS conduit will extend from the point of flow measurement in the raw water transmission main to protect the flow meter sensor cables.

The flood insurance rate map shows a 100-year flood zone elevation of 102 ft (NGVD 29) in this area. Some portions of the sedimentation pond areas are below the 100-year flood zone elevation. Filling to construct berms around the sedimentation ponds will reduce storage within the 100-year flood zone. Compensating storage for this fill will be provided by the excavation of the sedimentation ponds below the existing 100-year elevation. Material excavated for sedimentation pond construction will be used to construct berms around lower portions of the sedimentation ponds, berms for the outfall canal, berms around the proposed floc drying/storage area, and to fill lower portions of the sludge drying beds. No material will be hauled off-site.

The accumulated sludge will be pumped from the bottom of the sedimentation ponds to two 1-million gallon gravity sludge thickeners by remote-controlled portable dredge units. A rail system will be constructed on each long side of each of the three sedimentation ponds. A cable will extend from the rail systems to the portable dredge located in each of the sedimentation ponds. The dredges can be remote-controlled from the control panels located adjacent to maintenance berms to remove floc material from any point in the sedimentation pond. The discharge from the dredges will connect to an 8-inch camlock located between each of the ponds. An 8-inch floc discharge line will extend from the camlock connection to the gravity thickeners. Costs presented later in this section include two dredges, control panels, and rail/cable traverse system for all three ponds. Following gravity thickening, sludge will be mechanically dewatered by belt filter press. Dewatered sludge will then be pumped to four sludge drying beds. Each sludge drying bed would be capable of holding a total of three months of 15% dewatered sludge at the peak water flow rate. Based on the 1-ft sludge depth, four 11.5-acre drying beds are required at 190-cfs and four 3-acre drying beds are required at 68-cfs. The bottom of the drying bed would include 12 inches of stabilized soil and 12 inches of crushed concrete. Front-end loaders would be used to distribute sludge in drying beds and to load dried sludge onto trucks. Paved maintenance drives will be constructed on the north and south sides of the parcel to provide access for delivery trucks, solids hauling trucks, and general maintenance vehicles.

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SOUTHWEST FLORIDA WATER MANAGEMENT DISTRICT

LAKE HANCOCK OUTFALL TREATMENT PROJECT ALTERNATIVE TREATMENT TECHNOLOGIES

Fig. No. 6.2-1

190 CFS SEDIMENTATION PONDS CONCEPTUAL PLAN

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SOUTHWEST FLORIDA WATER MANAGEMENT DISTRICT

LAKE HANCOCK OUTFALL TREATMENT PROJECT
ALTERNATIVE TREATMENT TECHNOLOGIES

Fig. No. 6.2-2

68 CFS SEDIMENTATION PONDS CONCEPTUAL PLAN

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6.2.4 Expected Finished Water Quality

A summary of estimated annual raw water and treated water mass loadings and load reductions for total nitrogen and total phosphorus are provided in Table 6.2-3. Treating 76% of the flow, the 190-cfs sedimentation pond design is expected to reduce total nitrogen loadings by approximately 131,200 kg/yr (45.3%) and total phosphorus loadings by 21,600 kg/yr (68.4%). Treating 46% of the flow, the 68-cfs sedimentation pond design is expected to reduce total nitrogen loadings by approximately 78,200 kg/yr (27%) and total phosphorus loadings by 13,000 kg/yr (41%).

Table 6.2-3 Estimated Annual Pollutant Load Reductions for the Lake Hancock Outfall Treatment Project Sedimentation Pond Alternative

Parameter	Raw Water Load (kg/yr)	Treated Water Load (kg/yr)	Load Reduction	
			(kg/yr)	(%)
45% Nitrogen Reduction Goal (Treatment of 76% of annual average flow, peak capacity = 190-cfs)				
Total Nitrogen	289,600	158,400	131,200	45
Total Phosphorus	31,600	10,000	21,600	68
27% Nitrogen Reduction Goal (Treatment of 46% of annual average flow, peak capacity = 68-cfs)				
Total Nitrogen	289,600	211,400	78,200	27
Total Phosphorus	31,600	18,600	13,000	41

6.2.5 Operation and Maintenance Requirements

Operation and maintenance of the sedimentation ponds includes alum injection, water control equipment, remote-control dredging, mechanical dewatering equipment, front-end loaders, trucks, and general maintenance of floc settling and sludge drying beds, handling and loading of dried sludge for truck hauling, and general building maintenance. Liquid aluminum sulfate (alum) will also need to be purchased. Approximately 1,300,000 gallons of liquid aluminum sulfate will be required each year. Power consumption for the chemical treatment system and controls, remote-control dredge, and miscellaneous items is estimated at 100,000 kwh/yr. This does not include the power requirements of the raw water pump station, belt filter presses, or sludge pumps. Anticipated average annual labor requirements are four full-time operators/maintenance personnel or 160 hours/week. Labor requirements will be lower during low flow conditions and higher during high flow conditions.

6.2.6 Residuals Disposal

Operation of the sedimentation ponds will produce approximately 38,000 yd^3 of semi-dry material in an average annual rainfall year. Alum residuals have additional phosphorus and heavy metal adsorption capacity and can provide a beneficial use. The St. Johns River Water Management District (SJRWMD) has successfully utilized water treatment plant alum residual from the Lake Washington water treatment plant in Melbourne, FL to reduce phosphorus export from marsh flow-way areas following periodic maintenance. A Technical Publication titled "The Application of Alum Residual as a Phosphorus Abatement Tool within the Lake Apopka Restoration Area" (Hoge, et al.) is provided in Appendix E. The SJRWMD and the Florida Department of Environmental Protection (FDEP) performed numerous tests on the Lake Washington water treatment plant residual, including pesticide scans, biological analysis, toxicity characteristic leaching procedure (TCLP), and synthetic precipitation leaching procedure (SPLP). Based on sediment guidelines for the State of Florida, only arsenic was present in levels that presented a potential environmental hazard. The arsenic concentration of the water treatment plant residual is slightly higher but not significantly different from the arsenic content of the surface sediments of the former farm fields on the north shore of Lake Apopka. The application did not significantly change the soil concentration of arsenic in the top 6 cm. The SJRWMD applied 6.5 wet tons/acre to approximately 2000 acres of marsh flow-way in 1999.

Due to the phosphorus and heavy metal binding capacity of the aluminum sulfate residual, the material may also be useful in reducing water and sediment phosphorus concentrations in former mining pits. The Polk County area in and around Lake Hancock was extensively mined, leaving multiple open wet borrow areas with high water and sediment phosphorus concentrations. The application of alum residuals in these pits could significantly reduce water and sediment phosphorus concentrations. The least desirable option for residuals disposal is to take all semi-dry material to a Class I landfill. The cost to dispose of the material at a Class I landfill would be almost an order of magnitude higher than the previous two alternatives. Another option would be to haul the semi-dry material to the landfill and allow the landfill to use the material as daily cover and not pay the disposal fee.

6.2.7 Regulatory Requirements

Environmental Research & Design, Inc. (ERD) and the Southwest Florida Water Management District (SWFWMD) recently met with FDEP in Tampa, FL to discuss the beneficial use of alum residual from the McIntosh Park Enhanced Treatment Wetland and the Eastshore Commerce Park Parcel Stormwater Retrofit Project to amend constructed wetland soils at the McIntosh Park Enhanced Treatment Wetland site. FDEP indicated that, per Chapter 62-701 FAC, solid wastes must be taken to a permitted or exempt solid waste facility. Disposal is defined to include the placing of any solid waste into any pond, land, or water. Section 403.7045 of the Florida Statutes allows the Department to exempt materials which are beneficially used from regulation of solid wastes. To be exempt, the materials must meet the following three conditions:

1. A majority of the industrial byproducts are demonstrated to be sold, used, or re-used within one year;

2. The industrial byproducts are not managed as to create a threat of contamination in excess of Department standards and criteria; and

3. The industrial byproducts are not hazardous wastes

In order to demonstrate that the byproducts are not hazardous wastes, testing similar to that performed by SJRWMD will need to be performed on the alum residuals, including TCLP and

SPLP. The Department is most concerned about heavy metals in the alum residual, with specific concern related to arsenic. To receive authorization for use of the alum residual, information is submitted to the Solid Waste Department at the FDEP office in Tallahassee. If the material meets the three conditions, a letter is issued which allows approval of the beneficial use of the material.

6.2.8 Capital, Operation, and Maintenance Costs

A summary of the estimated capital costs for the 190-cfs (i.e., 45% nitrogen load reduction design) and 68-cfs (27% nitrogen load reduction design) sedimentation ponds is provided in Tables 6.2-4 and 6.2-5, respectively. A breakdown of capital costs is provided in Appendix F, Tables 2 and 9. The estimated capital cost includes all components needed to complete the sedimentation ponds. All excavated soil will be used to construct embankments around the sedimentation ponds and the sludge drying beds. The cost includes the purchase of front-end loaders to handle semi-dry floc material and to load the material into trucks for hauling off-site and the purchase of two dredges and control panels and all accessories.

A breakdown of operating costs is provided in Appendix F, Tables 3 and 4 respectively, for the 190-cfs and 68-cfs designs. Operating costs include purchase of aluminum sulfate (alum); power to operate the chemical feed system, water flow, and chemical feed system controls; portable dredges; and other miscellaneous items. The cost also includes labor to operate the chemical feed system, portable dredges, gravity thickeners, belt filter presses, general area maintenance, and loading of dried sludge into trucks for hauling by front end loaders. The labor item also includes the cost of fuel and miscellaneous materials required for general maintenance. It was assumed that the dried alum sludge would need to be disposed of in a Class I landfill at a unit cost of $25.50/ton. If a beneficial use can be identified, annual operating costs would be lower.

A summary of present worth cost per pound of total nitrogen removed for the sedimentation ponds is provided at the bottom of Table 6.2-4 and 6.2-5 for the 190-cfs and 68-cfs designs, respectively. This table includes capital cost, annual operating cost, equipment replacement cost, engineering contingencies, present worth cost, and present worth cost per pound of total nitrogen removed. The assumptions are listed at the bottom of the table. The present worth cost is $7.92 per pound of total nitrogen ($/lb TN) removed for the 190-cfs sedimentation pond and is $8.15 per pound of total nitrogen ($/lb TN) removed for the 68-cfs sedimentation pond.

6.3 SEDIMENTATION BASINS

In the context of this memorandum, sedimentation basins refer to a three step treatment process that includes: chemical coagulation, flocculation and sedimentation. Unlike sedimentation ponds, as discussed in Section 6.2, the process of coagulation and flocculation is carried out in a series of separate basins rather than in one pond.

Table 6.2-4 Capital and Operating Costs for 190-cfs Capacity Sedimentation Ponds needed to achieve 45% total nitrogen load reduction goal.

System	Capital Costs ($)	Annual Operating Costs ($)	Equipment Replacement Costs[1] ($)
Clearing and Grubbing	$303,000		
Earthwork	$653,000		
Intake and Inflow Pump Station	$2,688,000	$258,000	$1,774,000
Inflow Transmission Main	$1,788,000	$18,000	
Sedimentation Ponds	$1,319,000	$310,000	$500,000
Discharge Channel	$1,184,000	$12,000	
Gravity Thickening	$1,935,000	$101,000	$1,278,000
Mechanical Dewatering	$8,990,000	$470,000	$5,934,000
Sludge Drying Beds	$2,654,000	$623,000	
Operations & Maintenance Bldg	$3,600,000	$54,000	
Alum Metering and Storage Facilities	$1,530,000	$860,000	$1,010,000
Pavement	$842,000	$9,000	
SUBTOTAL	$27,483,000	$ 2,712,000	$ 10,495,000
Land Acquisition [2]	0		
Engineering [3]	$4,123,000		
Total	$31,605,000	$2,712,000	$10,495,000
Present Worth Cost [4]	$31,605,000	$78,890,000	$9,447,000
Total Present Worth Cost	$ 119,941,000		
Per Pound Nitrogen Removed [5]	$ 7.92		

1 Replacement of equipment and material items every 20 years.

2 Land costs were not included in this present worth analysis. These costs and their influence on the present worth analysis are discussed in further detail in Section 7.0.

3 Estimated as 15% of capital costs.

4 Estimated at an interest rate of 5.625% for a 50-year period. Annual O&M costs were inflated at 3% per year. Salvage of equipment purchased at 40 years estimated at 1/3 the purchase value at the end of 50 years.

5 Listed cost based on estimated per pound nitrogen removed by sedimentation ponds over 50 year operating period.

Table 6.2-5 Capital and Operating Costs for 68-cfs Capacity Sedimentation Ponds needed to achieve 27% total nitrogen load reduction goal.

System	Capital Costs ($)	Annual Operating Costs ($)	Equipment Replacement Costs[1] ($)
Clearing and Grubbing	$121,000		
Earthwork	$329,000		
Intake and Inflow Pump Station	$1,501,000	$153,000	$991,000
Inflow Transmission Main	$1,208,000	$13,000	
Sedimentation Ponds	$1,106,000	$310,000	$500,000
Discharge Channel	$840,000	$9,000	
Gravity Thickening	$1,200,000	$63,000	$792,000
Mechanical Dewatering	$5,500,000	$288,000	$3,630,000
Sludge Drying Beds	$1,234,000	$385,000	
Operations & Maintenance Bldg	$1,800,000	$27,000	
Alum Metering and Storage Facilities	$790,000	$514,000	$522,000
Pavement	$597,000	$6,000	
SUBTOTAL	$ 16,233,000	$ 1,763,000	$ 6,434,000
Land Acquisition [2]	0		
Engineering [3]	$2,434,000		
Total	$18,657,000	$1,763,000	$6,434,000
Present Worth Cost [4]	$18,657,000	$51,284,000	$5,792,000
Total Present Worth Cost	$ 75,732,000		
Per Pound Nitrogen Removed [5]	$ 8.15		

1 Replacement of equipment and material items every 20 years.
2 Land costs were not included in this present worth analysis. These costs and their influence on the present worth analysis are discussed in further detail in Section 7.0.
3 Estimated as 15% of capital costs.
4 Estimated at an interest rate of 5.625% for a 50-year period. Annual O&M costs were inflated at 3% per year. Salvage of equipment purchased at 40 years estimated at 1/3 the purchase value at the end of 50 years.
5 Listed cost based on estimated per pound nitrogen removed by sedimentation ponds over a 50 year operating period.

6.3.1 General Description

Coagulation is a process by which chemical coagulants are added to assist in the removal of suspended particles. Particles in water typically have negative charges that repel each other preventing them from coming in contact with each other and thereby remain in suspension. Positively charged metal-ion coagulants (i.e., alum, ferric sulfate and ferric chloride) are added to neutralize these charges allowing the suspended particles to make contact with each other and accumulate. This accumulation, which forms a larger particle known as a “floc”, continues to grow and as a result gains enough mass to be removed by physical means through sedimentation and/or filtration.

Flocculation refers to a process by which the coagulated particles are forced to make contact with each other forcing them to accumulate producing a “floc”. This can occur by static means using baffled channel flocculator, static mixers or by mechanical means using motor-driven paddles installed into multiple staged basins. This is in contrast with the design of sedimentation ponds where flocculation is carried out along with sedimentation in the same pond. Mixing in this case is provided by changes in water current as provided by the incoming flow or by wind action at the surface of the pond.

During sedimentation, flocculated particles are allowed to settle by providing ponds or basins that are large enough to provide sufficient quiescent time for the “flocs” to settle and be captured. A discussion of sedimentation ponds is provided in Section 6.2. There are many different designs of sedimentation basins. The simplest design is a rectangular basin with a slanted floor to one end provided with drains for periodic removal of the consolidated “floc” particles, also referred to as sludge. However, this design is usually labor intensive to remove the sludge. Other, more common designs include mechanical sludge removal equipment such as motor-driven chained flights used in rectangular basins to scrape settled sludge to hoppers where pumps are used to remove the sludge. Circular settling basins are also commonly used with rotating scrapers that scrape sludge to center hoppers where pumps are used in removing the sludge. Sedimentation basins can also be equipped with enhancements such as inclined plates or tubes referred to as plate and tube settlers, respectively, that can increase both treatment and cost efficiency.

6.3.2 Conceptual Design

Based on jar test results as discussed in Section 4.0, a 7.5 mg/L (as Al^-) alum dose reduced particulate nitrogen by 89 percent. As an aside, this is an equivalent dose of 165 mg/L (as $Al_2(SO_4)_3$ + 14 H_2O) which is more commonly the units expressed in the drinking water industry. Although this dose is relatively high, it is not uncharacteristic for treatment of Floridian surface waters. Based on the average particulate nitrogen to total nitrogen ratio estimated at 0.67 (see discussion of raw water quality in Section 3.0), the projected average reduction in total nitrogen is 60 percent. This assumes no reduction of dissolved nitrogen which was shown to vary from 0 to 65 percent. Therefore to achieve a 45% or 27% reduction in total nitrogen loading will require 76% or 46% of the discharged volume to be treated, respectively. This will require the sedimentation basin facility to be sized for a maximum capacity of 190-cfs (123-mgd) or 68-cfs (44-mgd) respectively, which by drinking water standards, both would be considered large water treatment facilities. Because the treated annual average discharge flowrate to achieve a 45% reduction is 44-cfs (28-mgd) and to achieve 27% is 27-cfs (17.5-mgd), most of the facilities would be idle and out-of-service during most of the year. Only during the rainy or wet seasons would the full capacity of the system be needed and used.

Figure 6.3-1 shows the process flow diagram for both facilities. Figures 6.3-2 and 6.3-3 show the layout for the proposed 190-cfs (123-mgd) and 68-cfs (44-mgd) sedimentation basin facilities. Both facilities consist of the following major processes:

- Inflow intake and pump station
- Inflow transmission main
- Mechanical rapid mix
- Flocculation
- Sedimentation
- Discharge channel and outfall structure
- Alum storage and metering facilities
- Gravity thickeners
- Mechanical Dewatering
- Sludge drying beds
- Surge basin

Also included on-site is an operations and maintenance building.

To accommodate fluctuating flowrates, the treatment facility was divided into equal sized trains based on a hydraulic capacity. Trains can either be placed into service or taken out of service based on the flowrate of water needed to be treated. If the flowrate is less than one train's capacity, then only one train is needed and operated. Although the train would be hydraulically oversized, its performance would not be affected.

The raw water upstream from the P-11 control structure would be drawn into the raw water pump station through a set of parallel bar screens to remove fish, debris and trash from entering the pump wet well. A series of submersible pumps installed in the well would be used to pump the raw water into a transmission main where the water would be conveyed and split into four rapid mix basins, each sized for one quarter the maximum capacity. Liquid alum would be injected and mixed in the rapid mix basins using vertically mounted, high speed mixers. From the rapid mix basins, the now coagulated water would flow into the two-staged, flocculation basins where vertically mounted, low speed mixers would be used to slowly mix the water allowing the coagulated particles to make contact with each other forming flocculated particles or "floc". The flocculated water would then flow into the plate settler, sedimentation basins where the floc would be allowed to settle and collect, generating sludge at the bottom. The resultant water for the plate settlers would then discharge directly into the discharge channel where it would be conveyed by gravity into Saddle Creek.

The accumulated floc or sludge from the bottom of the plate settlers would be withdrawn and transferred to gravity thickeners where the sludge would be allowed further concentrate. Typically, gravity thickeners will concentrate the sludge from 1 to 2 percent solids discharging sedimentation to 3 to 4 percent solids which is equivalent to a 50 percent decrease in water. Decanted water from the thickeners will flow by gravity to the surge basin where the water will be returned ahead of the rapid mix basins. Thickened sludge from the gravity thickeners will be pumped to belt filter presses located in the mechanical sludge dewatering building. Belt filter presses will typically increase the solids content of alum sludges to 15 to 20 percent solids. The resultant filtrate will flow by gravity to the surge basin where the water will be returned ahead of the rapid mix basins. The dewatered sludge would discharge into hoppers where it would be pumped to the four sludge drying beds where it would be distributed along each bed using a header pipe connected with flexible hoses. Front end loaders would be used to distribute the sludge evenly across the bed for drying and used in transferring the dried sludge to hauling trucks. The sludge would then be transferred and either land applied or disposed into a landfill. Typically, the sludge needs to be dewatered to at least 30 percent solids to prevent leakage onto the roadway during hauling.

6.3.3 Operation and Maintenance Requirements

Operation and maintenance of the facility described would be comparable to that needed at a large scale drinking water or wastewater treatment facility. To put this in perspective, the 190-cfs (123-mgd) facility is nearly twice the capacity of Tampa Bay Water's new regional surface water treatment plant rated at 66-mgd or 100-cfs.

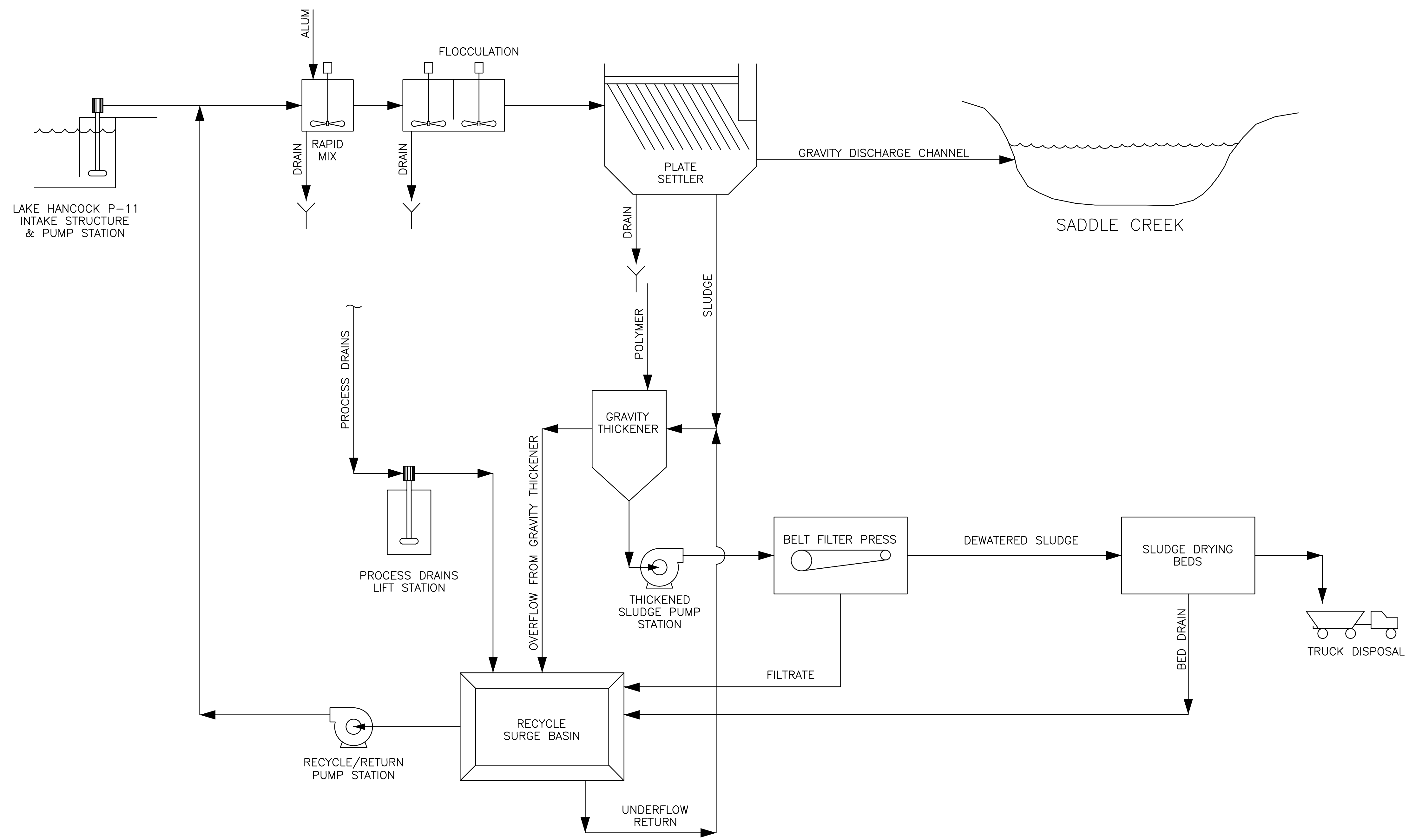
Operation. Depending on the discharge flowrate from P-11 as measured by lake level, the pumps at the inflow pump station can be placed in or out of service as needed to capture the required flow for treatment. Trains can also be placed in or out of service, but the decision to do so should be based on climatic changes, weekly and/or monthly trends and not diurnal fluctuations.

Processing and management of sludge will probably be the most labor intensive aspect of operating the facility due to the mechanical equipment involved and the need to transfer dewatered sludge to drying beds (see discussion provided in Section 6.3.5). Belt filter presses are simplistic in concept, but can be problematic to operate unless properly maintained.

Maintenance. Maintenance of the facility should focus primarily on maintaining:

- Bar screens and submersible pumps at the inflow pump station,
- Sedimentation basin flight and chain mechanisms,
- Alum chemical metering pumps,
- Sludge transfer and surge basin return pumps, and

Belt filter presses.



NOT TO SCALE

FIGURE NO. 6.3-1

SEDIMENTATION BASIN
TREATMENT FACILITY
PROCESS FLOW DIAGRAM

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FIGURE NO. 6.3-2

190 CFS SEDIMENTATION BASIN
TREATMENT FACILITY
SITE PLAN

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FIGURE NO. 6.3-3

68 CFS SEDIMENTATION BASIN
TREATMENT FACILITY
SITE PLAN

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Because a large portion of the facility will not be operated during certain periods of the year, there should be ample opportunity to maintain the equipment without the need to take operating systems out of service. However, because the equipment will be idle for lengthy periods of time, maintenance will need to protect exposed equipment from corrosion and operations will need to rotate equipment on a frequent basis to maintain operability.

Staffing. Because the water is treated and discharged to a receiving stream, the U.S. Environmental Protection Agency (EPA) and Florida Department of Environmental Protection (FDEP) will probably view this facility as a wastewater treatment plant and its discharge as treated wastewater, requiring a National Pollutants Discharge Elimination System (NPDES) Permit. As a consequence, FDEP under Section 62-699 of the Florida Administrative Code (F.A.C.) establishes minimum staffing requirements which would include staffing by a Class C of higher operator, 24 hours per day, 7 days per week and the lead/chief operator to be Class A. For plants under electronic surveillance or automated control, daily staffing hours may be reduced upon written approval from FDEP. Given the capacity of the proposed facility, the equipment that will need to be operated and maintained and the quantity of solids that will need to be processed, staffing requirements could vary between 5 to 8 full-time employees depending on the volume of water treated. More staff would be needed at higher flowrates to maintain equipment and transfer sludge in and out of drying beds. During times where water is not discharging the lake, only a minimum staff would be required to maintain the equipment.

6.3.4 Expected Finished Water Quality

Table 6.3-1 lists the average raw water and average treated water quality from jar testing at an alum dose of 7.5 mg/L (as Al^-) followed by 3-hours of sedimentation. In addition, listed in the far right column are the percent reductions that would be expected from a sedimentation facility as described in this section.

From the standpoint of improving water quality, nutrients and the clarity of the treated water should be drastically improved by the sedimentation facility. Nutrients such as total nitrogen, total phosphorus and biochemical oxygen demand (BOD) are expected to be reduced by 60, 95 and 80 percent, respectively. The clarity of the water as measured by turbidity, total suspended solids (TSS), color and Chlorophyll-a are also expected to be reduced by 95, 85, 90 and 99 percent, respectively.

Issues that may be a concern, however, is the reduction of pH, alkalinity and the increase chloride, sulfate and dissolved aluminum levels as a result of adding alum. Alum reacts with bicarbonate alkalinity (HCO_3^-) according to the following reaction:

$$Al_2(SO_4)_3 \bullet 14.3H_2O + 6HCO_3^- \rightarrow 2Al(OH)_3 \downarrow + 6CO_2 + 14.3\,H_2O + 3SO_4^{2-} \qquad \text{(EQ. 6.3-1)}$$

Based on stoichiometry, 1 mg/L of alum (as $Al_2(SO4)_3$ + 14 H_2O) reacts with 0.24 mg/L (as $CaCO_3$) of bi-carbonate (HCO_3^-) alkalinity. Jar test results indicate a slightly lower value of 0.20 mg/L of bi-carbonate (HCO_3^-) alkalinity was consumed per 1 mg/L of alum added ((as $Al_2(SO_4)_3$ + 14 H_2O).

Table 6.3-1 Expected Treatment Performance for Alum Coagulation Followed by 3-hours of Sedimentation at a 7.5 mg/L (as Al^-) Alum Dose

Parameter	Units	Lake Hancock Water Quality[1]	Treated Water Quality[1]	Expected Percent Reduction[2]
pH	s.u.	8.21	6.47	10 to20
Conductivity	µmho/cm	171.5	200	(-10 to -20)[3]
Alkalinity	mg/l	54.25	21.0	40 to 60
Ammonia	µg/l	575	658	0
Nitrate and Nitrite	µg/l	<5	23	0
Dissolved Organic Nitrogen	µg/l	380	258	0[4]
Particulate Nitrogen	µg/l	2382.5	253	85 to 95
Total Nitrogen	µg/l	3346	1181	50 to 60
Soluble Reactive Phosphorus (SRP)	µg/l	216.5	< 1	90 to 95
Dissolved Organic Phosphorus	µg/l	34.5	7	70 to 80
Particulate Phosphorus	µg/l	235	16	80 to 90
Total Phosphorus	µg/l	486	23	90 to 95
Turbidity	NTU	20.1	0.5	90 to 95
Total Suspended Solids (TSS)	mg/l	27.85	3.1	80 to 90
Biochemical Oxygen Demand (BOD)	mg/l	10.4	< 2.0	70 to 80
Color	Pt-Co	97	8	80 to90
Chlorophyll-a	mg/m^3	107.5	0.5	95 to 99
Calcium	mg/l	22.95	20.6	5 to 10
Chloride	mg/l	11.8	12.7	(-10 to -20)[3]
Sulfate	mg/l	10.5	45	(-300 to -400)[3]
Dissolved Aluminum	µg/l	46	57	(-20 to -30)[3]

1 Average of jar test results collected on September 17 and October 25, 2004. Jar test results collected on September 28, 2004 were not included as results seemed to differ from average raw water quality.
2 Based on jar test results discussed in Section 4.0. Values have been rounded.
3 Negative values indicate an increase in concentrate as a result of adding alum.
4 Reduction was variable from -55 to + 65 percent. Assumed as zero.

Although not considered in the conceptual design of this facility, pH adjustment may be needed to increase pH and alkalinity levels in the treated water prior to discharge into Saddle Creek. Per Chapter 62-600.455 of the F.A.C., all wastewater treatment "facilities shall be designed and operated to maintain the pH in the reclaimed water or effluent, after disinfection, within the range of 6.0 to 8.5 S.U." The pH of the treated water as shown by jar testing is marginally acceptable at a pH of 6.47 S.U. FDEP would need to be consulted to determine if pH adjustment would be required. If pH adjustment is needed, typically lime would be used for a facility of this size due to the costs of lime compared with other alternatives such as sodium hydroxide which is substantially more costly. The use of lime would require storage silos and lime slakers to process the lime into a slurry and an additional rapid mix to blend the lime slurry into the treated water.

6.3.5 Residuals Disposal

A significant aspect to the operations of the sedimentation facility will be processing and handling of the generated sludge and the associated costs. During alum coagulation, sludge is produced from the capture of suspended solids and the formation of aluminum hydroxide particles. As described by Equation 6.3-1 for every 1 mg of alum added, 0.44 mg of aluminum hydroxide particles are formed which adds to the sludge produced. Table 6.3-2 and 6.3-3 provides a summary of the annual average and maximum daily sludge quantities as generated from sedimentation and processed through gravity thickening, mechanical dewatering and sludge drying beds. Maximum sludge quantities are approximately 4.5 times greater than annual average sludge quantities requiring sludge handling equipment be oversized to accommodate this increased production.

Table 6.3-2 Expected annual average sludge quantities from each step in the sludge handling process in reducing total nitrogen loads by 45% and 27%.

Sludge Handling Process	Assumed Percent Solids	Annual Average Sludge Production 45% load reduction goal		Annual Average Sludge Production 27% load reduction goal	
		(cy/day)	(WTONS/Day)	(cy/day)	(WTONS/Day)
Sedimentation	1%	2,300[1]	1,900	1400[1]	1200
Gravity Thickening	3%	700[1]	600	500[1]	400
Mechanical Dewatering	15%	170[2]	120	100[2]	80
Sludge Drying Beds	30%	100[3]	60	70[3]	40

1 Assumes a dewater sludge density of 8.34 lb/gal or 62.4 lbs/ft^3.
2 Assumes a dewater sludge density of 7.0 lb/gal or 52.4 lbs/ft^3.
3 Assumes a dewater sludge density of 6.0 lb/gal or 44.9 lbs/ft^3.

Table 6.3-3 Expected maximum sludge quantities from each step in the sludge handling process in reducing total nitrogen loads by 45% and 27%.

Sludge Handling Process	Assumed Percent Solids	Maximum Daily Sludge Production 45% load reduction goal		Maximum Daily Sludge Production 27% load reduction goal	
		(cy/day)	(WTONS/Day)	(cy/day)	(WTONS/Day)
Sedimentation	1%	10,000[1]	8,400	3,600[1]	3,000
Gravity Thickening	3%	3,300[1]	2,800	1,200[1]	1,000
Mechanical Dewatering	15%	790[2]	560	280[2]	200
Sludge Drying Beds	30%	460[3]	280	160[3]	100

4 Assumes a dewater sludge density of 8.34 lb/gal or 62.4 lbs/ft^3.
5 Assumes a dewater sludge density of 7.0 lb/gal or 52.4 lbs/ft^3.
6 Assumes a dewater sludge density of 6.0 lb/gal or 44.9 lbs/ft^3.

6.3.6 Regulatory Requirements

Because the sludge is a water rather than wastewater generated sludge, disposal of the sludge would most likely be regulated as a water treatment plant residual under the Chapter 62 of the F.A.C. Because the sludge is not a residual from biological treatment of wastewater, there should not be the need for pathogen reduction and restrictions on its land application or disposal to a land fill should be minimal. This would need to be verified, however, with FDEP.

6.3.7 Capital, Operation and Maintenance Costs

Tables 6.3-4 and 6.3-5 list the capital, operating and equipment replacement costs (December 2004 dollars) for the 190-cfs and 68-cfs sedimentation basin treatment facilities, respectively. Details of how these estimates were calculated are provided in Tables 3 and 10 in Appendix F. Capital costs including engineering were estimated at $40.3 million and $22.6 million, respectively. For comparative purposes, this is equivalent to $0.33 and $0.51 per treated gallon which compared to Tampa Bay Water's Regional Water Treatment Plant at a cost of $1.40/gal (2002 dollars) is reasonable given the proposed facility only includes coagulation, flocculation and sedimentation. Tampa Bay Water's Regional Water Treatment Plant also includes ozone disinfection, filtration, disinfection, laboratories, pilot facilities, etc. Significant cost items were the flocculation/sedimentation basins estimated and mechanical sludge dewatering. Annual operating costs were estimated at $2.9 million and $1.7 million per year, respectively. The majority of these costs were associated with operation and maintenance of the flocculation/sedimentations basins, transportation and landfill disposal of sludge and purchase of alum (see Appendix F, Table 6 and 12 for detailed breakdown). Equipment replacement costs incurred every 20-years was estimated at $16.8 and $9.6 million, respectively. Based on a 50-year present worth analysis that accounts for capital, operations and maintenance and equipment replacement costs, the total present worth investment was estimated at $140 million and $81 million, respectively. Based on the level of nitrogen removed, this equates to $9.25 and $8.74 per lb of nitrogen ($/lb TN) removed, respectively.

Table 6.3-4 Capital and Operating Costs for 190-cfs Capacity Sedimentation Basins needed to achieve 45% total nitrogen load reduction goal.

System	Capital Costs	Annual Operating Costs	Equipment Replacement Costs[1]
	($)	($)	($)
Clearing and Grubbing	$182,000	$0	$0
Earthwork	$209,000	$0	$0
Intake and Inflow Pump Station	$2,688,000	$236,000	$1,774,000
Inflow Transmission Main	$639,000	$7,000	$0
Flocculation/Sedimentation Basins	$10,350,000	$541,000	$6,831,000
Discharge Channel	$1,036,000	$11,000	$0
Discharge Channel	$1,935,000	$101,000	$1,278,000
Mechanical Dewatering	$8,990,000	$470,000	$5,934,000
Sludge Drying Beds	$2,654,000	$623,000	$0
Operations & Maintenance Bldg	$3,600,000	$54,000	$0
Alum Metering and Storage	$1,530,000	$860,000	$1,010,000
Pavement	$1,194,000	$12,000	$0
SUBTOTAL	**$35,005,000**	**$2,912,000**	**$16,826,000**
Land Acquisition [2]	$0	$0	$0
Engineering [3]	$5,251,000		
Total	$40,255,000	$2,912,000	$16,826,000
Present Worth Cost [4]	$40,255,000	$84,682,000	$15,146,000
Total Present Worth Cost	$140,082,000		
Per Pound Nitrogen Removed [5]	$9.25		

1 Replacement of equipment and material items every 20 years.
2 Land costs were not included in this present worth analysis. These costs and their influence on the present worth analysis are discussed in further detail in Section 7.0.
3 Estimated as 15% of capital costs.
4 Estimated at an interest rate of 5.625% for a 50-year period. Annual O&M costs were inflated at 3% per year. Salvage of equipment purchased at 40 years estimated at 1/3 the purchase value at the end of 50 years.
5 Listed cost based on estimated per pound nitrogen removed by sedimentation basins over a 50 year period.

Table 6.3-5 Capital and Operating Costs for 68-cfs Capacity Sedimentation Basins needed to achieve 27% total nitrogen load reduction goal.

System	Capital Costs	Annual Operating Costs	Equipment Replacement Costs[1]
	($)	($)	($)
Clearing and Grubbing	$61,000	$0	$0
Earthwork	$70,000	$0	$0
Intake and Inflow Pump Station	$1,501,000	$139,000	$991,000
Inflow Transmission Main	$432,000	$5,000	$0
Flocculation/Sedimentation Basins	$5,500,000	$288,000	$3,630,000
Discharge Channel	$735,000	$8,000	$0
Gravity Thickening	$1,200,000	$63,000	$792,000
Mechanical Dewatering	$5,500,000	$288,000	$3,630,000
Sludge Drying Beds	$1,234,000	$385,000	$0
Operations & Maintenance Bldg	$1,800,000	$27,000	$0
Alum Metering and Storage	$790,000	$514,000	$522,000
Pavement	$842,000	$9,000	$0
SUBTOTAL	**$19,662,000**	**$1,720,000**	**$9,564,000**
Land Acquisition [2]	$0	$0	$0
Engineering [3]	$2,950,000		
Total	$22,611,000	$1,720,000	$9,564,000
Present Worth Cost [4]	$22,611,000	$50,024,000	$8,610,000
Total Present Worth Cost	$81,243,000		
Per Pound Nitrogen Removed [5]	$8.74		

1 Replacement of equipment and material items every 20 years.
2 Land costs were not included in this present worth analysis. These costs and their influence on the present worth analysis are discussed in further detail in Section 7.0.
3 Estimated as 15% of capital costs.
4 Estimated at an interest rate of 5.625% for a 50-year period. Annual O&M costs were inflated at 3% per year. Salvage of equipment purchased at 40 years estimated at 1/3 the purchase value at the end of 50 years.
5 Listed cost based on estimated per pound nitrogen removed by sedimentation basins over a 50 year period.

6.4 SEDIMENTATION BASINS FOLLOWED BY FILTRATION

6.4.1 General Description

Filtration is a physical and, in some instances, a biological process for removing suspended particles, organic and inorganic matter from water by passing the water through a porous or granular media. Because some suspended particles and/or small floc (referred to as pin floc) may discharge from sedimentation, adding filtration can enhance the removal of these particles and thereby enhance the removal of particulate nitrogen. Filtration can also improve the clarity of the water by reducing turbidity.

Filters can be classified in various ways. They can be classified according to (1) the type of filter media, (2) the type of filter rate-control, (3) the direction of flow through the bed, or (4) whether they operate under gravity or pressure. The most commonly used filter for treatment of surface water is the rapid sand filter. A rapid sand filter consists of a vessel whereby water is passed through layers of graded sand or other layers of media placed on top of the sand such as anthracite or granular activated carbon (GAC) to remove suspended particles and organics. GAC can also be used to support a substrate that can be used to biologically remove organic and inorganic contaminants, and is also referred to as a biological active filter. As an aside, Tampa Bay Water's Regional Surface Water Treatment Plant employs biologically active filters to reduce natural organic matter. Typical filtration rates may vary depending on raw water quality but usually range between 2-5 gallons per minute per square foot (gpm/ft^2). This is in contrast with slow sand filters where filtration rates are much lower ranging between 0.04 to 0.2 gpm/ft^2, requiring more land be used to accommodate slow sand filters (Schulz and Okun, 1984).

Rapid sand filters require backwashing and possibly auxiliary scour wash systems to remove embedded particles from the filter media. The need for backwashing is governed by head loss and/or breakthrough of particles as typically determined by measuring turbidity. Backwashing requires a source of filtered water and a surge basin to collect the backwash water waste for further processing and recycle to the headworks of the treatment plant.

6.4.2 Conceptual Design

Based on jar test results as discussed in Section 4.0, a 7.5 mg/L (as Al^-) alum dose resulted in total nitrogen reductions that varied from 26 to 71 percent after 5-minutes of settling followed by filtering through a 0.45 μm filter and 21 to 81 percent after 24 hours of settling followed by filtering through a 0.45 μm filter. Filtering the sample through a 0.45 μm filter was used to simulate filtration. Comparing this with sedimentation alone, the same dose resulted in reductions that varied from 30 to 69 percent after 3-hours of settling and 25 to 78 after 24 hours of settling. Given this wide variation in results it is difficult to identify an improvement in water quality as a result of filtering the clarified water from sedimentation. If there is an improvement it is only marginal based on the collected data. Further discussions of the impact filtration has on water quality is discussed in Section 6.4.4.

Figure 6.4-1 shows the proposed process diagram for sedimentation followed by filtration. Figures 6.4-2 and 6.4-3 show the proposed site layouts for the 190-cfs and 68-cfs facilities, respectively. Eight gravity feed filter cells, each rated at 25-cfs (16.25-mgd) based on a 4-gpm/ft^2 loading rate were included in the 190-cfs facility. Four filter cells, each rated at 18-cfs (11.63-mgd) were included in the 68-cfs facility. Multiple filter cells are required in both cases based on the length of laterals that can be used in filter underdrains. The clarified water from the sedimentation basins would feed to a common header feeding the eight filter cells. The filtered water would discharge through the bottom of the filter cells into a clearwell prior to discharging into the discharge channel. Submersible pumps located in the clearwell would be used for backwashing the filters. The backwash water would enter through the bottom of the filters expanding the bed and dislodging the accumulated floc and particles in the backwash water waste. An additional surface wash would also need to be provided to clean the surface of accumulated floc and to minimize the buildup and growth of "mud balls" on the surface of the media. The backwash water and surface wash water waste would then discharge to the surge basin where any accumulated particles would be allowed to settle prior to the water being recycled to the headworks of the facility.

6.4.3 Operation and Maintenance

Operations and maintenance of a facility that includes filters would be more extensive than the sedimentation basin facility described in Section 6.3. Depending on the amount of flow requiring treatment, filter cells can be brought in or out of service as needed. Backwashing of the filters would be automated requiring the use of motorized actuated valves for feed water, filtered water, backwash water, and backwash waste water for each cell. Backwashing would be initiated based on either the buildup of head as a result of the filter clogging and/or filtered water turbidity. Most of the time maintenance would be limited to maintaining the motorized actuated valves and monitoring instrumentation. Every 20 years, the filter media may need to be replaced. Staffing requirements should not be any more than that needed for the sedimentation basin facility described in Section 6.3.3.

6.4.4 Expected Finished Water Quality

Table 6.4-1 lists the average raw water and average treated water quality from jar testing at an alum dose of 7.5 mg/L (as Al-) following filtration through a 0.45 µm filter. Because samples were not filtered after 3-hours of settling which would resemble more closely the concept design of the facility, the results collected after 5-minutes and 24-hours of settling were averaged. For comparison, the expected percent reduction from sedimentation followed by filtration were listed in the far right column.

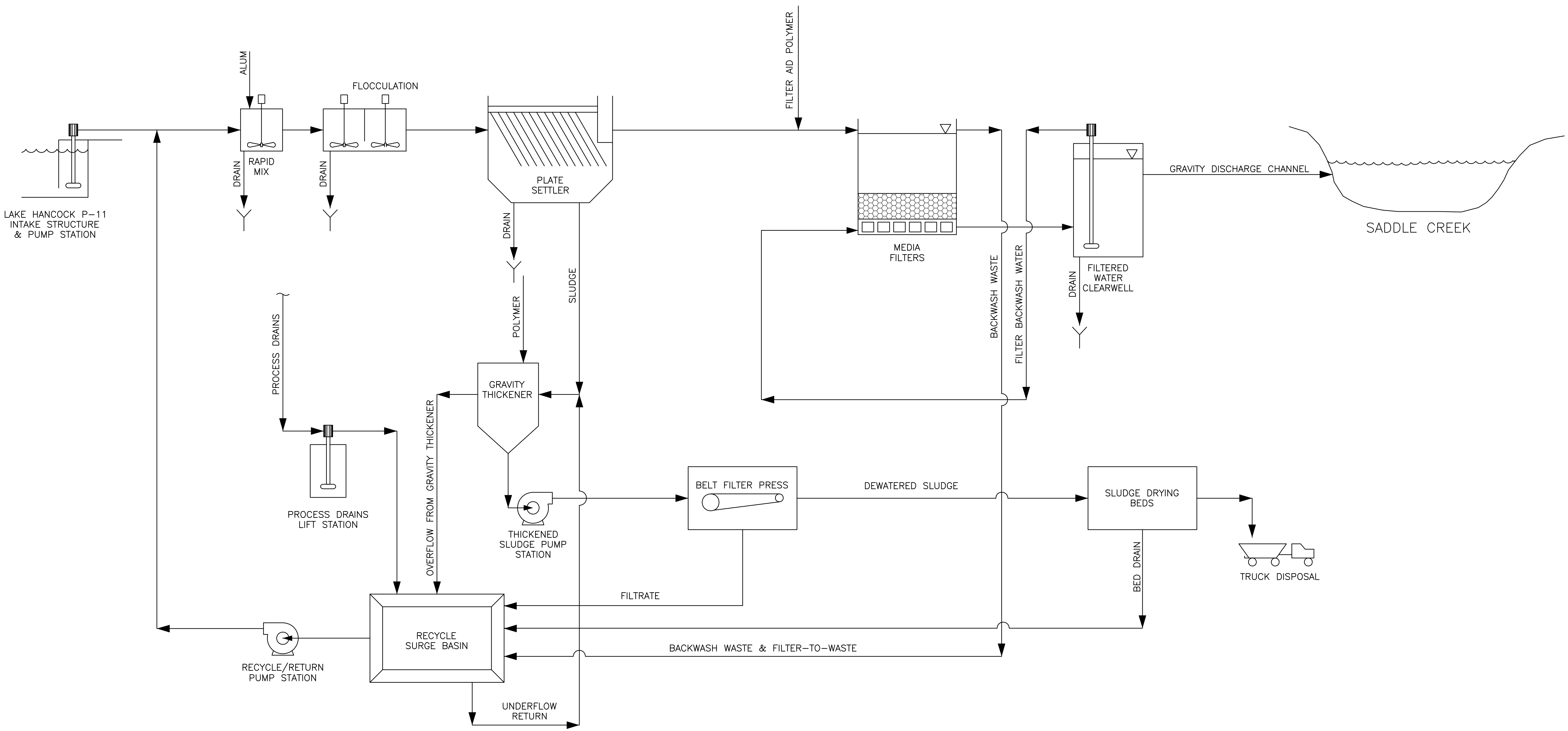
With filtration, nutrient levels and the clarity of the water should be slightly improved over that provided by sedimentation alone. Total nitrogen and turbidity are expected to be reduced by 60 to 70 and 95 to 99 percent, respectively, which are 10 and 5 percent reductions over sedimentation alone.

6.4.5 Residuals Disposal

Residuals disposal should be the same as described in Section 6.3.5.

6.4.6 Regulatory Requirements

Regulatory requirements should be the same as described in Section 6.3.6.



NOT TO SCALE

FIGURE NO. 6.4-1

SEDIMENTATION BASIN
AND FILTRATION TREATMENT FACILITY
PROCESS FLOW DIAGRAM

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FIGURE NO. 6.4-2

190 CFS SEDIMENTATION BASIN AND FILTRATION TREATMENT FACILITY SITE PLAN

LAKE HANCOCK OUTFALL TREATMENT PROJECT
ALTERNATIVE TREATMENT TECHNOLOGIES
SOUTHWEST FLORIDA WATER MANAGEMENT DISTRICT

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FIGURE NO. 6.4-3

68 CFS SEDIMENTATION BASIN
AND FILTRATION TREATMENT FACILITY
SITE PLAN

LAKE HANCOCK OUTFALL TREATMENT PROJECT
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Table 6.4-1 Expected Treatment Performance for Alum Coagulation Followed by 3-Hours of Sedimentation and Filtration at a 7.5 mg/L (as Al⁻) Alum Dose

Parameter	Units	Lake Hancock Water Quality[1]	Treated Water Quality[2]	Expected Percent Reduction[3]
pH	s.u.	7.61	6.54	10 to 20
Conductivity	µmho/cm	164	188	(-10 to -20)[4]
Ammonia	µg/l	409	356	0
Nitrate and Nitrite	µg/l	728	286	0
Organic Nitrogen	µg/l	2883	1430	0
Total Nitrogen	µg/l	3779	2024	-[5]
Soluble Reactive Phosphorus (SRP)	µg/l	263	1.33	90 to95
Total Phosphorus	µg/l	588	9.17	90 to 95
Turbidity	NTU	24.7	0.38	95 to 99

1. Average of jar test results collected on September 17, 28 and October 25, 2004.
2. Based on averaging jar test results filtered after 5-minutes and 24-hours of settling.
3. Based on jar test results discussed in Section 4.0. Ranges were provided rather than specific values based on expected variability in water quality and performance.
4. Negative values indicate an increase in concentration as a result of adding alum.
5. Chemical coagulation followed by sedimentation and filtration will only remove particulate nitrogen and thus the effectiveness to reduce total nitrogen is governed by the amount of particulate nitrogen present. Based on jar test results, total nitrogen should be reduced by approximately 60%.

6.4.7 Capital, Operations and Maintenance Costs

Tables 6.4-2 and 6.4-3 list the capital, operating and equipment replacement costs (December 2004 dollars) for the 190-cfs and 68-cfs sedimentation basin followed by filtration facilities, respectively. Details of how these costs were estimated are provided in Table 4 and 11 in Appendix F. Capital costs including engineering were estimated at $77.6 million and $38.4 million, respectively. This is equivalent to $0.63/gallon and $0.87/gallon which is reasonable given the processes that are included. Filtration alone was estimated to cost $32.4 million and $13.7 million, respectively. Annual operating costs were estimated at $4.6 million and $2.4 million per year. Equipment replacement costs incurred every 20-years was estimated at $38.2 million and $18.6 million, respectively. Based on a 50-year present worth analysis that accounts for capital, operations and maintenance and equipment replacement costs, the total present worth investment was estimated at $245 million and $126 million or $16.23/lb and $13.55/lb nitrogen removed, respectively.

Table 6.4-2 Capital and Operating Costs for 190-cfs Capacity Sedimentation Basins followed by Filtration needed to achieve 45% total nitrogen load reduction goal.

System	Capital Costs	Annual Operating Costs	Equipment Replacement Costs[1]
	($)	($)	($)
Clearing and Grubbing	$182,000	$0	$0
Earthwork	$279,000	$0	$0
Intake and Inflow Pump Station	$2,688,000	$236,000	$1,774,000
Inflow Transmission Main	$639,000	$7,000	$0
Sedimentation Basins	$10,350,000	$541,000	$6,831,000
Filtration	$32,400,000	$1,691,000	$21,384,000
Discharge Channel	$1,036,000	$11,000	$0
Thickening	$1,935,000	$101,000	$1,278,000
Mechanical Dewatering	$8,990,000	$470,000	$5,934,000
Sludge Drying Beds	$2,654,000	$623,000	$0
Operations & Maintenance Bldg	$3,600,000	$54,000	$0
Alum Metering and Storage	$1,530,000	$860,000	$1,010,000
Pavement	$1,194,000	$12,000	$0
SUBTOTAL	**$67,474,000**	**$4,602,000**	**$38,210,000**
Land Acquisition [2]	$0	$0	$0
Engineering [3]	$10,122,000		
Total	$77,595,000	$4,602,000	$38,210,000
Present Worth Cost [4]	$77,595,000	$133,864,000	$34,396,000
Total Present Worth Cost	$245,855,000		
Per Pound Nitrogen Removed [5]	$16.23		

1 Replacement of equipment and material items every 20 years.
2 Land costs were not included in this present worth analysis. These costs and their influence on the present worth analysis are discussed in further detail in Section 7.0.
3 Estimated as 15% of capital costs.
4 Estimated at an interest rate of 5.625% for a 50-year period. Annual O&M costs were inflated at 3% per year. Salvage of equipment purchased at 40 years estimated at 1/3 the purchase value at the end of 50 years.
5 Listed cost based on estimated per pound nitrogen removed by sedimentation basins followed by filtration over a 50 year operating period.

Table 6.4-3 Capital and Operating Costs for 68-cfs Capacity Sedimentation Basins followed by Filtration needed to achieve 27% total nitrogen load reduction goal..

System	Capital Costs	Annual Operating Costs	Equipment Replacement Costs[1]
	($)	($)	($)
Clearing and Grubbing	$91,000	$0	$0
Earthwork	$105,000	$0	$0
Intake and Inflow Pump Station	$1,501,000	$139,000	$991,000
Inflow Transmission Main	$432,000	$5,000	$0
Sedimentation Basins	$5,500,000	$288,000	$3,630,000
Filtration	$13,700,000	$715,000	$9,042,000
Discharge Channel	$735,000	$8,000	$0
Gravity Thickening	$1,200,000	$63,000	$792,000
Mechanical Dewatering	$5,500,000	$288,000	$3,630,000
Sludge Drying Beds	$1,234,000	$385,000	$0
Operations & Maintenance Bldg	$1,800,000	$27,000	$0
Alum Metering and Storage	$790,000	$514,000	$522,000
Pavement	$842,000	$9,000	$0
SUBTOTAL	**$33,427,000**	**$2,435,000**	**$18,606,000**
Land Acquisition [2]	$0	$0	$0
Engineering [3]	$5,014,000		
Total	$38,441,000	$2,435,000	$18,606,000
Present Worth Cost [4]	$38,441,000	$70,820,000	$16,749,000
Total Present Worth Cost	$126,009,000		
Per Pound Nitrogen Removed [5]	$13.55		

1 Replacement of equipment and material items every 20 years.
2 Land costs were not included in this present worth analysis. These costs and their influence on the present worth analysis are discussed in further detail in Section 7.0.
3 Estimated as 15% of capital costs.
4 Estimated at an interest rate of 5.625% for a 50-year period. Annual O&M costs were inflated at 3% per year. Salvage of equipment purchased at 40 years estimated at 1/3 the purchase value at the end of 50 years.
5 Listed cost based on estimated per pound nitrogen removed by sedimentation basins followed by filtration over a 50 year operating period.

6.5 DISSOLVED AIR FLOTATION

6.5.1 General Description

Like sedimentation, DAF is used in removing flocculated particles. DAF, however, relies on flotation of the flocculated particles rather than gravity settling. Infilco Degremont, supplier of the AquaDAF™ High-Rate Clarifier, provided a proposal for this project, which is included in Appendix G. Their proposal provides a detailed process description and photographs of installed systems. In general, DAF occurs in a flotation tank where dissolved air under pressure (typically 70 psig) is injected into a recycle stream that is fed through a series of nozzles installed in the bottom of the flotation tank. Due to the pressure differential created by these nozzles, small bubbles in the range of 10 to 100-µm are released. As these bubbles migrate to the top of the tank, they collect and concentrate the flocculated particles forming a mat on the surface. This mat is continuously scoured into a collection trough where it is typically transported to a thickener for further thickening and dewatering.

6.5.2 Conceptual Design

The capacity of the DAF was designed based on the same jar test results used for sizing the sedimentation basins. Although the mechanisms for removal are different, the performance of a DAF system can only be tested with pilot testing. Based on a 60 percent reduction of total nitrogen, the maximum capacity of the facility was designed at 190-cfs (123-mgd).

Figures 6.5-1 shows the process flow diagram and Figures 6.5-2 and 6.5-3 show the layout for the 190-cfs and 68-cfs DAF facilities, respectively. The facility consists of the following major processes:

- Inflow intake and pump station
- Inflow transmission main
- Dissolved Air Flotation (includes mechanical rapid mix and flocculation)
- Discharge channel and outfall structure
- Alum storage and metering facilities
- Gravity thickeners
- Mechanical Dewatering
- Sludge drying beds
- Surge basin

Figure 6.5-4 shows the process flow diagram and Figures 6.5-5 and 6.5-6 show the layout for a 190-cfs and 68-cfs DAF facility that includes filtration, respectively. As with sedimentation, filters may be required to prevent floc carryover from discharging into Saddle Creek. In either case, DAF with or without filtration are similar to the designs previously described for sedimentation basins in Section 6.3 and 6.4.

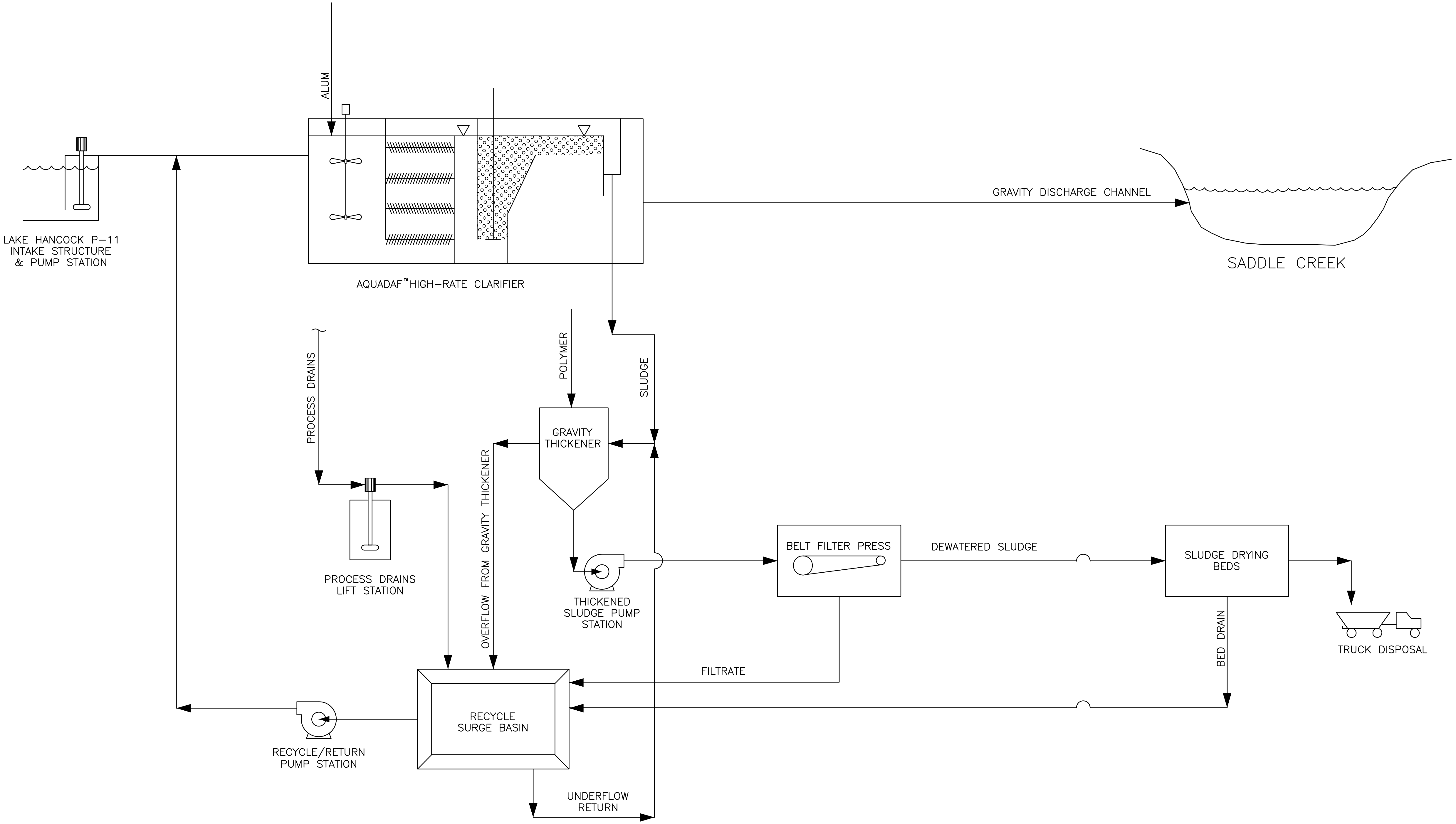

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FIGURE NO. 6.5-1

AQUADAF HIGH-RATE CLARIFICATION
TREATMENT FACILITY
PROCESS FLOW DIAGRAM

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FIGURE NO. 6.5-2

190 CFS AQUADAF HIGH-RATE CLARIFICATION TREATMENT FACILITY SITE PLAN

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FIGURE NO. 6.5-3

68 CFS AQUADAF HIGH-RATE CLARIFICATION TREATMENT FACILITY SITE PLAN

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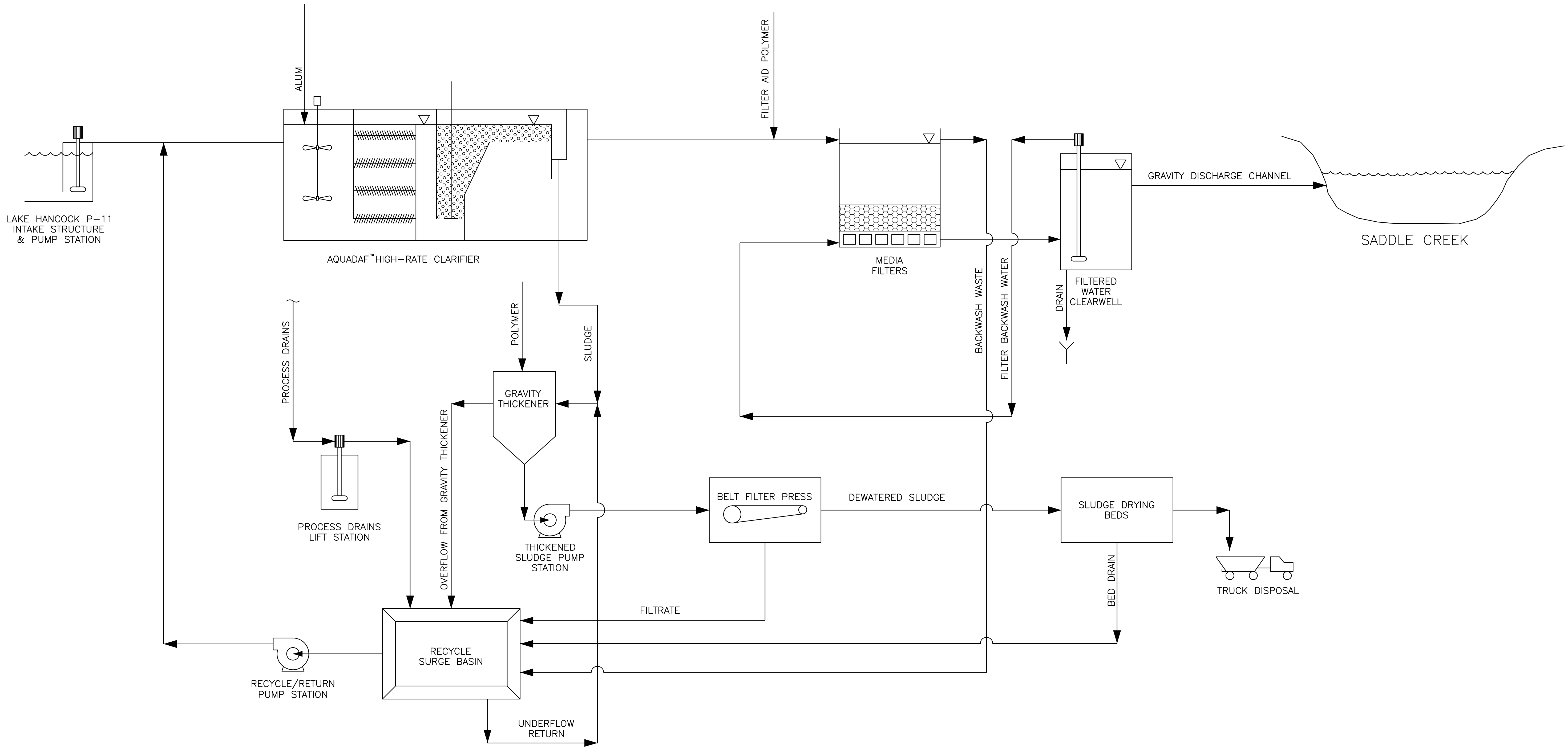

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FIGURE NO. 6.5-4

AQUADAF HIGH-RATE CLARIFICATION
AND FILTRATION TREATMENT FACILITY
PROCESS FLOW DIAGRAM

LAKE HANCOCK OUTFALL TREATMENT PROJECT
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FIGURE NO. 6.5-5

190 CFS AQUADAF HIGH-RATE CLARIFICATION
AND FILTRATION TREATMENT FACILITY
SITE PLAN

LAKE HANCOCK OUTFALL TREATMENT PROJECT
ALTERNATIVE TREATMENT TECHNOLOGIES
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J:\CADD\743785\Figures\7385FIG6-3-2to6.5-4 rev1.dwg Sep 08, 2005 - 4:06pm, LAST ACCESSED BY: p0011258

FIGURE NO. 6.5-6

68 CFS AQUADAF HIGH-RATE CLARIFICATION AND FILTRATION TREATMENT FACILITY SITE PLAN

LAKE HANCOCK OUTFALL TREATMENT PROJECT
ALTERNATIVE TREATMENT TECHNOLOGIES
SOUTHWEST FLORIDA WATER MANAGEMENT DISTRICT

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XREFS: contours-topo
J:\CADD\743785\Figures\7385FIG6-3-2to6.5-4 rev1.dwg Sep 08, 2005 - 4:09pm, LAST ACCESSED BY: p0011258

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In their proposal for the 190-cfs DAF, Infilco Degremont proposes a design that includes eight 23-cfs (15.0-mgd) units and two 8-cfs (5-mgd) units to accommodate the variability in flows. It is proposed for the 68-cfs DAF, four 23-cfs (15.0-mgd) units be used. Incorporated in the design of the AquaDAF™ High Rate Clarifier is a mechanical rapid mix and a static flocculator which precede the DAF. The flocculated water enters the DAF unit from below where a fine diffuser adds air to the floc particles floating them to the water surface as the water is forced to flow upward over a weir (see process diagram on page 2 of the Infilco Degremont Proposal provided in Appendix G). The accumulated floc forms a mat on the surface that towards the other end of the DAF unit where it falls over a weir into a sludge collection channel and is discharged into a common header that feeds the gravity thickeners. The clarified water from the DAF discharges from below the floating mat into a common chamber along with the clarified water from the other DAF units. From there, the water discharges either directly to the discharge channel in the case where no filters are provided (Figures 6.5-1 thru 6.5-3) or to a common header pipe that feeds the filters (Figures 6.5-4 thru 6.5-6).

6.5.3 Operation and Maintenance

Operations and maintenance of a DAF will be different than a sedimentation basin. The DAF requires compressor air to operate and in total eight compressors would be included (6 duty and 2 standby) for the 190-cfs design and six (4-duty and 2 standby) for the 68-cfs deign. Because the sludge is withdrawn from the top, submerged chain and flight scrapers are not needed which is a common maintenance issue for sedimentation basins. Staffing requirements should not be any more than that needed for the sedimentation basin facility described in Section 6.3.3.

6.5.4 Expected Finished Water Quality

Table 6.3-1 in Section 6.3.4 lists the expected performance from a DAF without filtration and Table 6.4-1 in Section 6.4.4 lists the expected performance with filtration.

6.5.5 Residuals Disposal

Residuals disposal should be the same as described in Section 6.3.5.

6.5.6 Regulatory Requirements

Regulatory requirements should be the same as described in Section 6.3.6.

6.5.7 Capital, Operation and Maintenance Costs

Tables 6.5-1 and 6.5-2 list the capital, operating and equipment replacement costs (December 2004 dollars) for a 190-cfs and 68-cfs DAF without filtration, respectively. Tables 6.5-3 and 6.5-4 list the capital, operating and equipment replacement costs (December 2004 dollars) for a 190-cfs and 68-cfs DAF that includes filtration, respectively. Details of how these costs were estimated are provided in Tables 5, 6, 12 and 13 in Appendix F.

DAF without Filtration. Capital costs including engineering and contingency were estimated for the 190-cfs and 68-cfs facilities at $39.6 million and $22.3 million, respectively. This is equivalent to $0.32/gal and $0.51/gal, respectively, which is less than sedimentation basins (i.e., $0.35/gal and $0.44/gal). The DAF alone was estimated to cost $9.8 million and $4.2 million. Annual operating costs were estimated at $2.9 million and $1.7 million per year, respectively. Equipment replacement costs incurred every 20-years was estimated at $16.5 million and $9.4 million, respectively. Based on a 50-year present worth analysis that accounts for capital, operations and maintenance and equipment replacement costs, the total present worth investment was estimated at $138 million and $80.3 million or $9.13/lb and $8.64/lb nitrogen removed, respectively.

Table 6.5-1 Capital and Operating Costs for 190-cfs Capacity AquaDAF High Rate Clarification needed to achieve 45% total nitrogen load reduction goal.

System	Capital Costs	Annual Operating Costs	Equipment Replacement Costs[1]
	($)	($)	($)
Clearing and Grubbing	$182,000	$0	$0
Earthwork	$209,000	$0	$0
Intake and Inflow Pump Station	$2,688,000	$236,000	$1,774,000
Inflow Transmission Main	$639,000	$7,000	$0
AquaDAF High Rate Clarification	$9,825,000	$513,000	$6,485,000
Discharge Channel	$1,036,000	$11,000	$0
Gravity Thickening	$1,935,000	$101,000	$1,278,000
Mechanical Dewatering	$8,990,000	$470,000	$5,934,000
Sludge Drying Beds	$2,654,000	$623,000	$0
Operations & Maintenance Bldg	$3,600,000	$54,000	$0
Alum Metering and Storage	$1,530,000	$860,000	$1,010,000
Pavement	$1,194,000	$12,000	$0
SUBTOTAL	**$34,479,000**	**$2,879,000**	**$16,479,000**
Land Acquisition [2]	$0	$0	$0
Engineering [3]	$5,172,000		
Total	$39,651,000	$2,884,000	$16,479,000
Present Worth Cost [4]	$39,651,000	$83,884,000	$14,834,000
Total Present Worth Cost	$138,369,000		
Per Pound Nitrogen Removed[5]	$9.13		

1 Replacement of equipment and material items every 20 years.
2 Land costs were not included in this present worth analysis. These costs and their influence on the present worth analysis are discussed in further detail in Section 7.0.
3 Estimated as 15% of capital costs.
4 Estimated at an interest rate of 5.625% for a 50-year period. Annual O&M costs were inflated at 3% per year. Salvage of equipment purchased at 40 years estimated at 1/3 the purchase value at the end of 50 years.
5 Listed cost based on estimated per pound nitrogen removed by dissolved air flotation over a 50 year operating period.

Table 6.5-2 Capital and Operating Costs for 68-cfs capacity AquaDAF High Rate Clarification needed to achieve 27% total nitrogen load reduction goal.

System	Capital Costs	Annual Operating Costs	Equipment Replacement Costs[1]
	($)	($)	($)
Clearing and Grubbing	$61,000	$0	$0
Earthwork	$70,000	$0	$0
Intake and Inflow Pump Station	$1,501,000	$139,000	$991,000
Inflow Transmission Main	$432,000	$5,000	$0
AquaDAF High Rate Clarification	$4,231,000	$221,000	$2,793,000
Discharge Channel	$735,000	$8,000	$0
Gravity Thickening	$1,200,000	$63,000	$792,000
Mechanical Dewatering	$6,500,000	$340,000	$4,290,000
Sludge Drying Beds	$1,234,000	$385,000	$0
Operations & Maintenance Bldg	$1,800,000	$27,000	$0
Alum Metering and Storage	$790,000	$514,000	$522,000
Pavement	$842,000	$9,000	$0
SUBTOTAL	**$19,392,000**	**$1,706,000**	**$9,386,000**
Land Acquisition [2]	$0		
Engineering [3]	$2,909,000		
Total	$22,301,000	$1,706000	$9,386,000
Present Worth Cost [4]	$22,301,000	$49,6150	$8,449,000
Total Present Worth Cost	$80,364,000		
Per Pound Nitrogen Removed [5]	$8.64		

1 Replacement of equipment and material items every 20 years.
2 Land costs were not included in this present worth analysis. These costs and their influence on the present worth analysis are discussed in further detail in Section 7.0.
3 Estimated as 15% of capital costs.
4 Estimated at an interest rate of 5.625% for a 50-year period. Annual O&M costs were inflated at 3% per year. Salvage of equipment purchased at 40 years estimated at 1/3 the purchase value at the end of 50 years.
5 Listed cost based on estimated per pound nitrogen removed by dissolved air flotation over a 50 year operating period.

DAF with Filtration. Capital costs including engineering and contingency were estimated for the 190-cfs and 68-cfs facilities at $77.0 million and $37.0 million, respectively, which is equivalent to $0.62/gal and $0.84/gal, respectively. Filtration alone was estimated to cost $33.4 and $13.7 million. Annual operating costs were estimated at $4.6 million and $2.4 million per year. Equipment replacement costs incurred every 20-years was estimated at $37.8 million and $17.8 million (2004 dollars). Based on a 50-year present worth analysis that accounts for capital, operations and maintenance and equipment replacement costs, the total present worth investment was estimated at $244 million and $122 million or $16.12/lb and $13.11/lb nitrogen removed.

Table 6.5-3 Capital and operating costs for 190-cfs capacity AquaDAF High Rate Clarification followed by filtration needed to achieve 45% total nitrogen load reduction goal.

System	Capital Costs	Annual Operating Costs	Equipment Replacement Costs[1]
	($)	($)	($)
Clearing and Grubbing	$182,000	$0	$0
Earthwork	$279,000	$0	$0
Intake and Inflow Pump Station	$2,688,000	$236,000	$1,774,000
Inflow Transmission Main	$639,000	$7,000	$0
AquaDAF High Rate Clarification	$9,825,000	$513,000	$6,485,000
Filtration	$33,400,000	$1,691,000	$21,384,000
Discharge Channel	$1,036,000	$11,000	$0
Gravity Thickening	$1,935,000	$101,000	$1,278,000
Mechanical Dewatering	$8,990,000	$470,000	$5,934,000
Sludge Drying Beds	$2,654,000	$623,000	$0
Operations & Maintenance Bldg	$3,600,000	$54,000	$0
Alum Metering and Storage	$1,530,000	$860,000	$1,010,000
Pavement	$1,194,000	$12,000	$0
SUBTOTAL	**$66,949,000**	**$4,575,000**	**$37,863,000**
Land Acquisition [2]	$0	$0	$0
Engineering [3]	$10,043,000		
Total	$76,991,000	$4,575,000	$37,863,000
Present Worth Cost [4]	$76,991,000	$133,067,000	$34,084,000
Total Present Worth Cost	$244,141,000		
Per Pound Nitrogen Removed[5]	$16.12		

1 Replacement of equipment and material items every 20 years.
2 Land costs were not included in this present worth analysis. These costs and their influence on the present worth analysis are discussed in further detail in Section 7.0.
3 Estimated as 15% of capital costs.
4 Estimated at an interest rate of 5.625% for a 50-year period. Annual O&M costs were inflated at 3% per year. Salvage of equipment purchased at 40 years estimated at 1/3 the purchase value at the end of 50 years.
5 Listed cost based on estimated per pound nitrogen removed by dissolved air flotation followed by filtration over a 50 year operating period.

Table 6.5-4 Capital and Operating Costs for 68-cfs Capacity AquaDAF High Rate Clarification Followed by Filtration needed to achieve 27% total nitrogen load reduction goal.

System	Capital Costs	Annual Operating Costs	Equipment Replacement Costs[1]
	($)	($)	($)
Clearing and Grubbing	$91,000	$0	$0
Earthwork	$105,000	$0	$0
Intake and Inflow Pump Station	$1,501,000	$139,000	$991,000
Inflow Transmission Main	$432,000	$5,000	$0
AquaDAF High Rate Clarification	$4,231,000	$221,000	$2,793,000
Filtration	$13,700,000	$715,000	$9,042,000
Discharge Channel	$735,000	$8,000	$0
Gravity Thickening	$1,200,000	$63,000	$792,000
Mechanical Dewatering	$5,500,000	$288,000	$3,630,000
Sludge Drying Beds	$1,234,000	$385,000	$0
Operations & Maintenance Bldg	$1,800,000	$27,000	$0
Alum Metering and Storage	$790,000	$514,000	$522,000
Pavement	$842,000	$9,000	$0
SUBTOTAL	**$32,157,000**	**$2,369,000**	**$17,768,000**
Land Acquisition[2]	$0		
Engineering [3]	$4,824,000		
Total	$36,981,000	$2,369,000	$17,768,000
Present Worth Cost[4]	$36,981,000	$68,893,000	$15,995,000
Total Present Worth Cost	$121,867,000		
Per Pound Nitrogen Removed[5]	$13.11		

1 Replacement of equipment and material items every 20 years.
2 Land costs were not included in this present worth analysis. These costs and their influence on the present worth analysis are discussed in further detail in Section 7.0.
3 Estimated as 15% of capital costs.
4 Estimated at an interest rate of 5.625% for a 50-year period. Annual O&M costs were inflated at 3% per year. Salvage of equipment purchased at 40 years estimated at 1/3 the purchase value at the end of 50 years.
5 Listed cost based on estimated per pound nitrogen removed by dissolved air flotation followed by filtration over a 50 year operating period.

6.6 MICROSCREEN FILTRATION

6.6.1 General Description

Microscreens are different than sand filters in that woven fabric is used as the filtering media. Kruger, Inc, supplier of the Hydrotech Discfilter, provided a proposal for this project which is included in Appendix H. Their proposal along with brochure provides a detailed process description and photographs of installed systems. In general, the fabric is stretched and attached across a rigid frame. These frames are placed on the side of a rotating disc. Filtered water passes through the screens where the screened particulate matter is left on the fabric. The discs which are partially submerged rotate out of the water where the collected solids are rinsed off using jets into a sludge trough while the cleaned screens are rotated simultaneously back into water. Microscreens provide the advantage of filtering the water without the need for chemical coagulation. However, the use of microscreens is limited based on feed water quality and removal is limited typically to particle sizes greater than 10-um.

6.6.2 Conceptual Design

As part of monitoring the water quality of Lake Hancock, a particle size analysis was performed based on total nitrogen, total phosphorus, total suspended solids (TSS) and volatile suspended solids (VSS) as discussed in Section 2.0. Table 6.6-1 summarizes the results for total nitrogen for samples collected on April 23 and July 26, 2004. As indicated, approximately 90 percent of the total nitrogen is associated with particles that are less than 11 µm in size. Because the smallest size microscreen is 10 µm in size, about 90 percent of the total nitrogen would pass through. Only 10 percent would be retained which is far less than the goal of a 45 percent. However, Kruger is currently experimenting with the use of coagulants using a pilot system on a wastewater discharge into Lake Okeechobee (Telecommunication with Robert Bierhorst, MTS Environmental, January 21, 2005) and has reported some success in rinsing the coagulated particles from the screens which are normally considered sticky and hard to remove. It is possible, in the case of Lake Hancock, that a small dose of a coagulant could produce a pin floc that could be captured by the microscreen. However extensive pilot testing would be required to define operating conditions and the design criteria for a full-scale system.

Table 6.6-1 Lake Hancock Particulate Nitrogen Size Distribution

Particle Size	April 23, 2004		July 26, 2004	
	Total Nitrogen (µg/L)	Percent Finer (%)	Total Nitrogen (µg/L)	Percent Finer (%)
>180 um	45	0.00	107	0.00
140 um	9	99.26	20	99.09
100 um	19	99.85	70	99.83
60 um	177	99.69	91	99.40
30 um	188	97.07	97	99.21
11 um	686	96.79	808	99.15
< 11 um	4,991	**87.92**	10,509	**92.86**
Total	6,115		11,702	

For cost estimating purposes, a 190-cfs and 68-cfs disc filter microscreen facilities were developed based on vendor provided information. Similar in design to the other alternatives discussed previously, a raw water pump station would withdrawal water upstream of P-11 feeding the microscreens directly without sedimentation. Alum would still be required to create a pin floc that could be captured by the disc filter; however, the dose is expected to less than a third of the dose used for sedimentation and DAF. It is possible the dose could be even less, however pilot tests would be required to determine the actual dose. Alum would be added in the transmission main utilizing an inline mixer. The coagulated water would be filtered through the disc filters and the resultant filtrate would be discharged back to the Saddle Creek. The collected residual on the disc filters would be rinsed off and collected in the gravity thickener for thickening prior to mechanical dewatering. Belt filter presses would be used to dewater the thickened sludge prior to being pumped to drying beds for further dewatering.

It is important to note that alum has never been added prior to disc filters and their operating performance has not been documented under full-scale conditions. Extensive pilot testing would be needed if disc filters were selected as a viable alternative for further evaluation.

6.6.3 Operation and Maintenance

Operations and maintenance of the disc filters will be different than the previous technologies described. Residuals that are captured by the discs are jetted off periodically using filtered water and collected and concentrated in gravity thickeners. The filter fabric is stretched across plates that are mounted in a triangular fashion resembling pie slices onto the rotating discs. Overtime the filtering fabric may become irreversible fouled requiring the plates be removed and replaced. In addition, the discs rotate in and out of the raw water as part of the cleaning and rinsing process. The motors and drives that operate this system will need to be periodically checked and maintained to ensure proper operation. Staffing requirements should not be any more than that needed for the sedimentation basin facility described in Section 6.3.3.

6.6.4 Expected Finished Water Quality

The expected finished water quality is unknown at this time. However, it is expected if alum or another coagulant could be used to effectively produce a pin floc that could be captured by the disc filters, that performance would be similar to the performance of the sedimentation ponds (discussed in Section 6.2) and basins (discussed in Section 6.3).

6.6.5 Residuals Disposal

Because it is anticipated that less alum would be needed to produce the pin floc, it was estimated that one third of the alum dose would be needed. As a result, the aluminum hydroxide particulate matter that is chemically formed during this process would be one third less as compared to other alternatives. This would result in the production of less alum sludge which would reduce the amount sludge processed and disposed. The costs of this benefit are reflected in the summaries provided and discussed in Section 6.6.7.

6.6.6 Regulatory Requirements

Regulatory requirements should be the same as described in Section 6.3.6.

6.6.7 Capital, Operation and Maintenance Costs

Tables 6.6-2 and 6.6-3 list the capital, operating and equipment replacement costs (December 2004 dollars) for a 190-cfs and 68-cfs disc filters, respectively. Details of how these costs were estimated are provided in Tables 7 and 14 in Appendix F. Capital costs including engineering and contingency were estimated for the 190-cfs and 68-cfs facilities at $40.9 million and $23.1 million, respectively. This is equivalent to $0.33/gal and $0.52/gal, respectively, which is approximately equivalent to sedimentation basins (i.e., $0.33 and $0.51/gal). Disc filters alone was estimated to cost $12.1 million and $5.9 million, respectively. Annual operating costs were estimated at $2.5 million and $1.5 million per year, respectively. Equipment replacement costs incurred every 20-years was estimated at $17.8 million and $9.8 million, respectively. Based on a 50-year present worth analysis that accounts for capital, operations and maintenance and equipment replacement costs, the total present worth investment was estimated at $129 million and $74.6 million or $8.57/lb and $8.02/lb nitrogen removed, respectively.

Table 6.6-2 Capital and Operating Costs for 190-cfs Capacity Microscreen needed to achieve 45% total nitrogen load reduction goal.

System	Capital Costs	Annual Operating Costs	Equipment Replacement Costs[1]
	($)	($)	($)
Clearing and Grubbing	$121,000	$0	$0
Earthwork	$140,000	$0	$0
Intake and Inflow Pump Station	$2,688,00	$236,000	$1,774,000
Inflow Transmission Main	$639,000	$7,000	$0
Disc Filter (Microscreen)	$12,100,000	$662,000	$7,986,000
Discharge Channel	$1,036,000	$11,000	$0
Gravity Thickening	$1,657,000	$87,000	$1,094,000
Mechanical Dewatering	$8,990,000	$470,000	$5,934,000
Sludge Drying Beds	$1,852,000	$633,000	$0
Operations & Maintenance Bldg	$3,600,000	$54,000	$0
Alum Metering and Storage	$1,530,000	$337,000	$1,010,000
Pavement	$1,194,000	$12,000	$0
SUBTOTAL	**$35,544,000**	**$2,505,000**	**$17,797,000**
Land Acquisition [2]	$0	$0	$0
Engineering [3]	$5,332,000		
Total	$40,875,000	$2,500,000	$17,797,000
Present Worth Cost [4]	$40,875,000	$72,865,000	$16,021,000
Total Present Worth Cost	$129,760,000		
Per Pound Nitrogen Removed[5]	$8.57		

1 Replacement of equipment and material items every 20 years.
2 Land costs were not included in this present worth analysis. These costs and their influence on the present worth analysis are discussed in further detail in Section 7.0.
3 Estimated as 15% of capital costs.
4 Estimated at an interest rate of 5.625% for a 50-year period. Annual O&M costs were inflated at 3% per year. Salvage of equipment purchased at 40 years estimated at 1/3 the purchase value at the end of 50 years.
5 Listed cost based on estimated per pound nitrogen removed by microscreens over a 50 year operating period.

Table 6.6-3 Capital and Operating Costs for 68-cfs Capacity Microscreen needed to achieve 27% total nitrogen load reduction goal.

System	Capital Costs	Annual Operating Costs	Equipment Replacement Costs[1]
	($)	($)	($)
Clearing and Grubbing	$61,000	$0	$0
Earthwork	$88,000	$0	$0
Intake and Inflow Pump Station	$1,501,00	$139,000	$991,000
Inflow Transmission Main	$432,000	$5,000	$0
Disc Filter (Microscreen)	$5,920,000	$319,000	$3,907,000
Discharge Channel	$735,000	$8,000	$0
Gravity Thickening	$1,200,000	$63,000	$792,000
Mechanical Dewatering	$5,500,000	$288,000	$3,630,000
Sludge Drying Beds	$1,234,000	$412,000	$0
Operations & Maintenance Bldg	$1,800,000	$27,000	$0
Alum Metering and Storage	$790,000	$200,000	$522,000
Pavement	$842,000	$9,000	$0
SUBTOTAL	**$20,099,000**	**$1,465,000**	**$9,841,000**
Land Acquisition[2]	$0	$0	$0
Engineering [3]	$3,015,000		
Total	$23,113,000	$1,465,000	$9,841,000
Present Worth Cost[4]	$23,113,000	$42,593,000	$8,859,000
Total Present Worth Cost	$74,565,000		
Per Pound Nitrogen Removed[5]	$8.02		

1 Replacement of equipment and material items every 20 years.
2 Land costs were not included in this present worth analysis. These costs and their influence on the present worth analysis are discussed in further detail in Section 7.0.
3 Estimated as 15% of capital costs.
4 Estimated at an interest rate of 5.625% for a 50-year period. Annual O&M costs were inflated at 3% per year. Salvage of equipment purchased at 40 years estimated at 1/3 the purchase value at the end of 50 years.
5 Listed cost based on estimated per pound nitrogen removed by microscreens over a 50 year operating period.

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SECTION 7

EVALUATION OF TREATMENT ALTERNATIVES

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SECTION 7.0
EVALUATION OF TREATMENT ALTERNATIVES

In the previous sections, conceptual designs and cost estimates for treatment technologies to reduce the nitrogen load discharging from Lake Hancock were presented. This section presents the evaluation of these alternatives and this information, related to the following six quantitative/qualitative criteria:

1. Performance (quantitative criteria),
2. Cost effective (quantitative criteria),
3. Proven technological track record (qualitative criteria),
4. Simplicity of operations (qualitative criteria),
5. Permitable (qualitative criteria), and
6. Achieves District mission (qualitative criteria).

For reference, quantitative criteria are criteria whereby a number or value is either assigned or determined such as costs. This is in contrast with qualitative criteria where it is difficult to assign a specific number or value. Qualitative criteria include considerations that can be either positive or negative or describe the magnitude of success that is fulfilled by an alternative being considered.

The objective of this evaluation is to use these criteria to select highly ranked technologies for further evaluation and ultimately a primary technology for design and construction of an outfall treatment facility for Lake Hancock. It is important to note that the information used in this evaluation is based on conceptual-level designs. These by definition are preliminary in nature and based on many technical and construction assumptions that are not currently defined. Cost estimates that are generated based on this information are typically conservative (i.e., higher) to account for unknown issues that have not yet been identified, but will be resolved later in the design process when more detailed information is gathered. Important in the evaluation of these alternatives is to provide consistency so that the alternatives can be compared and evaluated based on the same assumptions and unit costs. Further analyses and investigations are planned in subsequent phases of this project that will refine the designs and cost estimates for the selected alternatives.

7.1 PERFORMANCE (QUANTITATIVE CRITERIA)

7.1.1 Comparison of Treatment Technology Performances

As listed in Table 7.1-1, the average removal efficiencies or performance for the eight treatment technologies vary from 45 to 75 percent with the highest and lowest efficiencies reported for Surface Flow Wetlands and the WHS™, respectively. Because treatment efficiencies are higher than the load reduction goals, only a portion of the discharge will need to be treated to reduce the nitrogen load to the targeted goals of either 45 or 27 percent. Thus the size of the alternatives vary based on the percentage of the average annual flow receiving treatment and the range of annual lake discharges as defined by the operational guidelines of the proposed LLMP as discussed in Section 2.3. Other ancillary improvements in water quality include reductions in phosphorus and suspended solids estimated to range from 45 to 95 percent and 80 to 95 percent, respectively.

Table 7.1-1 Estimated Removal Efficiencies for the Evaluated Treatment Alternatives[1].

Treatment Technology	Percent Removed [2]		
	Total Nitrogen (%)	Total Phosphorus (%)	Total Suspended Solids (%)
Average Lake Concentrations (mg/L)	5.530	0.603	115
Surface Flow Wetlands	65 - 75	80 – 90	95 – 99
Water Hyacinth Scrubber (WHS™)	40 - 50	45 – 55	85 – 95
Sedimentation Ponds	50 - 60	90 – 95	80 – 90
Sedimentation Basins	50 - 60	90 – 95	80 – 90
Sedimentation Basins and Filtration	60 - 70	90 – 95	90 – 95
Dissolved Air Flotation	50 - 60	90 – 95	80 – 90
Dissolved Air Flotation and Filtration	60 - 70	90 – 95	90 – 95
Microscreens	No data [3]	No data [3]	No data [3]

1 The reported removal efficiencies listed represent only that portion of the discharge treated by the technology. Because in many cases the goal is lower than removal efficiency, not all of the discharge needs to be treated. A portion of the discharge can be blended with the treated water flow thereby reducing the size, capacity and cost of the treatment plant.

2 Removal efficiencies were based on the average lake concentrations listed in table. Removal efficiencies would be expected to vary based on fluctuations in lake concentrations. Values listed apply to the concentration and do not reflect load reduction by a treatment technology.

3 The performance of microscreen filtration is unknown at this time. To achieve the required performance, a coagulant would need to be added to create a pin floc of sufficient size to be captured by the screen which until recently, had never been tested by the supplying vendor. For costing purposes, the same removal efficiency of the sedimentation basins was assumed.

7.1.2 Comparison of Aquatic Plant-Based Treatment Technologies

The nitrogen removal efficiencies provided by Surface-Flow Constructed Wetlands and WHS™ indicate that both technologies will be able to achieve either of the nitrogen goals. Because the removal efficiency of the Water Hyacinth Scrubber (WHS™) is lower, averaging 45 percent, more of the Lake discharge will need to be treated to achieve the targeted goals as compared to the other technologies. HydroMentia, the developers of the WHS™, reports in their proposal provided in Appendix D the need to capture and treat a maximum of 300-cfs (194-mgd) to achieve both targeted goals. This is in contrast with Surface-Flow Constructed Wetlands where flow requirements are substantially less, requiring only 110-cfs (71-mgd) or 52-cfs (34-mgd) to achieve the 45 or 27 percent goals, respectively. As a consequence, the capacity of the inflow / intake pump station and the WHS™ is much larger than if the removal efficiency were higher.

7.1.3 Comparison of Physical Treatment Technologies

The design and performance of the physical technologies, including Sedimentation Ponds, Sedimentation Basins and Dissolved Air Flotation (DAF), were based on jar tests preformed on collected samples from Lake Hancock as discussed in Section 3.0. This is in contrast with the aquatic plant-based technologies where the designs and performance were based on previous designs and studies. The expected nitrogen removal efficiencies for the physical technologies were estimated based on the jar test results. The performance of DAF is expected to be slightly better but without actual pilot test results it is not possible to estimate efficiencies at this time. Filtration, which was added after Sedimentation Basins and DAF to improve water quality as simulated in laboratory using laboratory filters, is expected to improve removal efficiencies for total nitrogen and total suspended solids (TSS) by 10 and 5 percent, respectively.

7.2 COST EFFECTIVENESS (QUANTITATIVE CRITERIA)

The capital, annual operation and maintenance (O&M) costs and actual land cost for the eight treatment alternatives are summarized in Table 7.2-1 for the 45% and 27% annual nitrogen reduction goals. The cost effectiveness of these technologies were evaluated based on 50-year facility-life present worth costs as listed in Table 7.2-1 in units of dollars per pound total nitrogen removed (i.e., $/lb TN). Two present worth costs were calculated and listed: one that considers all costs including land costs and the other that excludes land costs. This was done to compare the value of land used which varies depending upon area needed by each alternative technology. The proposed treatment alternatives were sited on District-owned land near the lake outfall. Biological technologies were sited on the former Old Florida Plantation (OFP) property and the physical technologies were located on the Saddle Creek property, west of the outfall. Land costs were calculated based on the actual purchase price paid in November 2003 and December 2004 for $30.5 million ($9,113 per acre) and $4.9 million ($24,776 per acre), respectively.

7.2.1 Cost Effectiveness to Achieve 45% Annual Nitrogen Reduction Goal

Based on present worth costs, conceptual plans provided by HydroMentia and additional design considerations and costs by Parsons, the WHS™ is the least costly alternative to achieve the 45% annual nitrogen reduction goal, estimated at $5.42/lb TN or $5.63/lb TN considering the cost of land. This includes an initial capital investment of $22.0 million, an estimated annual O&M cost of $1.9 million per year (2004 dollars) and an equipment replacement cost of $3.1 million every 20 years (2004 dollars). The next least costly alternative is Surface Flow Wetlands estimated at $6.38/lb TN or $8.19/lb TN considering cost of land. This includes an initial capital investment of $61.4 million, an estimated annual O&M cost of $0.9 million per year and an equipment replacement cost of $4.2 million every 20 years (2004 dollars). All other alternatives are higher ranging from $7.92 to $16.39/lb TN or $9.41 to $16.39/ lb TN considering the costs of land. Adding filters after sedimentation basins or DAF adds $6.98/lb TN to the present worth costs or $37 million in capital investment, $1.7 million in annual O&M and $21.4 million in equipment replacement for a slight marginal improvement in water quality. Filters are expected to improve removal efficiencies for total nitrogen and total suspended solids (TSS) by 10 and 5 percent, respectively.

The capital costs to construct the 2,540 acre Surface Flow Wetlands is the highest estimated at $61.4 million. These costs are high due to the large amount of excavation, transportation and fill needed to construct a wetland that covers a majority of the former Old Florida Plantation site. Because of elevation differences, a lift station is needed in the middle of the wetland, which adds $1.9 million to the capital costs, $0.1 million to annual O&M, and $1.3 million to replacement costs. However, since the total cost of O&M is relatively small compared to the other alternatives (i.e., estimated at $0.9 million/yr), the overall present worth cost is low even though capital costs are the highest for this alternative. This is in contrast with Sedimentation Basins, for example, where capital costs are lower at $40.3 million, however, due to annual O&M costs being substantially higher at $2.9 million/year the resulting present worth cost is higher, estimated at $9.25/lb TN as compared to $6.38/lb TN for Surface Flow Wetlands.

Critical to estimating the costs of both WHS™ and Surface Flow Wetlands is the estimate for earthwork (i.e., excavation and grading) and construction of levees. Because both alternatives utilize a large amount of land and require a large amount of excavation to create treatment cells and levees, the total capital investment is highly sensitive to earthwork costs. At this conceptual level of design, it is difficult to determine the level of earthwork needed as detailed land and geotechnical surveys have not been initiated nor have grading plans been developed and prepared. These activities are initiated later in the design process in designing a selected alternative.

Also critical to estimating the costs of the WHS™ is understanding the costs associated with harvesting, processing and disposal of residuals. Given the estimated 140 wet tons of hyacinths harvested and composted per day and the labor and equipment involved, it is critical to understand the operation and maintenance of this system to better define costs. The WHS™ estimates listed in this report and provided in Appendix D are based on design criteria, operational conditions and maintenance requirements for smaller, pilot-scale facilities ranging in size from 1.5 acres to 32 acres. Currently there are no operational pilot or full-scale WHS™ in existence to visit or evaluate operating and maintenance conditions, nor has any WHS™ been designed and operated at the 88 and 210-acre sizes contemplated for this project.

7.2.2 Costs to Achieve 27% Annual Nitrogen Reduction Goal

Based on present worth costs, both the Surface Flow Wetlands and the WHS™ are the least costly alternatives to achieve the 27% annual nitrogen reduction goal. Surface Flow Wetlands is estimated at $4.79/lb TN or $5.99/lb TN considering the cost of land, and WHS™ is estimated at $5.68/lb TN or $5.83/lb TN considering the cost of land. All the other technologies are substantially higher ranging from $8.15 to $13.71/lb TN. The lower relative cost for the Surface Flow Wetlands, in this case, is afforded by optimizing the use of existing levees in the clay settling areas of the OFP site.

7.2.3 Selection of a Nitrogen Reduction Goal

To select which of the nitrogen goals to use as a basis for design, the improvement in water quality index for the Peace River at Bartow (i.e., the Benefit) was estimated based on increasing nitrogen load reduction goals and compared with associated capital costs (i.e., the Cost) as illustrated in Figure 7.2-1 (SWFWMD, 2006). At the 27 percent load reduction goal, the

Table 7.2-1 Costs for annual nitrogen load reduction in Lake Hancock Outfall for the alternative treatment technologies evaluated.

Technology	45% Nitrogen Load Reduction [1]					27% Nitrogen Load Reduction [2]				
	Capital Cost[3] ($)	Annual O&M Cost[3] ($)	Land Costs ($)	Present Worth Cost without LandCost[4] ($/lb TN)	Present Worth Cost with Land Costs[5] ($/lb TN)	Capital Cost[3] ($)	Annual O&M Cost[3] ($)	Land Costs ($)	Present Worth Cost without LandCost[4] ($/lb TN)	Present Worth Cost with Land Costs[5] ($/lb TN)
Surface Flow Wetlands	$61.4 M	$0.9 M	$24.3 M	$6.38	$8.19	$19.6 M	$0.7 M	$10.5 M	$4.79	$5.99
Water Hyacinth Scrubber (WHS™)	$22.0 M	$1.9 M	$ 3.1 M	$5.42	$5.63	$16.2 M	$1.1 M	$1.4 M	$5.68	$5.83
Sedimentation Ponds	$31.6 M	$2.7 M	$ 5.0 M	$7.92	$8.24	$18.7 M	$1.8 M	$4.0 M	$8.15	$8.57
Sedimentation Basins	$40.3 M	$2.9 M	$ 2.5 M	$9.25	$9.41	$22.6 M	$1.7 M	$1.4 M	$8.74	$8.89
Sedimentation Basins and Filtration	$77.6 M	$4.6 M	$ 2.5 M	$16.23	$16.39	$38.4 M	$2.4 M	$1.4 M	$13.55	$13.71
Dissolved Air Flotation (DAF)	$39.7 M	$2.9 M	$ 2.5 M	$9.13	$9.30	$22.3 M	$1.7 M	$1.4 M	$8.64	$8.80
Dissolved Air Flotation and Filtration	$77.0 M	$4.6 M	$ 2.5 M	$16.12	$16.28	$37.0 M	$2.4 M	$1.4 M	$13.11	$13.26
Microscreen (Disc Filters)	$40.9 M	$2.5 M	$ 2.5 M	$8.57	$8.73	$23.1 M	$1.5 M	$1.4 M	$8.02	$8.18

1 Initial goal.
2 The 27% reduction goal was based on the expected performance of surface flow wetlands treating a flow rate of 52-cfs. All other technology capacity's were adjusted to achieve a 27% load reduction based on respective treatment efficiencies.
3 Costs were estimated in December 2004 dollars.
4 Listed costs are reported per pound nitrogen removed by each technology based on present worth investment that includes capital, annual operation, maintenance and equipment replacement costs for a period of 50 years. Cost of land is not included.
5 Listed costs are reported per pound nitrogen removed by each technology based on present worth investment that includes capital, annual operation, maintenance, equipment replacement costs and cost of land for a period of 50 years. Cost of land based on purchased price per acre as reported by Southwest Florida Water Management District and the required footprint needed for each treatment technology. Land costs were based on the purchase price for the Old Florida Plantation property purchased in November 21, 2003 at a cost of $30,500,000 ($9,113/acre) and the Saddle Creek property on December 30, 2004 at a cost of $4,900,000 ($24,776/acre).

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resulting water quality index improves from 90 to 58, the water quality designation shifts from "Poor" to "Fair", and the projected load increase at Charlotte Harbor from the Peace River Basin is off-set for next 19 years for an estimated capital cost varying between \$16.2 to \$38.4 million. At the 45 percent load reduction goal, the resulting improvement in water quality index is marginally better at 50, the water quality index remains "Fair", and the projected load increase at Charlotte Harbor from the Peace River Basin is off-set for next 32 years for an estimated cost that is significantly greater at \$22.0 to \$77.6 million. Because the costs do not justify the marginal benefits afforded by a 45 percent nitrogen load reduction goal, a 27 percent nitrogen load reduction goal is recommended for the basis of design of the outfall treatment system.

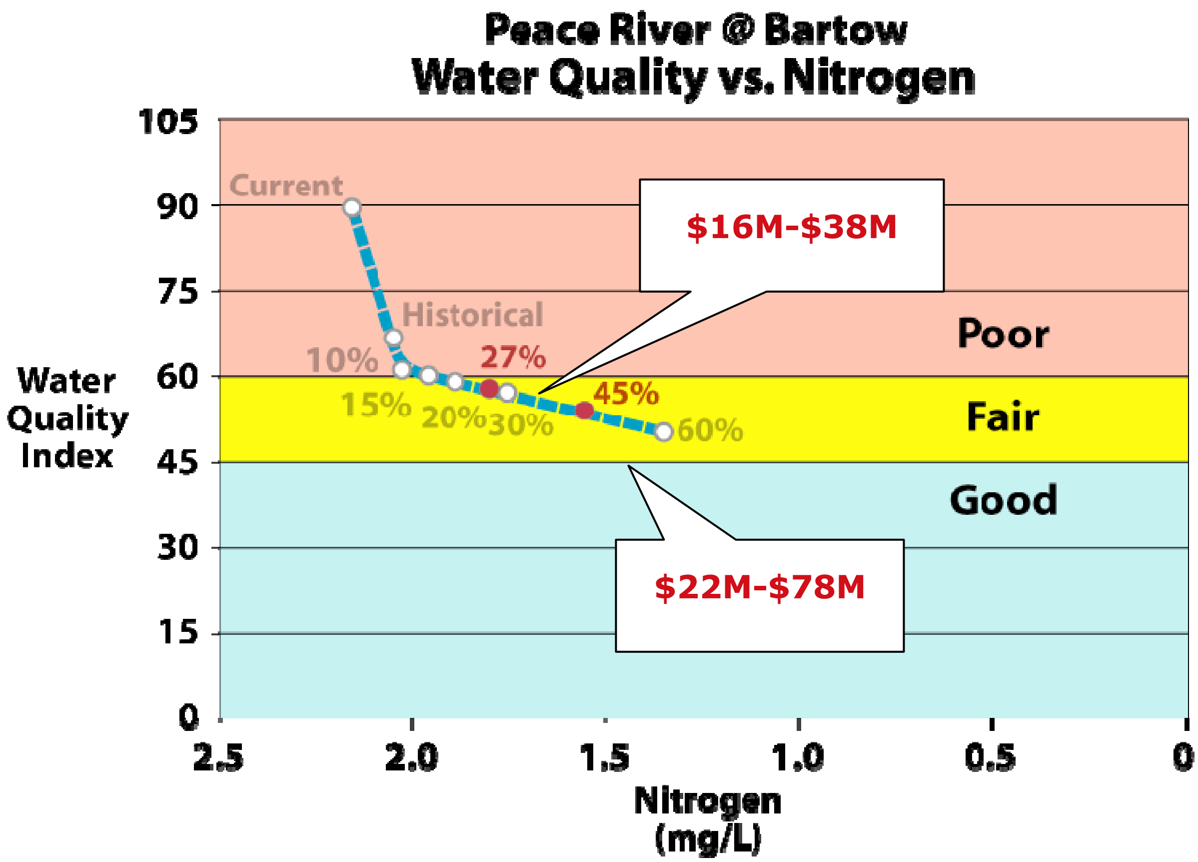

Figure 7.2-1 Estimated Water Quality Improvements with Treatment Reduction Goals of 27 and 45 Percent (SWFWMD, 2006). Costs shown in the plot were modified from original plot provided in reference to reflect updated costs.

7.2.4 Surplus Land Value

The District Governing Board approved the purchase of the Old Florida Plantation, "OFP" property for \$30,500,000 in October 2003 recognizing that a number of benefits could be realized, including water quality benefits to the Peace River and Charlotte Harbor. The Governing Board also recognized the opportunity to surplus portions of the property not necessary for District Projects. The OFP property has a vested DRI approval. The District has preserved the development rights and could potentially offset future land acquisition funding needs by surplusing the portions not necessary for District projects. In February 2006, an updated appraisal of the OFP property estimated land value of approximately $24,000/ per developable acre. The primary reason for purchase was to protect an important conservation corridor as identified in the District's land acquisition plan that specifically identified the Upper Peace River Corridor in 1996 and Lake Hancock project (addition to the Upper Peace River Corridor) in 1999 for acquisition. The OFP property is a significant and integral piece of a potential, extensive corridor along the Peace River connecting to the Green Swamp. Other potential benefits of the purchase were associated with the Southern Water Use Caution Area recovery strategy, where a portion of the property may be inundated through a lake level modification and water quality would be enhanced through a surface water treatment system. Eight technologies were evaluated with each having differing site requirements, varying in size from nearly 60 acres to over 2,600 acres. In addition to the present worth costs presented in Sections 5 and 6, all alternatives were evaluated based on actual land expenditures (2004). At the 27 percent annual nitrogen load reduction, the footprint of the two most cost effective treatment technologies, treatment wetlands and WHS™, encumbers approximately 306 and 114 acres of developable land. Placement and development of some of the proposed technologies may likely be in direct conflict with the existing DRI approvals. A treatment wetland constructed in the clay settling area of the property is considered the most consistent with the existing DRI approval.

7.3 TECHNOLOGY TRACK RECORD (QUALITATIVE CRITERIA)

The dependability of a technology is evaluated through documentation of successful systems implemented at an equivalent scale and function. All of the technologies evaluated have a well documented history in treating both water and wastewater and facilities of comparable size are in existence and in operation with the exceptions of the WHS™ and microscreens using coagulant addition.

Although the use of water hyacinths for wastewater treatment has been documented since the 1970’s as cited in the HydroMentia proposals (see Appendix D), none of the WHS™ systems have been in operation for a period of more than 3 years and currently there are no existing operating systems in the US. The last operating system was a 0.8-cfs (0.5-mgd) combination WHS™ - Algal Turf Scrubber (ATS™) prototype system, located in the Lake Okeechobee watershed, that operated from January 2003 to October 2004. The South Florida Water Management District (SFWMD) contracted with HydroMentia for this demonstration scale project that was funded through the Phase II Phosphorous Source Control Grant Program. The ATS™ technology is being further investigated through a second demonstration scale project for the SFWMD. They have contracted for the design and construction of a 4-acre ATS™ demonstration-scale project in the Taylor Creek/Nubbin Slough Basin of Lake Okeechobee that is anticipated to be operating by the end of 2006. The Florida Department of Agriculture and Consumer Services is providing funding for the project from a Phosphorus Source Control Grant. Previous WHS™ systems implemented by HydroMentia have ranged in size between 1.5 to 32 acres and in capacity from 0.23 to 9 cubic feet per second (cfs). None of these systems have approached the 88 and 210-acre scale or 300-cfs capacity proposed for Lake Hancock. Since a system of similar size has not been constructed or operated, the information provided by HydroMentia in their proposals can not be verified.

Microscreens have been used as pretreatment to drinking water treatment plants and in treating treated wastewater effluents prior to discharge to removal particulate matter. Because nearly 70 percent of the total nitrogen in Lake Hancock is associated with particulate matter, microscreens were considered. Unfortunately, the majority of the particulate matter is less than 10 um in size which is less than the smallest size microscreen available on the market today. As a possible process improvement, addition of alum as a means to increase particle size by forming a pin-floc that could be captured and subsequently removed by the microscreens was considered. The equipment manufacturer was contacted and unfortunately, had only recently begun testing the use of coagulants in conjunction with microscreens. Although the results appear to be positive, there is no track record or existing facilities of similar size using a coagulant in this manner.

7.4 SIMPLICITY OF OPERATIONS (QUALITATIVE CRITERIA)

The simplicity of operating and maintaining a system is measured by the level of effort and can be directly related to the number of employees, the amount and complexity of the equipment needed, and costs to operate a facility. Typically more intensive operations require more operators and equipment at increased annual costs. However in considering the simplicity of a system, it is important to consider all costs. A simple to operate and maintain technology may have higher capital costs but lower O&M costs compared to other more complex technologies. The present worth analysis presented in Section 7.2 incorporates all of these costs into a single value so that the costs between the technologies can be easily compared.

The Surface Flow Wetlands is probably the simplest technology to operate and maintain given it is a passive system requiring minimal operating and maintenance staff most of the time. Except for operations and maintenance of the pump stations (which are a requirement of all the technologies), most activities are associated with monitoring of water quality and maintenance of monitoring stations, maintenance of water control structures, maintenance of service roads, site security, and overall daily inspection of the facility. These same activities are required of all technologies. The build-up of sediments which has been an operational issue at other wetlands, such as the Orlando Easterly, has been accommodated by design of deeper wetland cells that allow for the build-up of flocculent and accreted sediments over the 50-year life of the facility (see Figure 5.2-6 and related discussion).

The WHS™ is much more labor intensive requiring regular scheduled harvesting and composting of hyacinths using mostly standard farm equipment and specialized equipment designed and fabricated by HydroMentia. Sediment build-up in the hyacinth ponds (which is estimated at 32,000 cubic yards per year at the 45% goal) is removed from the ponds using remote controlled dredges on a quarterly basis and combined manually with the hyacinths and composted. Resulting compost must be trucked off-site and either sold as a soil amendment or feed or trucked to a landfill at an additional cost. As with all the alternatives, O&M includes operations and maintenance of an inflow pump station, maintenance of monitoring stations, maintenance of service roads, site security, and overall daily inspection of the facility. In addition to these, O&M will also include maintenance of the harvesting, composting, dredging and hauling equipment, operations and maintenance of the aeration equipment and other miscellaneous equipment. Because hyacinths can be prone to insect infestation and other aggressive plant species, additional attention will be needed to maintain a viable and healthy hyacinth crop including periodic use of pesticides and herbicides. Additional nutrients may also need to be added to ensure a viable hyacinth crop.

For the physical treatment alternatives (i.e., sedimentation ponds, basins, DAF and microscreens), labor intensity varies but compared with the biological alternatives is much more labor intensive than wetlands and probably comparable with WHS™. Handling and management of residuals is likely the most labor intensive aspect of the physical systems and is a major capital and operations expense. Sludge generated from coagulation of sediments in the water with alum addition accounts for an average of 1,400 to 2,300 cubic yards or 1,200 to 1,900 wet tons of sludge per day at the 27 and 45 percent reduction goals, respectively. To allow for the sludge to be trucked to a landfill for disposal, the sludge will need to be dewatered to at least 30 percent solids. Dewatering is a three step process requiring gravity thickening, mechanical dewatering and drying beds. Mechanical dewatering which is typically done with either belt filter presses or centrifuges should achieve at least 15 percent solids. Drying beds which allow the sludge to air dry in the sun should be able to achieve 30 percent solids. Dewatered sludge would be collected from the drying beds using front end loaders and hauled off-site to a landfill. At each step, pieces of equipment are involved that require operator attention and routine maintenance. Given the amount of sludge generated by these technologies, handling, hauling, and disposal operations will require dedicated full-time operators and flexible staffing to account for these wide variations.

Compared to the other physical treatment technologies, Sedimentation Ponds provide the advantage of being relatively simple to design, construct and operate. Sedimentation Basins and DAF provide the advantage of requiring less land and a means of removing sludge on a continuous basis. Sedimentation Ponds are designed to store a month production of sludge at the maximum capacity. This has a distinct advantage in stabilizing the process handling of sludge, especially during times when the treated flow rate varies.

7.5 PERMITABLE (QUALITATIVE CRITERIA)

The permitable criterion describes not only the success of obtaining all permits from all applicable permitting agencies but also attaining approvals from all associated stakeholders as well as the relative amount of effort and time needed to obtain these permits and approvals. In addition to meeting environmental resource permit requirements, other agencies and stakeholders will need to be involved; which agencies and stakeholders will, to some extent, depend on the selected technology. Agencies and stakeholders include Florida Department of Environmental Protection (FDEP), Polk County, City of Bartow, US Army Corps of Engineers (USACE), and potentially the Federal Aviation Administration (FAA), Florida Department of Transportation (FDOT), and the Florida Fish and Wildlife Commission. Details of permitting and other regulatory issues in regards to each technology are discussed in Sections 5 and 6. Some of the more important issues are discussed here.

An Environmental Resource Permit (ERP) from FDEP will be required for all technologies under consideration. Wetland impacts associated with construction of a Surface Wetland system will likely be self mitigating and require a Noticed General ERP. This is not the case with all other technologies. In addition, additional impervious area associated with the physical technologies will require an Individual ERP.

The Surface Wetlands impact limited areas of existing wetlands and therefore will require permitting through the USACE. The ERP application will be made jointly to FDEP and the USACE. Authorization by the USACE for the Surface Wetlands could be made under a Nationwide Permit (NWP). This type of permit is activity specific, and is designed to relieve some of the administrative burdens associated with permit processing for both the applicant and the Federal government. It is anticipated that the Surface Wetlands would qualify for a NWP Number 27 covering wetland restoration. For the physical technologies located on the Saddle Creek property, existing wetlands are not expected to be impacted.

Specific regulatory approvals that differentiate the technologies under consideration are associated with management of exotic/invasive plant species, residuals handling and Aviation Authority coordination in conjunction with creation of a wildlife hazard to aircraft.

Implementation of a WHS™ requires an Aquatic Plant Management Permit from the Florida Bureau of Invasive Plant Management for stocking, harvesting and disposal of water hyacinths. Water hyacinths are listed as Class I Prohibited Aquatic Plants under F.A.C Chapter 62C-52. The level of effort and degree of difficulty in processing this permit is unknown at this time; however, HydroMentia, Inc. has been successful in permitting smaller scale pilot systems in the past.

The residuals generated from chemical coagulation using alum in all of the physical treatment technologies and biosolids generated from WHS™ will need to be disposed or reused. The physical treatment technologies will generate an alum sludge and the WHS™ will generate a hyacinth compost combined with inert solids. The constructed wetlands are not expected to generate residuals that will need to be disposed of off-site. If a beneficial use can not be identified for these residuals then per F.A.C. Chapter 62-701, these residuals will need to be taken to a permitted or exempt solid waste facility. Florida Statues (FS) Section 403.7045 does allow FDEP to exempt residuals which are beneficially used from regulation. There are three conditions for exemption:

1. A majority of the industrial byproducts are demonstrated to be sold, used, or re-used within one year;

2. The industrial byproducts are not managed as to create a threat of contamination in excess of Department standards and criteria; and

3. The industrial byproducts are not hazardous wastes

FDEP has been contacted about the reuse of alum sludge generated from a similar process proposed for the St. Johns River Water Management District. FDEP is concerned about heavy metals in the alum residual, with specific concern related to arsenic. In order to be exempt, the alum residual generated from the physical technologies would need to be sampled and tested using Toxic Characteristic Leaching Procedure (TCLP) and Synthetic Precipitate Leaching Procedure

(SPLP) to identify if leaching of heavy metals from the residuals would be a concern; the results would dictate the disposal options.

The beneficial reuse of the hyacinth/sediment compost and its possible exemption under FS Section 403.7045 is unknown at this time. It has been reported by HydroMentia, Inc., the hyacinth/sediment compost from their pilot systems has been used in the past as a soil amendment for agriculture and also as cattle feed. However a viable, sustainable market for this product has not been established at this time.

Due to the proximity of the District's former OFP property to the Bartow Municipal Airport, coordination with the airport authority will be necessary. Any type of development on the District's property must be consistent with the aviation easement associated with the airport. Based on discussions with airport representatives, it is not anticipated that a wetland treatment system constructed on the District's OFP property would adversely affect the airport's operations.

7.6 ACHIEVES DISTRICT MISSION (QUALITATIVE CRITERIA)

While the primary objective of the Lake Hancock project is to reduce the annual nitrogen load discharging from the lake, the treatment technologies evaluated have the potential to offer ancillary benefits that align with the Districts Mission and benefit local communities, Polk County and the residents of the State of Florida.

The mission of the Southwest Florida Water Management District is to manage the water and water-related natural resources to ensure their continued availability while maximizing environmental, economic and recreational benefits. To this end, the District has established areas of responsibility including: Water Supply, Flood Protection, Water Quality Management, and Natural Systems Management. As related to this project, the responsibilities are restated below (SWFWMD, 2005a)

- Water Supply - Ensure an adequate supply of the water resource for all existing and future reasonable and beneficial uses, while protecting and maintaining water resources and related natural systems.

- Flood Protection - Minimize flood damage by optimizing and maintaining storage and conveyance in natural and built systems, and by encouraging appropriate locations and design standards for growth.

- Water Quality Management - Protect water quality by preventing further degradation of the water resource and enhancing water quality where practical.

- Natural System Management - Preserve, protect and restore natural systems in order to support their natural hydrologic and ecologic functions.

A qualitative examination of these secondary benefits is done through the measure of potential for such things as wildlife habitat creation, water storage, recreation benefits. Table 7.3-1, discussed in Section 7.7, summarizes the qualitative ranking of the alternatives.

7.6.1 Wildlife Habitat

The creation and expansion of natural wildlife habitat supports the function of natural systems. Alternatives that offer no or limited habitat value were assigned a low ranking whereas systems that offered the potential for functional, diverse habitat rank higher. At this time no attempt was made to determine the amount of change in habitat or evaluate the value of habitat outside of the respective technology footprints. Of the technologies evaluated, only the proposed surface flow wetlands offer the potential of added benefit to wildlife habitat. Physical technologies and WHS™ require active maintenance activities which precludes wildlife. Furthermore, the WHS™ consists of a monoculture of an exotic plant species that are not characterized as a good wildlife habitat.

7.6.2 Water Storage

Water storage potential of the respective systems can also be considered an ancillary benefit towards the District's mission of natural systems management and water supply. Benefits to water supply are symbiotic to the improvements in the water quality of Saddle Creek in as it serves as headwaters to the Peace River watershed and potentially beneficial the District's Minimum Flow Level goals.

Water storage was evaluated based on the amount of excess volume available with a higher qualitative rank assigned to the larger the detention volume. Due to its large footprint, the Surface Wetland provides over 20 times the storage capacity of the WHS™ and over 130 times the storage capacity of Sedimentation Ponds, irrespective of treatment goal. However it should be noted that the actual amount of storage utilized and its affects on the minimum flow level program will be a function of the operating scheme of the system with respect to pump capacity and available free board storage capacity.

7.6.3 Recreation

For the qualitative evaluation of recreation opportunities, only compatible opportunities created within the footprint of the alternative were considered. Public access to nature trails, horseback riding, jogging paths and wildlife viewing areas are examples of recreation opportunities afforded by constructed wetlands. These public amenities are consistent with the District's Natural Systems Ecosystem policy to provide opportunities for compatible recreation activities on District-owned land. Therefore wetlands systems received a high qualitative ranking. Whereas the smaller size and need for intensive maintenance activities precludes such use at the physical and MAPS systems and in turn were given a low qualitative ranking.

7.7 SUMMARY

In order to attain an overall ranking of the technologies, each criterion was assigned either a "Low", "Medium" or "High" ranking based on a relative comparison between the other technologies. The only exceptions were "Technology Track Record", "Permitable", and "Recreation" where "Yes" or "No" were assigned indicating the technology satisfied the criterion or not and "Operational Simplicity" where "Simple" or "Complex" were assigned. Table 7.3-1 summarizes the results. Positive results were highlighted in **bold** font in the table. An overall ranking, which was assigned "Low", "Medium" or "High, was given to each technology based on a qualitative interpretation of the individual results shown in each row.

Cost effectiveness was ranked based on a relative scaling between the technologies using statistical analysis of the present worth costs. The following criteria were used:

- A technology was ranked as having "High" cost effectiveness when the present worth costs fell below the 25 percentile of the cost data.
- A technology was ranked as having "Medium" cost effectiveness when the present worth costs fell between the 25 and 75 percentile of the cost data.
- A technology was ranked as having "Low" cost effectiveness when the present worth costs fell above the 75 percentile of the cost data.

Performance was ranked on the process removal efficiency as listed in Table 7.1-1 according to the following:

- A technology was ranked "High" if the process removal efficiency was greater than 60 percent.
- A technology was ranked "Medium" if the process removal efficiency was greater than 50 to 60 percent.
- A technology was ranked "Low" if the process removal efficiency was less than 50 percent.

Operational simplicity was ranked based on the intensity of operating requirements and simplicity of the system. Surface Flow Wetlands was ranked high because of limited use of equipment and limited staff required to operate and maintain the system.

All technologies were considered permitable and were assigned "Yes". Each will require some level of permitting effort that includes an Environmental Resource Permit (ERP), FDEP permit, and local construction permits and may require a National Pollutant Discharge Elimination System (NPDES) permit. In addition to these, the WHS™ will require an Aquatic Plant Management Permit and Surface Flow Wetlands may require a 404 permit.

Wildlife habitat was ranked high for wetlands and low for the other technologies because only wetlands was considered as supporting a more diverse population of wildlife within the project limits.

Wetlands also was the only technology that ranked high for water storage potential as the other technologies provide only a faction of the storage capacity offered by wetlands.

7.8 CONCLUSION

Based on the evaluation of the technologies using the quantitative/qualitative criteria listed in Table 7.3-1, Surface Flow Wetlands ranks much higher than the other technologies considered. Comparing the ranking of the two most cost-effective alternatives Surface Flow Wetlands and the Water Hyacinth Scrubber (WHS™); Surface Flow Wetlands ranks higher for all the other criteria except for “Permitable” which both were assigned “Yes”. Surface Flow Wetlands has a well documented track record in the United States with over 45,000 acres currently permitted and in operation in Florida alone. Although there have been Water Hyacinth Scrubber (WHS™) systems in operation in the past, none are currently in operation and for those that were, none were at the scale proposed for this project. Surface Flow Wetlands are simple to operate and maintain, requiring minimum staff and equipment. This is in contrast with Water Hyacinth Scrubber (WHS™) which requires routine harvesting, composting and disposal of the composted hyacinths. Surface Flow Wetlands is not expected to generate a residual that will require disposal. Surface Flow Wetlands is also expected to provide additional wildlife habitat, recreation opportunities and water storage which could potentially benefit the District in further achieving Minimum Flow Level goals discharging Lake Hancock.

Table 7.3-1 Summary of quantitative and qualitative criteria used in evaluating treatment technologies.

Treatment Technology	Quantitative Criteria		Qualitative Criteria						Overall Ranking
	Cost Effectiveness	Performance	Technology Track Record	Operational Simplicity	Permitable	Wildlife Habitat	Water Storage	Recreation	
Surface Flow Wetlands	**High**	**High**	**Yes**	**Simple**	**Yes**	**High**	**High**	**Yes**	**High**
Water Hyacinth Scrubber (WHS™)	**High**	Low	No	Complex	**Yes**	Low	Low	No	Low
Sedimentation Ponds	Medium	Medium	**Yes**	Complex	**Yes**	Low	Low	No	Low
Sedimentation Basins	Medium	Medium	**Yes**	Complex	**Yes**	Low	Low	No	Low
Sedimentation Basins w\ Filtration	Low	High	**Yes**	Complex	**Yes**	Low	Low	No	Low
Dissolved Air Flotation	Medium	Medium	**Yes**	Complex	**Yes**	Low	Low	No	Low
Dissolved Air Flotation w\ Filtration	Low	High	**Yes**	Complex	**Yes**	Low	Low	No	Low
Microscreen (Disc Filters)	Medium	Low	No	Complex	**Yes**	Low	Low	No	Low

SECTION 8

RECOMMENDATIONS

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SECTION 8.0

RECOMMENDATIONS

- Based on evaluation of the alternatives, Surface Flow Wetlands leads the other technologies in meeting the project evaluation criteria.
- Select an annual total nitrogen load removal goal of 27 percent for the project.
- Further development of the Surface Flow Wetlands option is recommended that includes the following:
    - Gather further information on construction techniques and costs, system performance, water quality, and soil conditions of proposed site.
    - Prepare a preliminary design of Surface Flow Wetlands and update cost estimates.
- In conjunction with Surface Flow Wetlands demonstration-scale study, further investigate opportunities to incorporate modifications or system components aimed at enhancing and/or optimizing system performance.
- Monitor performance of Taylor Creek ATS™ implemented by SFWMD.
- If wetland performance, preliminary design and cost estimates are acceptable to stakeholders, then prepare final design for permitting, bidding and construction.

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SECTION 9

REFERENCES

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SECTION 9.0

REFERENCES

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APPENDICES A THROUGH I

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