# PROCEEDINGS

## *6th Biennial Stormwater Research and Watershed Management Conference*

Southwest Florida Water Management District

**September 14–17, 1999 • Tampa, Florida**

**PROCEEDINGS OF THE SIXTH BIENNIAL STORMWATER RESEARCH AND WATERSHED MANAGEMENT CONFERENCE**
**SEPTEMBER 14-17, 1999**
**FOUR POINTS SHERATON HOTEL TAMPA EAST**
**TAMPA, FLORIDA**

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## FOREWORD

This conference is the sixth in a continuing series of symposia sponsored by the Southwest Florida Water Management District to disseminate the findings of current stormwater research, as well as the latest developments in watershed management. The conference was designed to provide a forum from which a wide range of stormwater treatment and watershed management ideas and issues could be discussed and debated, and where research results could receive initial peer review. The ultimate goal of the conference is to present the engineers, scientists, and regulators working in the field of stormwater and watershed management with the most current ideas and data available so that more efficient and cost-effective best management practices can be developed and implemented. It is our hope that this conference and these proceedings will contribute to this goal.

The Sixth Biennial Stormwater Research and Watershed Management Conference was held September 14-17, 1999 at the Four Points Sheraton Hotel, Tampa, Florida and was attended by more than 250 government and consulting professionals. Thirty-six papers documenting various aspects of current stormwater research projects were presented. The conference proceedings include thirty complete papers and abstracts of the remaining six. Only the abstracts were printed for papers not available at the time the proceedings were compiled. The complete papers for these abstracts may be added to this document at a later date.

Craig W. Dye
Betty T. Rushton

## ACKNOWLEDGMENTS

**Diane Caban**, the Administrative Secretary for the Environmental Section at the Southwest Florida Water Management District, was the person most responsible for the detailed planning and administration necessary to organize and conduct the Sixth Biennial Stormwater Research and Watershed Management Conference and the publication of the Proceedings. **Barbara Pavone** of the Ecologic Evaluation Section, was responsible for compiling and assembling the Proceedings. **Gwen Brown** and **Josie Guillen** of the Resource Management Department, provided additional valuable administrative support for the Conference. **Allen Yarbrough** of the Visual Communication Section, designed the conference logo and provided significant additional graphics support. **John Frascone** of the District's Office Support Section, oversaw the printing of all conference materials, including the Proceedings. **Keith Kolasa** of the Environmental Section, oversaw converting the proceedings to digital format. Finally, **Tom Goodson** of the Technical Support Section, provided all Internet programming needed to assemble, format, and link the proceedings to the District's Web site.

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# BULK ATMOSPHERIC DEPOSITION OF NUTRIENTS AND METALS IN THE TAMPA BAY REGION OF FLORIDA

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## ABSTRACT

Bulk atmospheric deposition collections were performed weekly at ten locations in the Tampa Bay watershed between April 30, 1997, and February 3, 1998. Samples were analyzed for total nitrogen and phosphorus by the Environmental Protection Commission of Hillsborough County. Metal samples were volume-cornposited into approximately monthly intervals and analyzed by Mote Marine Laboratory for lead, copper, cadmium, and zinc. Annualized bulk deposition rates indicate that local activities were important in controlling the deposition of both nutrients and metals. The variation among sites in nutrient deposition was in addition to a substantial apparent background level of atmospheric loadings. There were seasonal variations in atmospheric loadings, particularly for nitrogen, with the lowest loadings during the fall quarter, and the highest loadings during the summer quarter. Wet-only deposition was collected independently at two additional sites, and dry-deposition collected at one of the two sites. Wet-only plus dry deposition of inorganic nitrogen had a highly significant relationship with the bulk deposition of total nitrogen, supporting the utility of the bulk method of collection for nitrogen. Statistically significant linear relationships were also present between the bulk and wet-only deposition of nutrients. Differences were attributed to dry deposition. At the urbanized site where metals were analyzed, wet-only deposition comprised only a portion of bulk or total deposition: total phosphorus (55%), copper (30%), lead (57%), and zinc (42%), and indicated that dry deposition was substantial.

## INTRODUCTION

Bulk atmospheric deposition samples were collected weekly over a 40 week period (4/30/97-2/3/98) at ten locations (Figure 1) in or near the Tampa Bay watershed. Table 1 outlines the sampling agencies for each site and their cooperation is gratefully acknowledged. Nutrient samples were analyzed weekly for total nitrogen (total Kjeldahl nitrogen and nitrate-nitrite-nitrogen) and total phosphorus at Environmental Protection Commission of Hillsborough County (EPCHC). Metal samples were volume composited into approximately monthly intervals and analyzed for copper, cadmium, lead, and zinc at Mote Marine Laboratory (MML). This project was funded by the Tampa Bay Regional Planning Council and Tampa Bay Estuary Program (Dixon et *al.*, 1998).

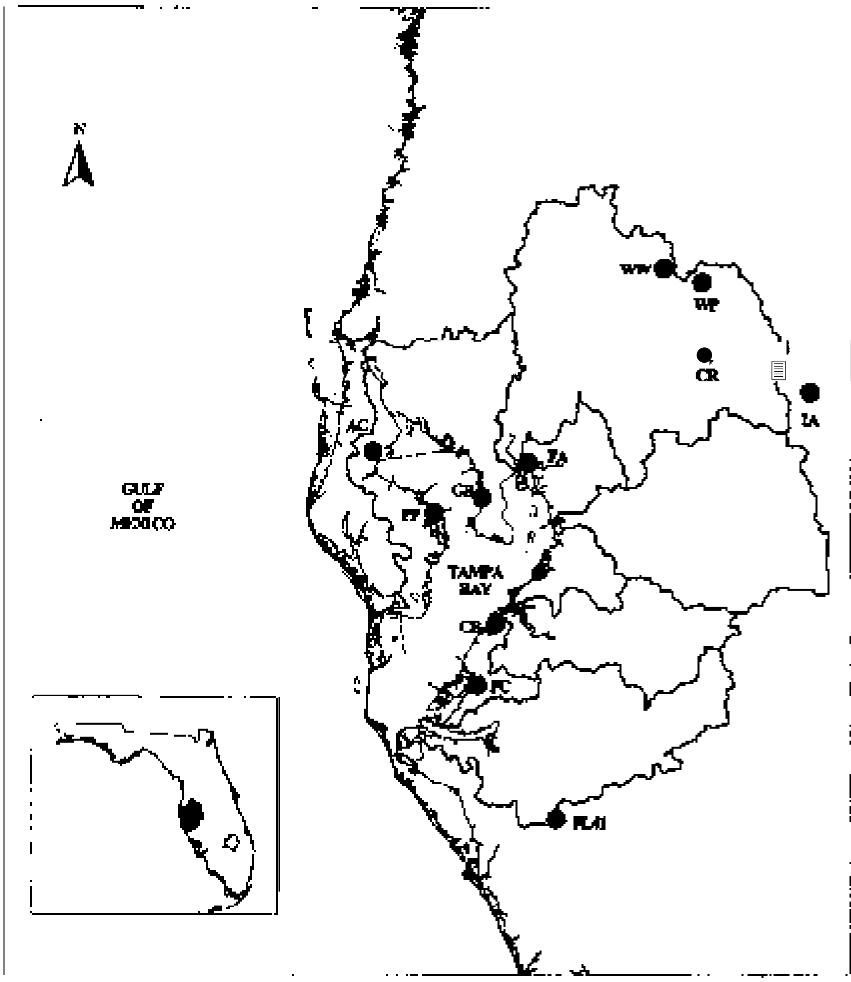


Figure 1. Site locations. FL41 is a wet only site maintained by the NADP/NTN national network,

Table 1. Agencies responsible for sample collection.

| **Agency** | **Sampling Site** |
|---|---|
| Polk County Natural Resources & Drainage | Inwood Alum (IA) |
| City of Tampa/Hillsborough County, Public Works - Stormwater Section | Florida Aquarium (FA) |
| | Gandy Bridge (GB) |
| | Cone Ranch (CR) |

| | |
|---|---|
| Manatee County Government Environmental Management Dept. | Cockroach Bay (CB) |
| | Frog Creek (FC) |
| Pinellas County Dept. of Environmental Management | Alligator Creek (AC) |
| | Pinellas Park (PP) |
| Pasco County Stormwater Management Division | Wildlife Preserve (WP) |
| | Wastewater Plant (WW) |

## METHODS

Bulk atmospheric samplers consisted of 113 $cm^2$ polycarbonate funnels at 3 m above grade. Nylon monofilament was stretched near funnel mouths to reduce bird contamination. Samples were collected in polyethylene bottles attached to the funnels via Teflon tubing. Separate funnels were used for nutrient and metal samples and cleaned equipment was provided by MML each week. At collection, the funnels were temporarily covered with polyethylene bags, the apparatus was returned to the samplers' office, and the funnel was then rinsed into sample bottles (with preacidified rinse water) to collect the dry as well as wet deposition. Sections of the funnels with non-representative contamination were not rinsed into the sample bottles. Sample bottles could collect approximately 9.5 cm of rain before overflow. If overflow occurred, the funnel was rinsed into a second sample bottle for that week. Equipment blanks were collected each week, by each sampling agency and analyzed along with each batch of rinse water for each analyte. Potential contamination was assessed on each sample for bird, insect, frog, particulate, and pollen. Samples were shipped to analytical laboratories (EPCHC for nutrients, MML for metals) where they were fully acidified for preservation until analysis.

## RESULTS AND DISCUSSION

**Rainfall:** The sampling period was marked by unusual patterns of rainfall. Cumulative rainfall deficits across the study area were between 13 and 34 cm below normal in May 1997. Several large storms in September and December 1997 brought large amounts of rainfall to the area, bringing rainfall totals to 2 and 17 cm below normal in September and 27 to 44 cm above normal in December. By February, annual rainfalls were between 54 and 80 cm above normal.

Atmospheric samplers overflowed with weekly rainfall above 9.5 cm. A total of 23 of the 400 weekly samples evidenced rain above 9.0 cm. Notable rainfall events were received during the weeks of September 30 (14.9 cm), December 16 (14.0 cm), and December 30 (11.8 cm), 1997.

**Nutrients:** By site, annualized total nitrogen deposition for the study ranged from 5.03 to 9.66 kg $ha^{-1}$ $yr^{-1}$, while total phosphorus values were between 0.62 and 1.06 kg $ha^{-1}$ $yr^{-1}$ (Table 2). For nitrogen, the higher average weekly loads (as the weekly mean of all sites) received during the summer (Figure 2) should be noted despite the large rainfall amounts received later in the study. Individually, however, only four stations received higher nitrogen loads during the summer

Table 2 Annualized bulk deposition rates, based on a 40 week sampling period. Values in parentheses are computed without outlier values.

| **Site** | **Total N** | **Total P** | **Copper** | **Cadmium** | **Lead** | **Zinc** |
|---|---|---|---|---|---|---|
| | kg $ha^{-1}yr^{-1}$ | kg $ha^{-1}yr^{-1}$ | g $ha^{-1}yr^{-1}$ | g $ha^{-1}yr^{-1}$ | kg $ha^{-1}yr^{-1}$ | $ha^{-1}yr^{-1}$ |
| Alligator Creek | 5.81 | 0.65 | 30.70 | 0.23 | 10.31 | 73.26 |
| Cockroach Bay | 5.03 | 0.80 | 12.64 | 0.34 | 6.50 | 58.31 |
| Cone Ranch | 5.10 | 0.66 | 26.50 | 2.30(0.43) | 7.02 | 261.95 |
| Florida Aquarium | 5.64 | 0.62 | 223.72(87.13) | 0.69 | 29.44 | 634.15 |
| Frog Creek | 5.53 | 1.01 | 6.68 | 0.27 | 4.25 | 71.82 |
| Gandy Bridge | 6.07 | 0.92 | 17.11 | 0.72 | 10.06 | 93.27 |
| Inwood Alum | 9.66 | 1.06 | 17.22 | 0.36 | 18.05(9.77) | 69.21 |
| Pasco WWTP | 6.75 | 0.92 | 6.19 | 2.39(0.40) | 6.46 | 56.85 |
| Pinellas Park | 7.21 | 0.87 | 13.25 | 0.55 | 16.40 | 143.65 |
| Wildlife Preserve | 7.99 | 0.87 | 9.18 | 0.40 | 5.17 | 40.40 |
| Station Mean | 6.48 | 0.84 | 36.3 l(22.66) | 0.83(0.44) | 11.37(10.54) | 150.29 |
| Minimum | 5.03 | 0.62 | 6.68 | 0.23 | 4.25 | 40.40 |
| Maximum | 9.66 | 1.06 | 223.72(87.13) | 2.39(0.72) | 29.44 | 634.15 |

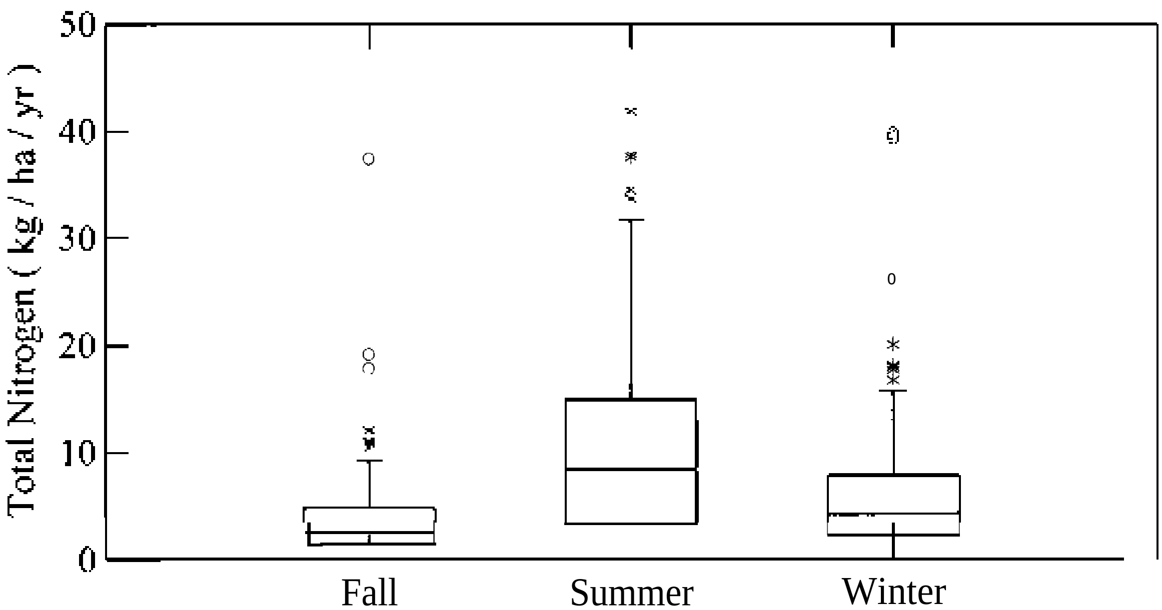


Figure 2. Distribution of weekly total nitrogen loads, measured as bulk atmospheric deposition, by season for all sites combined.

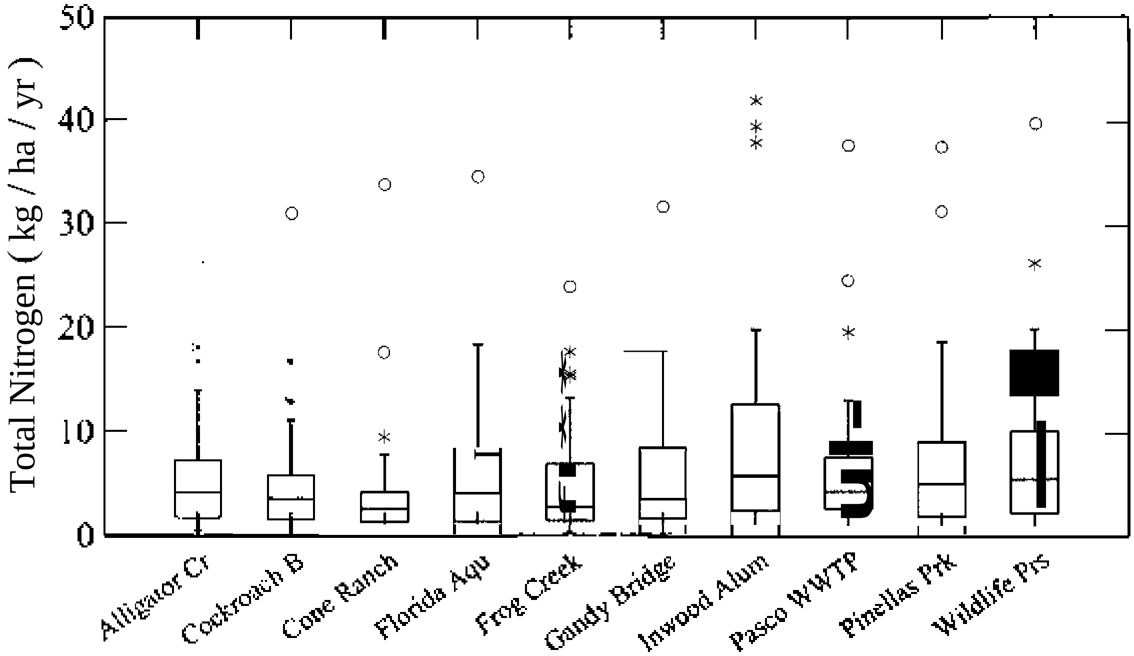


Figure 3. Distribution of weekly bulk deposition of total nitrogen; annualized values.

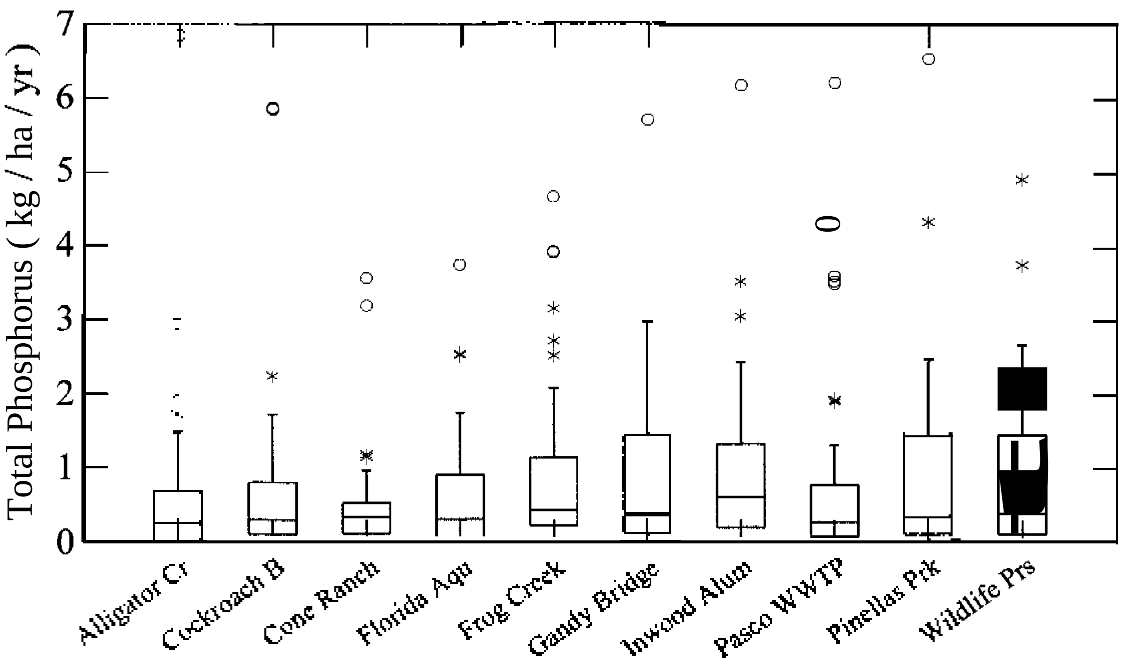


Figure 4. Distribution of weekly bulk deposition of total phosphorus; annualized values.

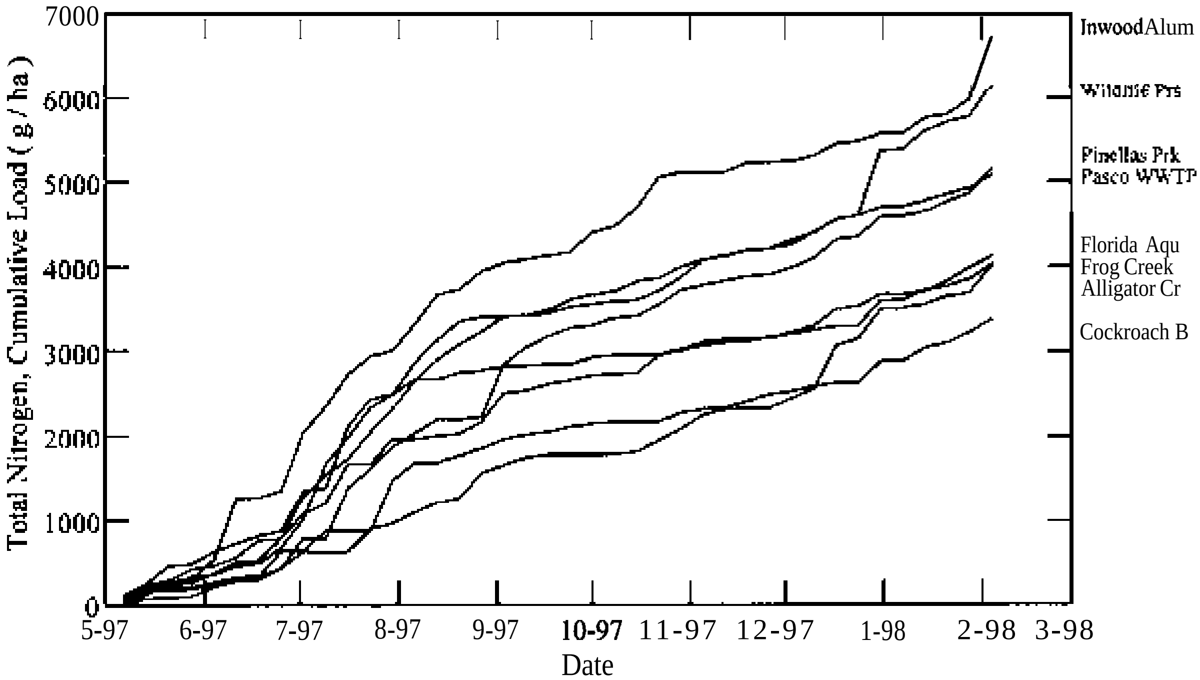


Figure 5. Cumulative atmospheric loadings of total nitrogen (measured as bulk deposition) for stations with greater than 85% completeness of uncontaminated samples.

Metals: Annualized metal deposition by site ranged as follows: copper, 6.19 to 87.13 g $ha^{-1}$ $yr^{-1}$; cadmium, 0.23 to 0.72 g ha-' $yr^{-1}$; lead, 4.25 to 29.44 g $ha^{-1}$ $yr^{-1}$, and zinc, 40.40 to 634 g ha-' $yr^{-1}$ (Table 2). There were several outlier values that were not included above in the annualized totals (cadmium [Cone Ranch and Pasco WWTP], copper [Florida Aquarium], lead [Inwood Alum]), but which have no evidence of contamination. Since atmospheric deposition can be highly episodic, these values were maintained in the database, but means computed in their absence are also presented in Table 2 (values in parentheses). The distribution of monthly composites of metal samples displayed more variation by station than did nutrients (Figures 6-9). Monthly deposition of copper, lead, and zinc were all significantly different among stations. The range among stations for individual metals was substantial, again implying localized influences. The Florida Aquarium site, in particular, was noted for consistently higher loads of Cu, Pb, and Zn.

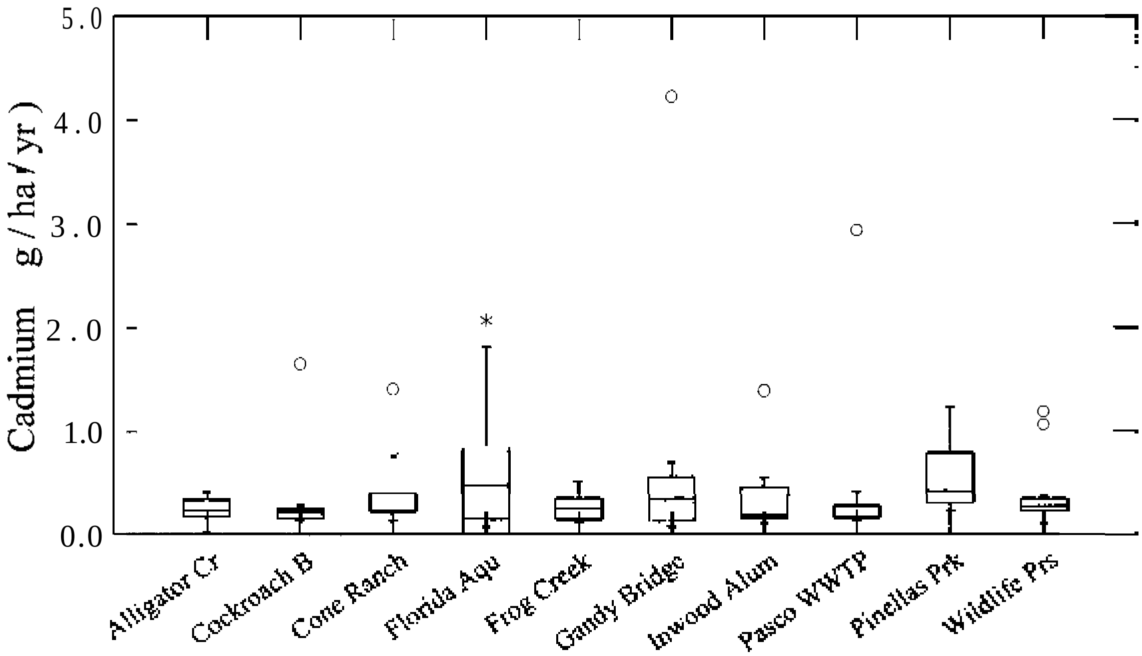


Figure 6. No significant site variations in the distribution of monthly cadmium loads measured in bulk atmospheric deposition.

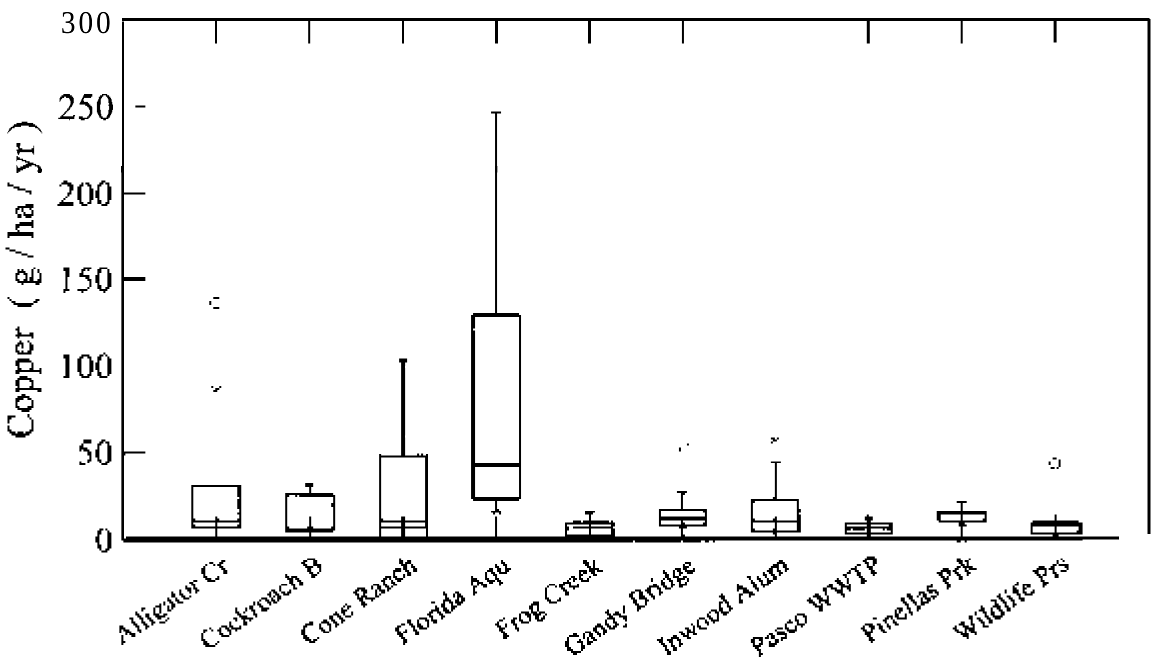


Figure 7. Significant site variations in the distribution of monthly copper loads measured in bulk atmospheric deposition.

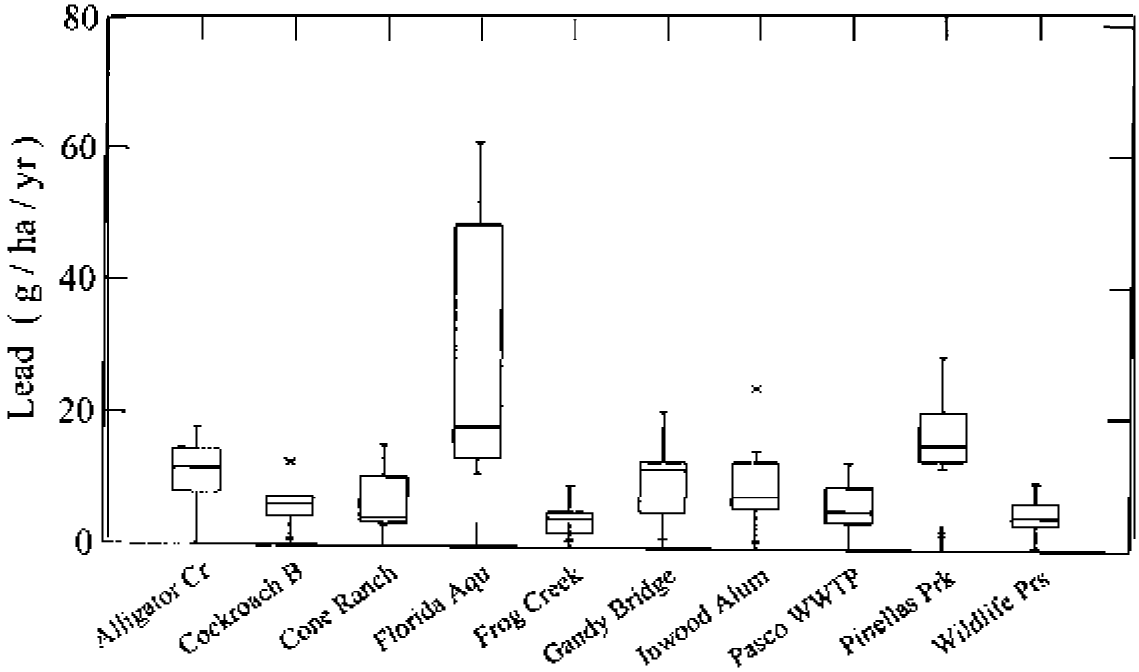


Figure 8. Significant site variations in the distribution of monthly lead loads measured in bulk atmospheric deposition.

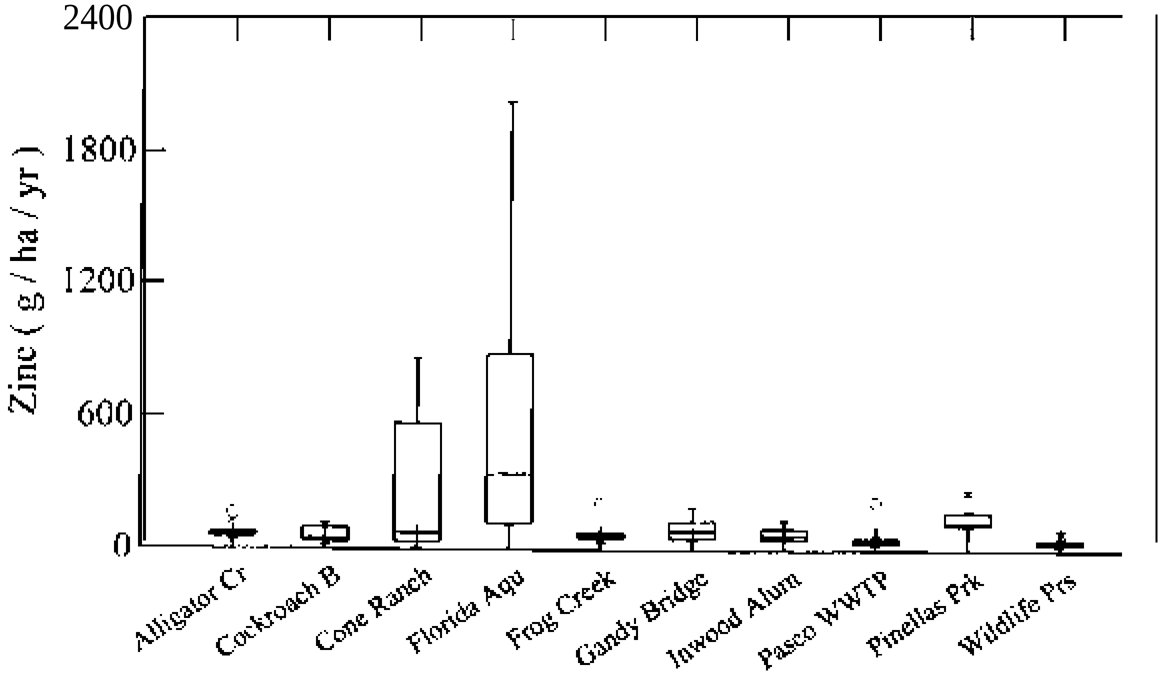


Figure 9. Significant site variations in the distribution of monthly zinc loads measured in bulk atmospheric deposition.

## Comparison to Wet-only And Dry Deposition Sites

**Gandy Bridge:** The Gandy Bridge intensive site was located on a 7.4 km causeway/bridge spanning Old Tampa Bay as part of the Tampa Bay Atmospheric Deposition Study. Daily wet-only and weekly dry deposition samples were collected. Available concurrent data were limited to May, June, July, and August and were summed to weekly totals for comparison with rainfall and atmospheric loadings measured as bulk deposition. Rainfall amounts measured by the two collectors were significantly related and the slope of the relationship indicated the bulk collector captured 90% of the rainfall measured by the intensive site. The relationship of the weekly loads of bulk deposition of total nitrogen to both wet-only and to wet plus dry deposition of inorganic nitrogen were statistically significant and support the utility of the bulk deposition technique for determining nitrogen loading (Figure 10).

**Florida Aquarium:** The Southwest Florida Water Management District (SWFWMD) established an intensive rainfall (wet-only) and stormwater monitoring site in highly urbanized downtown Tampa. The individual event data from the site were summed for weekly loadings to compare with bulk atmospheric deposition, Elimination of the five weeks when the bulk collector overflowed resulted in a significant correlation between bulk loadings and rainfall loadings of nitrogen (Figure 11) and phosphorus, The increased scatter in the nitrogen data (compared to the Gandy site) may be due to a higher and more variable proportion of dry deposition at the highly urbanized site. Table 3 presents the results from both installations as total loads received over the 35 weeks where no overflow occurred. Wet-only deposition consisted of 55%, 30%, 57%, and 42% of the respective phosphorus, copper, lead, and zinc loads collected in the bulk or total deposition (Rushton, 1997).

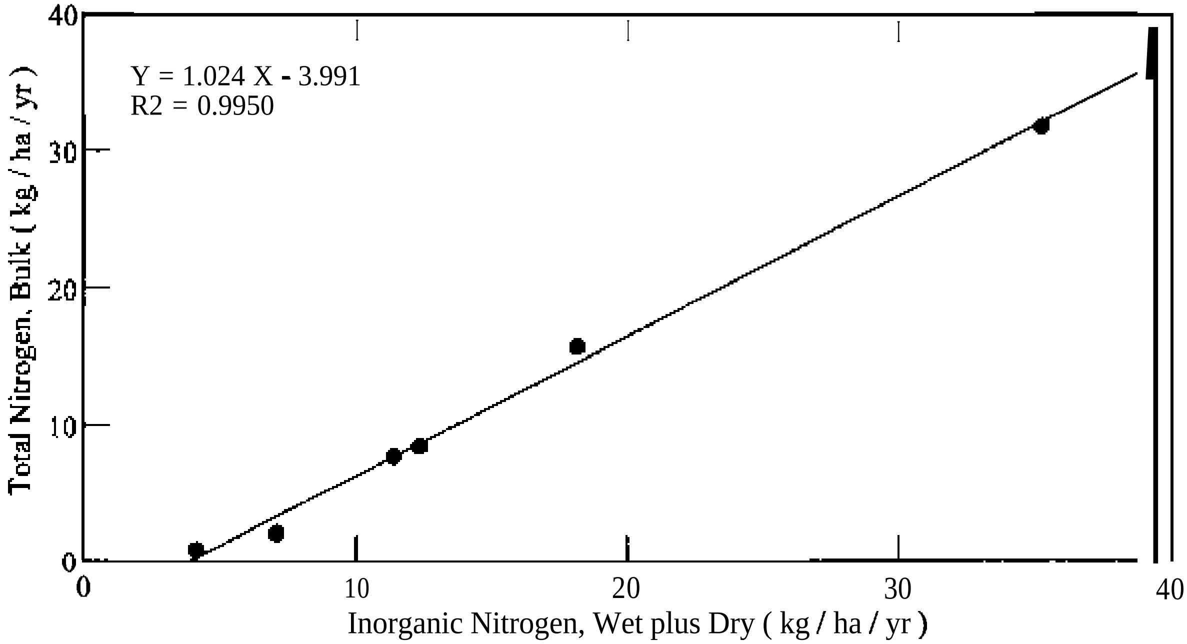


Figure 10. Relationship of bulk deposition of total nitrogen (inorganic plus organic) to wet plus dry deposition of inorganic nitrogen (May through mid-July 1997) at the TBADS intensive monitoring site,

Table 3. Comparison of bulk and wet-only deposition loads received at the Florida Aquarium site when rainfall amounts did not exceed 9.5 cm; n=35 weeks.

| **Parameter** | **Units** | **Bulk** | **Rainfall** |
|---|---|---|---|
| Rainfall | cm | **48.9** | 67.4 |
| Total Nitrogen | g $ha^{-1}$ | 3013. | 3088, |
| Total Phosphorus | g $ha^{-1}$ | 284. | 157. |
| Cadmium | g ha“ | 0.3 | 0.8 |
| Copper | g $ha^{-1}$ | 80.5 | 24.1 |
| Lead | g $ha^{-1}$ | 10.2 | 5.8 |
| Zinc | g $ha^{-1}$ | 229.2 | 96.9 |

## REFERENCES

Dixon, L.K., M.G. Heyl and S. Murray. 1998. Interpretation of bulk atmospheric deposition and stormwater quality data in the Tampa Bay region. Submitted to the Tampa Bay Regional Planning Council and Tampa Bay Estuary Program. Mote Marine Laboratory Technical Report #602.

Rushton, B. 1997. Processes that affect stormwater pollution. Proceedings of $5^{th}$ Biennial Stormwater Research Conference, Nov 5-7, 1997, Tampa, Florida, Southwest Florida Water Management District. p. 43-53,

# GRASS AND LEAF DECOMPOSITION AND NUTRIENT RELEASE STUDY UNDER WET CONDITIONS

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## ABSTRACT

A significant source of nutrient input to water bodies is from grass clippings and leaves (yard debris) washed into drainage systems during storms. Brevard County Surface Water Improvement conducted a study to determine the nutrient release rates from grass clippings and leaves in order to better understand the chemistry and resultant pollutant loading mechanisms.

Sixty-gram samples of mixed freshly cut St. Augustine yard grass (*Stenolaphrum secundalum*) and oak leaves (*Quercus sp.)* were placed into opaque containers. Coarsely filtered storm/ditch water was added to fill the containers to the S-liter marks. Samples were allowed to go anaerobic, typical of wet sump best management practice (BMP) structure conditions, and tested periodically after soaking and processing. At intervals of: 0, 1, 5, 9, 14, 22, 34, 50, 70, 130, and 180 days, triplicate bucket sets were agitated to simulate mixing from stormwater influx, then poured through a Number 35, US Standard Soil Sieve, and the liquid analyzed. The solids that remained in or on the sieve were analyzed, and the results compared to those of the corresponding liquid phase.

The results presented depict "typical" east-central Florida lawn and leaf litter decomposition and nutrient release rates. This information may be useful in the selection or site design of BMP's for treating nutrients in stormwater, and determining cleaning frequency.

## INTRODUCTION

Sediment carried by stormwater may reduce the ability of light to penetrate water thereby hindering the growth of marine plants; also possibly covering and smothering the plants, resulting in a die off. Leaves, grass clippings and organic matter from yards increase oxygen demands and may contribute nutrients to algae blooms that may result in fish kills. Brevard County has taken a pro-active stance to reduce sediment and nutrient contributions whenever possible through retrofitting areas that currently have little or no stormwater treatment provided. Several treatment methods currently utilized by the County include baffle boxes and stormwater inlet devices that retain these materials before they enter surface waters.

Baffle boxes often receive constant groundwater flows and retain standing water in the chambers where the sediment and debris are collected. The question has been posed whether organic constituents may leach out of the collected materials only to be carried to surface waters during the next storm event or by background flows. A significant source of nutrient input to water bodies is from grass clippings and leaves washed into drainage systems during storms. Brevard County Surface Water Improvement conducted this study to determine the nutrient release rates from grass clippings and leaves in order to better understand the chemistry and resultant pollutant loading mechanisms. The goal of this experiment was to identify variations in the concentrations of constituents, with an ultimate goal of determining a timetable for cleanout of applicable BMP structures to prevent the release of targeted pollutants.

## MATERIALS AND METHODS

As a result of visual inspections ofnumerous baffle boxes it was determined that the organic yard waste they collect is typically a mixture of grass clippings and leaf litter. This study therefore was conducted on grass clippings collected from a yard containing oak trees, and included between 3 1% to 66% of oak leaf litter by weight, This yard had never been fertilized or serviced by a sprinkler system (Figure 1).

Water was collected from two storm water conveyance canals. Cleaned opaque sample buckets and lids were rinsed with the filtered water prior to final filling with 8 liters of 180 micron (sieve opening or pore size) filtered water. Previously refrigerated, week old, sixty-gram samples of mixed St. Augustine yard grass (*Stenolaphrum secundalum*) and oak leaves (*Quercus sp.)* were placed into the containers, and mixed to wet the grass. This was to simulate rainfall washing grass clippings into a retaining BMP sump. The tops of the buckets were loosely fit to allow off-gassing but minimize evaporation. The buckets were allowed to remain undisturbed in a non-climate controlled storage area; subjected to indirect light, and temperature swings between 25 and 37 degrees C. Samples were allowed to go anaerobic, typical of wet sump BMP structure conditions, and tested periodically after soaking and processing.

At intervals of: 0, 1, 5, 9, 14, 22, 34, 50, 70, 130, and 180 days, triplicate bucket sets were selected by blind lottery, agitated to simulate mixing from stormwater influx, then sent to the contract laboratory for processing. The sample was poured through a #35, US Standard Soil Sieve. This sieve has a pore opening of 500 microns (0.0197 in). The solids that remained in or on the sieve were analyzed for weight at apparent external dryness, total kjeldahl nitrogen (TKN), biochemical oxygen demand at 5 days (BOD-5), and total phosphorous as P (TP as P). The liquids that passed through the filter were analyzed for: color, BOD-5 Day, TKN, and TP as P. A select group of constituent pollutants is discussed here, a more comprehensive list of analytes are discussed at length in the full report.

The residue (solid phase) was weighed, percentage moisture determined, then the residue analyzed. The mass of the mixed grass and oak leaves residue dropped from 28 grams to 16 grams within the first 15 days of saturation, a loss of 43%. The samples then stabilized at weights between 13 and 20 grams for the duration of the 180-day sampling period. No attempt was made to differentiate between percent decomposition of grass to oak leaf ratios, but observations made on the mixture throughout the study revealed almost total solution of the grass, with little obvious physical decomposition evident of the oak leaves, even out to the 180 day mark.

In order to allow a more straightforward comparison of concentrations of constituents that leached into the water to the constituent concentrations remaining in the solids, mg/L was correlated to the average dry weight of the grass that was placed in the container for leaching. (28 g grass/8 liters liquid = 3.5 g grass/one liter liquid, producing = x mg/L = x mg/3.5 g grass) A ratio was then applied to determine the constituent level that would have leached from 1000 g (1 kg) of dry solids. This allows direct comparison of mg/kg solids concentration to mg/kg leachate.

Six replicate samples of the raw grass and oak leaves were dried to constant weight in a desiccator. The samples averaged a loss of approximately 54% of their weight after the first day of desiccation. The average dried weight of these initial samples was 28 grams, with virtually all of the grass being retained on a U.S. Standard #35 sieve. The numbers returned upon analysis for chemical and physical characteristics varied with each sample. This was expected due to variation within the small (triplicate) sample group; particularly when considering the physical characteristics of the grass blades, grass stems and nodes, oak leaves, and oak stems. Virtually all of the weight lost during desiccation was due to water loss from the grass, as the oak leaves had dried out prior to falling. Most of the water released by the oak leaves had been gained through compaction and mixing with the freshly cut grass in the lawn mower grass catch bag. Sample weights for the 6 initial samples used to determine representative yard grass-oak leaf ratios actually decreased during the hour it took to sort out individual grass and oak leave fragments to determine component ratios.

## RESULTS

### Wet Weight/Dry Weipht Ratios

The weights of the grass and oak leaf samples at initial weighing before immersion were approximately 60 grams. The initial dried weight of these initial samples averaged 28 grams, indicating initial moisture content of 58%. By the end of day 1, the moisture content of the wet samples was up to approximately 85%, indicating that some absorption of water had taken place. Also after one day of immersion and subsequent drying, the grass and leaf mixtures weighed an average of 22 grams. By the end of day 5, the dried weight values averaged 18 grams, a total loss of 10 grams (36%) from the original dry weight. No further change in the moisture content was discerned throughout the 180 days of the experiment. Even though the total weight of the solids reduced by 7 1% over the course of the study, the ratio of 85% moisture remained constant. These values, combined with laboratory observations, indicate that the majority of the residual components after day 5 may be relatively inert oak leaves (Figure 2).

### Total Kieldahl Nitrogen

Initial values for TKN concentrations for the liquid phase raw mixed oak leaf and grass samples averaged 3.4 g/kg. After immersion for one day, the TKN concentrations rose 3 1%, to 4.9 g/kg (this corresponded with a loss of TKN from the solid phase of only 11%). The total kjeldahl nitrogen concentrations of the liquid portion of each sample fell steadily from that point, to stabilize around

1.4 g/kg by day 50. Approximately 70% of the total loss of TKN from the liquid phase sample took place by the day 50 sample. At the day 180 sample, the liquid TKN concentrations exhibited a slight increase from the day 130 values. This may have been the result of eventual decomposition of the oak leaves, but the study was halted at day 180 and a definite trend past that point could not be substantiated (Figure 3).

The concentrations depicted a quick leaching of the TKN fractions. The solid phase TKN concentrations began with an initial cut grass and leaf average value of 19.0 g/kg. This value dropped to 17.0 g/kg after one day of immersion; a loss of 11% in a single day. After the second day, concentrations remained somewhat stable until after day 22, whereupon they slowly began to rise. A correlating fluctuation was not observed in the liquid phase. The values for samples analyzed on day 34 averaged approximately 26.0 g/kg, an increase of some 28% over the day 22 values. The solid phase TKN values fell from their peak at day 34 to their lowest points during the study by the day 130 samples. Initially, this reduction corresponds to subsequent peaks in the nitrite and nitrate components of the liquid phase; and indicates ammonia, or some other unmeasured nitrogen fraction, was being released into the water by the solid mass. As TKN analysis can include the ammonia but not nitrite/nitrate Ii-actions of nitrogen, it appears that this was a period of rapid decomposition of the nitrogenous compounds present in the mixed grass and oak leaves. There was a sharp rise in both the liquid and solid fractions of TKN after the day 130 sample; suggesting breakdown of the much tougher oak leaf litter portions of the samples.

**Total Phosahorous as Phosphorous (P)**

Perhaps the most dramatic illustration of the effects of leaching on mixed lawn grass and oak leaves was observed in the variations in total phosphorous as phosphorous (P) concentrations. A kilogram of raw mixed grass and oak leaf solids yielded 1.9 grams of total phorphorous as P, prior to wetting. Analysis of the liquid phase for total phosphorous as P revealed an average raw water composition of approximately 125 mgikg of total phosphorous as P. When the grass and leaves were added, there was an 89% increase in the liquid phase total phosphorous as P concentrations (to 1,057 mg/kg) within the first day. By day 4, the values in the liquid phase had stabilized around 1,000 mg/kg; remaining there for the course of the study.

The solid phase of the total phosphorous as P analysis depicted a very rapid leaching of phosphorous from an initial fresh grass and leaf value of 1,900 mg/kg; to a value of 880 mg/kg after the first day (a reduction of 54%). There is evidence that this leached phosphorous made its way into the water column and increased the liquid phase total phosphorous as P concentrations significantly over the first day, and increased them slightly over the next 22 days. Simply put, for the first week, when the solid phase phosphorous concentrations went down, the liquid phase values went up. However, after day 22 both the liquid and solid phase values and ratios between the respective values fluctuated. This may have been due to phosphorous changing state between solid and liquid phases (Figure 4).

**Biochemical Oxygen Demand 5-Day (BOD-5)**

A kilogram of raw mixed grass and oak leaf solids yielded 21.3 grams of BOD, prior to wettingThe liquid phase BOD-5 values immediately rose sharply (700%) from an initial demand of approximately 4.5 g/kg to peak at 40.0 g/kg by day 9. This corresponds to the peak in biological activity for the decomposition process. The demand then fell just as rapidly, to stabilize by day 22 at concentrations between 2.5 and 4.0 mg/kg; which it maintained throughout the duration of the experiment. This can be thought of as the bloom and die off phases of aerobic bacteria and other organisms present and active in the liquid phase. Basically, virtually all those nutrients readily available to aerobic organisms were used up within the first 22 days of the study. As the chemistry of the static containers moved from aerobic to anaerobic situations, there was a progression of biological and chemical reactions that occurred to take advantage of the conditions present.

The biochemical oxygen demand values for the solid fractions illustrated a progression from an initial value of 21.5 g/kg for the raw grass, which fell 19% in the first day to 17.3 mg/kg. By day 5, the BOD-5 values of the grass and oak leaf solids had fallen by a total of 25% from their initial concentrations. From day 5 on, the BOD values began to rise again, reaching a maximum value of 33.0 g/kg by day 34. This corresponds to the maximum decomposition rate of the solid phase grass and oak leaf components. By the time day 70 arrived, the samples were essentially biologically "dead," with a steady solid and liquid phase BOD-5. Some minor biological activity was still taking place (nature abhors a vacuum), but relatively little more biological breakdown could be expected under continuation of the existing environmental conditions (Figure 5).

**Color**

The color of the water prior to mixing in the mixed leaves and grass was 140 PCU. After one day of soaking with the solids, the color then measured 193 PCU in the liquid phase, an increase of 38% percent. By day 22, the color levels had stabilized at 350 PCU, an increase of 150% percent over the original background water. Color is coming under increasing scrutiny as one of the major attenuating agents of sunlight reaching seagrasses and other submerged aquatic plants. In conjunction, color is one of the most expensive pollutant components of surface water to remove. In light of this, it would seem prudent that wet-detention/retention BMPs be cleaned as soon as possible after wash-down of yard debris entering the catch basin. There appears to be real value gained in doing so, up to 22 days after the grass and leaves being submerged (Figure 6).

## DISCUSSION AND CONCLUSION

The results of this study show that the majority of organic-based pollutants, which leach from grass clippings and leaves into water, will be released within 1 to 22 days, depending on the pollutant. For example, the BOD 5 Day concentrations peaked at 9 days; color was continuously released between 1 and 22 days; and most of the phosphorous was released within the first day of grass immersion. Based on these preliminary results it appears that in order to avoid significant leaching of most “pollutants”, it is desirable to quickly remove organic debris from collection devices that retain water. It would be best to design yard debris trap basins which retain the solids in a dry area, rather than dealing with the engineering and economic hardships of removing these released pollutants from the stormwater stream. Even traditional wet detention ponds or wetlands would benefit from upstream, dry, inlet devices to reduce the pollutant loadings by removing them as solids, rather than dealing with the leachate in the liquid form in the ponds.

Since particular pollutant concentrations peaked at different times, by matching the clean out schedule to the pollutant it is conceivable one may be able to selectively remove a particular pollutant fraction. If these devices are not regularly cleaned quickly, and there is background flow or a storm event, one would have to conclude that a large percentage of the organic matter previously collected is being released and the component pollutants are actively flushed out to the surface waters.

In a prior grass monoculture screening study, significant gas production (hydrogen sulfide, methane) was evident after about 7 days of soaking. This fermentation was not observed in the current mixed grass and oak leaf study, possibly due to a different bacterial flora. The water source for the prior study was primarily groundwater flowing into a baffle box, whereas the water source for the mixed grass and oak leaf study was surface and stormwater. It may be that the bacteria predominating in the groundwater for the prior study were anaerobic and better able to take advantage of the conditions in the sample containers than the aerobic bacteria thought to initially predominate in the present study. The concentrations and trends observed in the prior study were far different (greater concentrations, for longer periods) from those seen in the present study.

## RECOMMENDATIONS

The sample volumes and weights used in the present study were selected to be representative of conditions observed in “typical” stormwater treatment BMPs. The concentrations of the “pollutant” constituents being analyzed during this study were at times very near the minimum detection limits for the contract laboratory. As such, identification of statistically significant trends or day-point values is very difficult at the lower levels. It may be necessary to re-run this experiment with a greater volume of wet grass and leaves in order to more clearly quantify the possible contribution rates to the overlying water. Because of the great differences observed in the concentrations of the pollutants and their respective cycles between the prior and present study, it would be of value to run this study again with different grass types, or mixtures of grass and leaves.

A future study planned by Brevard County Surface Water Improvement will be to initially wet the yard debris to simulate stormwater wash down, then dry the yard debris for varying periods. The results of these analyses will be compared to values obtained from fresh, raw yard debris.

| Trial # | Raw Grass mass (gms) | Raw Oak Leaf mass (gms) | Percent Raw Grass by weight | Raw Grass Clipping % by visual estimate |
|---|---|---|---|---|
| 1 | 30.7 | 30.7 | 50% | 67% |
| 2 | 31.2 | 25 | 56% | 48% |
| 3 | 39.7 | 17.7 | 69% | 63% |
| 4 | 35.89 | 17.24 | 68% | 63% |
| 5 | 17.74 | 34.64 | 34% | 43% |
| 6 | 22.43 | 24.73 | 48% | 52% |

Figure 1

**Grass Clipping Wet Decomposition**
**Mixed Leaves and Grass**
**Dry Weight to Wet Weight Percentages**

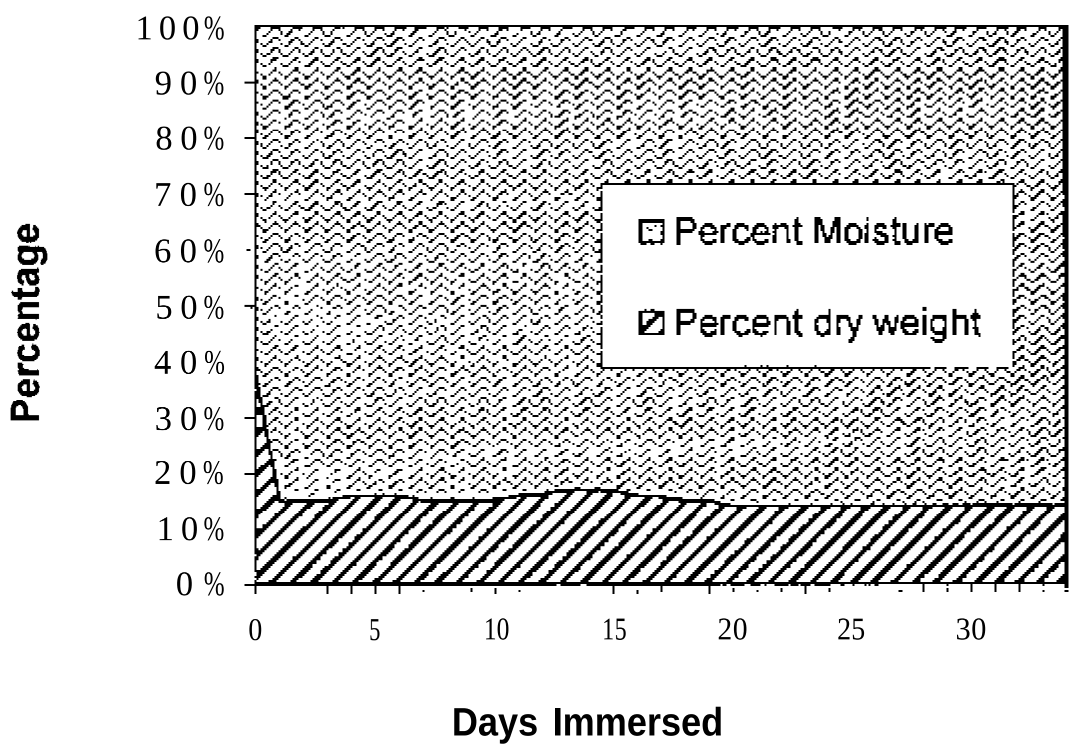


Figure 2

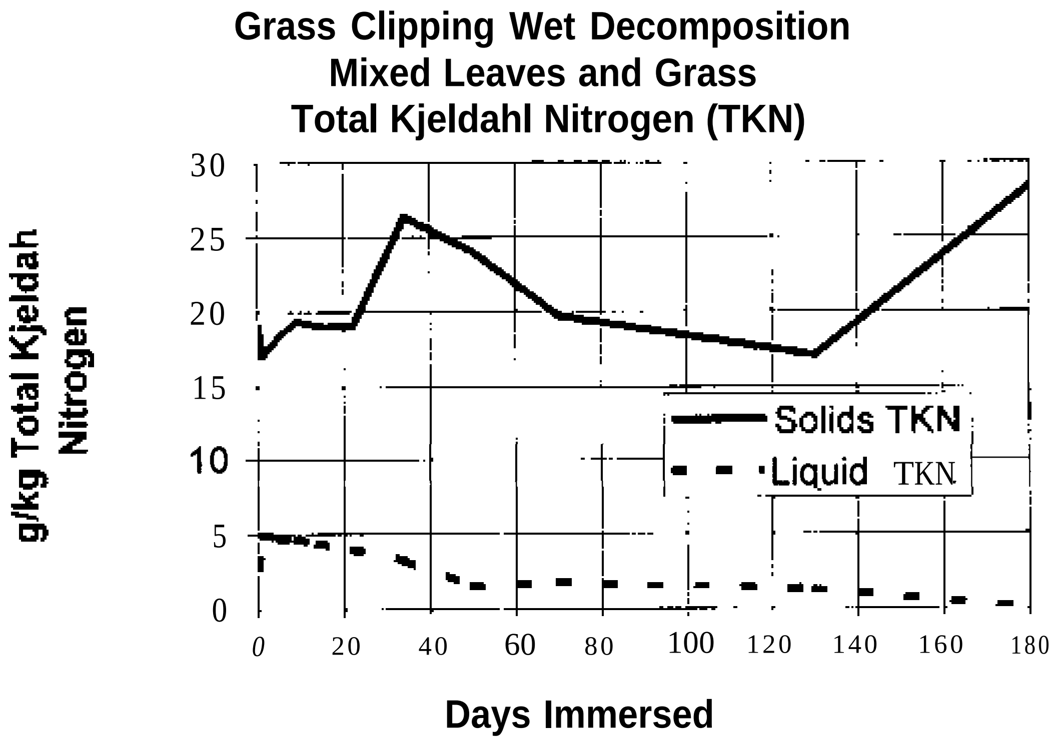


Figure 3

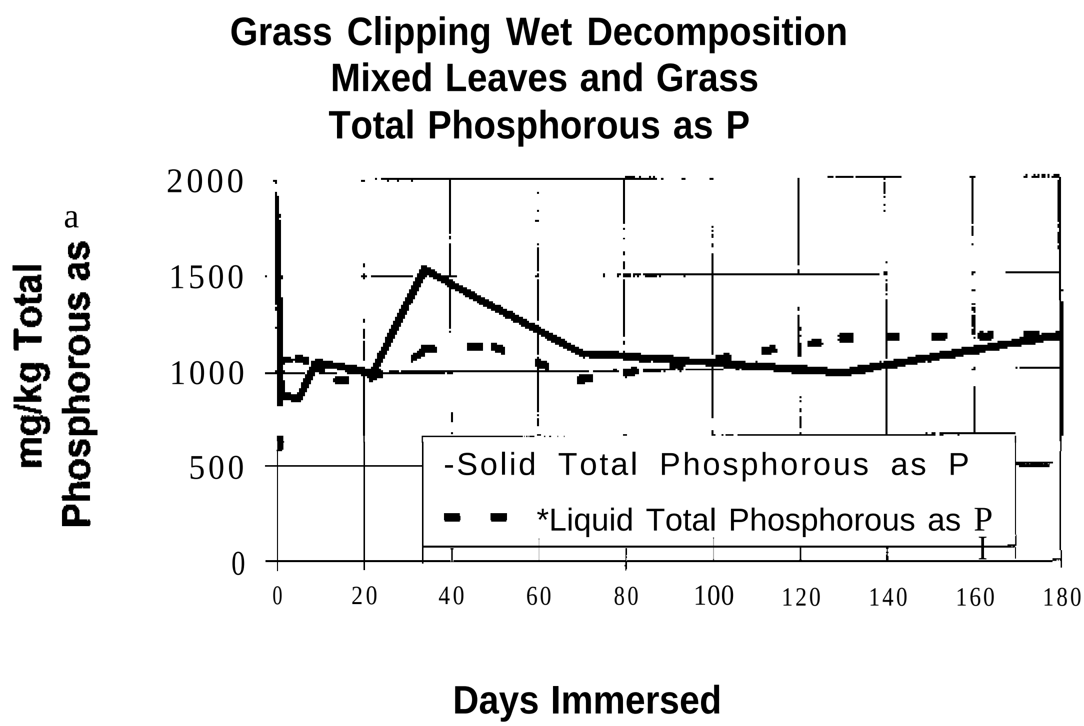


Figure 49

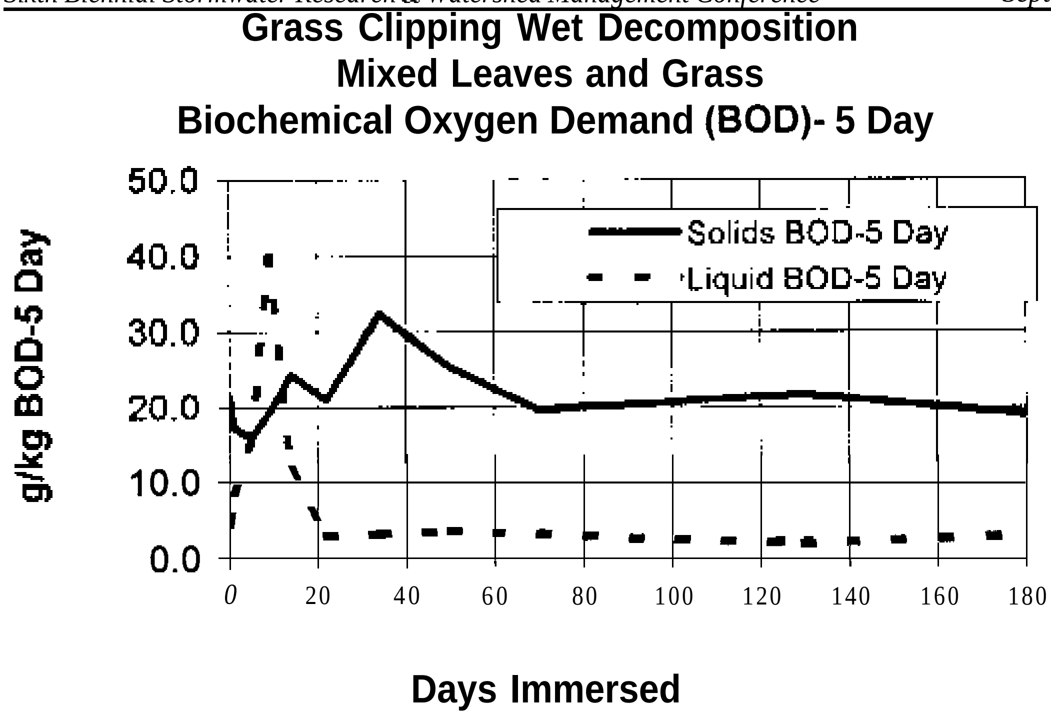


Figure 5

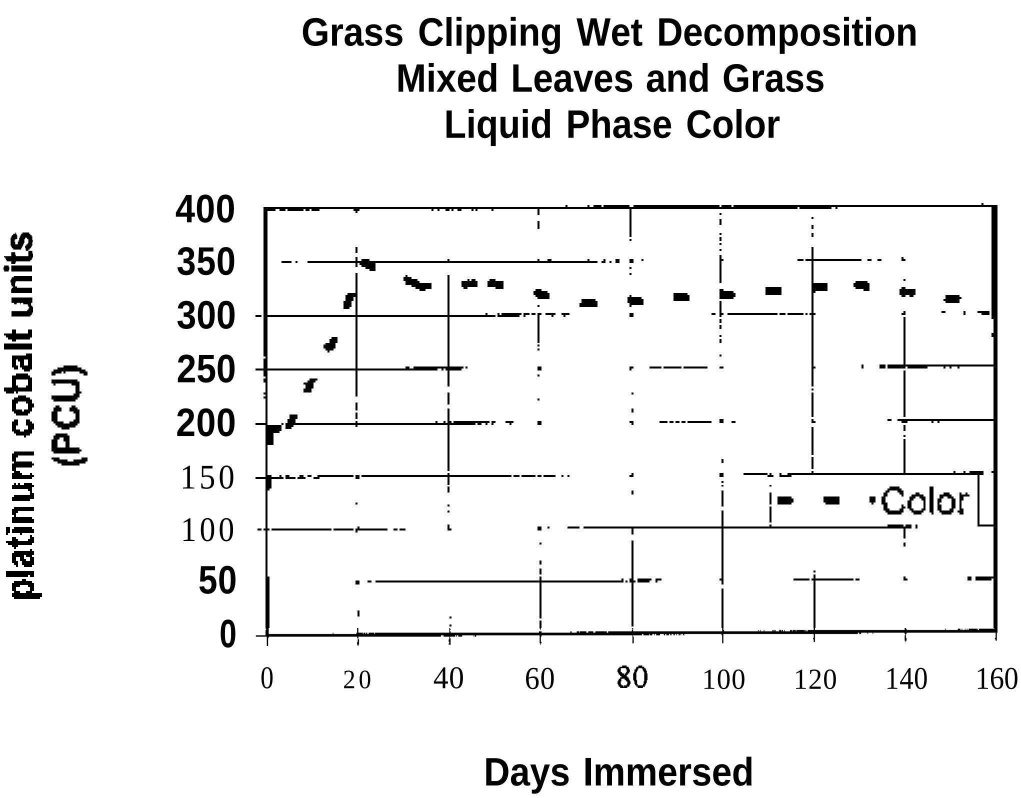


Figure 6

**EVALUATION OF STORMWATER RUNOFF QUALITY AT FLORIDA TRANSIT MAINTENANCE FACILITIES**

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## ABSTRACT

An evaluation of public transit maintenance and storage facilities in Florida was performed to determine the stormwater quality of runoff from these facilities and to evaluate the validity of the EPA's assumption of stormwater runoff pollution problems. The characteristics of the facilities were investigated to evaluate the potential for stormwater runoff pollution. These characteristics included maintenance performed at the facilities, materials used, and materials stored on-site at the facilities. It was determined that these characteristics, specifically activities such as vehicle repair, vehicle painting, vehicle washing, vehicle fueling, and storage of materials such as fuel, oils, lubricants, grease, and solvents, provide a large potential for stormwater runoff pollution. Analysis of stormwater runoff quality data from four facilities in Florida confirmed that stormwater runoffpollution problems do exist at these facilities, including BOD, COD, TSS, TP, Nitrate I- Nitrite, Fecal Coliform, Surfactants, Lead, Zinc, and Total Phenolics. The data was used to determine which Best Management Practices (BMPs) would potentially increase the quality of stormwater runoff at these facilities. Eighteen applicable BMPs were identified for transit maintenance and storage facilities to improve stormwater runoff quality.

## INTRODUCTION

Whereas much research has been done on the transit aspects of the public transportation industry, little research has been done on the effects of these trans-portation systems on water quality. This paper focuses on the stormwater runoff pollution from public transit system facilities. Transit vehicles pollute stormwater runoff in a number of ways. As these vehicles travel over roads and highways, they deposit oil, grease, heavy metals, and dust and dirt particles. However, a large majority of stormwater pollutants resulting from public transit vehicles are concentrated at the transit maintenance and storage facilities where the vehicles are stored, fueled, washed, painted, and maintained.

The federal focus on stormwater runoff has made it necessary for almost all industries, including the mass transit industry, to consider how their activities affect stormwater runoff quality. Public transit maintenance and storage facilities store the transit vehicles on-site and perform many different types of maintenance on a daily basis, from oil changes, brake repairs, and engine repairs to vehicle washing and painting. These practices use a large amount of chemicals, greases, and solvents, providing many potential sources of pollution.

Nineteen of the largest public transit systems in Florida were considered for this project. These facilities were located in Broward County, East Volusia County, Escambia County, Hillsborough, Jacksonville, Key West, Lakeland, Lee County, Orlando, Manatee County, Dade County, Palm Beach County, Pinellas County, Gainesville, Sarasota County, Smyrna, Brevard County, and Tallahassee. These facilities were evaluated by reviewing their compliance with federal, state, and local stormwater regulations, analyzing typical stormwater runoff quality results at these facilities, and determining current stormwater management practices used by these facilities. The composition and characteristics of the stormwater runoff from representative facilities were analyzed and the origin of these pollutants were estimated by examining the maintenance practices and materials used at the facility. Once the problem areas at the facility were isolated, best management practices (BMPs) applicable to all transit maintenance and storage facilities in Florida were developed.

The lack of research on runoff at transit facilities made it necessary to compile information about the pollution problems of industries with similar maintenance activities. Several EPA manuals assist with the development of pollution prevention plans. For example, *Storm Water Management for Industrial Activities: Developing Pollution Prevention Plans and Best Management Practices* introduces very generalized methods for developing pollution prevention plans and explains some of the typical most used best management practices. However, the suggestions are very generalized and do not contain specific material necessary to transit maintenance and storage facilities. Interviews with employees from transit maintenance and storage facilities, the Florida Department of Environmental Protection (DEP), the Florida Department of Transportation, and the City of Tallahassee Stormwater Division offered valuable information on the needs of the transit industry, the extent of stormwater quality problems, the availability of information, and the legislative requirements at all levels of government. Most of the specific details on the transit facilities and the informational needs of the facilities were obtained from questionnaires sent to each facility. A survey was sent to each transit facility in Florida to obtain information about their characteristics, stormwater management practices, and current legislative compliance. This information was supplemented with follow-up phone calls to the facilities. Overall, this paper was an exercise in compiling available information from many different sources and modifying it to apply to the needs of transit maintenance and storage facilities in the State of Florida.

## Characteristics of Transit Facilities in Florida

Public transit maintenance and storage facilities typically store transit vehicles on-site and perform many different types of maintenance on the transit vehicles on a daily basis. The size and layout of these facilities, as well as the types of maintenance performed on-site and the materials stored at these facilities provide insight to the characteristics of the facilities and help to determine their potential to contribute to stormwater runoff pollution.

There are 19 major bus and/or rail public transportation systems located throughout the state. Most of these systems are fixed-route motor bus systems, but some facilities offer other options of public transportation. Though most of the transit facilities are fixed- route motor bus systems, thirteen of the nineteen facilities also offer some type of demand response system.

Each of these transportation systems has one or more transit maintenance and storage facility that generally perform similar operations. The characteristics of these facilities have been separated into four categories: service characteristics, physical characteristics, maintenance characteristics, and stormwater management characteristics.

The service characteristics of each facility include the operating statistics of the facility (number of trips, vehicle hours, and vehicle miles) and the vehicle information (total vehicles owned, vehicles operated, and average age of the fleet). These service characteristics show the actual operation size of each facility investigated. Information such as passenger trips, vehicle miles, and vehicle hours gives an idea of the demand on the system. As the demand increases and more the vehicles are used, more maintenance is required which ultimately may lead to more pollution. Generally, the older the fleet, the more maintenance required. The demand on the system and the amount of maintenance performed at a facility may also have a large impact on quantity of stormwater pollution. The largest facility, by far, is the Metro-Dade Transit

Agency, while the smallest facility is the Space Coast Area Transit Agency in Brevard County. These two facilities are very extreme in size compared to the other facilities in Florida.

Almost all facilities have the following operations performed: vehicle repair, painting and washing; tire and brake repair; fueling and fuel storage; chemical storage; and waste oil storage. Bulk material storage is done at less than half of the facilities.

The stormwater management characteristics include isolated fuel and chemical storage areas, retention and detention ponds, oil/grease skimmers, trash racks, and other preventive measures. The other stormwater management characteristics listed include an inventory of any permits granted, stormwater management plans in effect, or environmental audits conducted at the facility. All the facilities had separated/isolated fuel storage areas, and all except four have oil/grease skimmers. About half have separated chemical areas, and only a few facilities have other stormwater facilities.

Most of the facilities have fuel areas that are separated from the rest of the facility. This separation restricts the stormwater runoff from this area from mixing with the stormwater from the other areas of the facility. Most of the facilities that have chemical storage areas also have these areas separated. Most of the facilities have some type of stormwater facility to detain or treat stormwater runoff before it exits their property. Four facilities have detention areas, seven facilities have retention areas, one facility has a wetland area, four of the facilities have trash racks, and most of the facilities have oil/grease skimmers. Three facilities retain all stormwater runoff within their property.

**Stormwater Quality of Florida Transit Facilities**

Four of the nineteen transit maintenance and storage facilities have had their stormwater tested. Some of these facilities have had the actual runoff tested during a rain event, while others have had stormwater from their retention/detention pond tested. These stormwater quality results were obtained from the transit facilities and from public documents filed with the State of Florida. Three different types of stormwater samples were taken: pond water samples, grab samples, and composite samples. Pond water samples consist of samples of water taken directly out of the retention/detention pond. Grab samples consist of stormwater samples taken during a storm event. These samples are usually taken during the first flush conditions, which usually occur within the first thirty minutes of the storm event, and usually consist of the most polluted portion of the stormwater runoff. Composite samples consist of samples taken throughout the duration of the storm. These samples are flow-weighted, meaning that both the amount of water collected and the flow of the water at that time period are recorded. These samples are combined in proportion to the flowrate of the water to come up with one sample that is tested. The differences in the type of samples collected must be considered in the interpretation and comparison of the stormwater quality results.

The facilities tested, listed in decreasing size, are the Metro-Dade Transit Authority (MDTA), Pinllas Suncoast Transit Authority (PSTA), Palm Beach County Tranportation Authority (PBTA) and the Lee County Transit Authority (LCTA). By comparing the characteristics of the facilities, it is obvious that the stormwater quality results obtained were for larger facilities. However, these results are adequate for this study and to give typical stormwater quality values for stormwater runoff from transit maintenance and storage facilities.

| **Chemical/** Pollutant | **Table 1** AVERAGE **STORMWATER** TESTING RESULTS (Facility Tested and Sample Type) | | | | | | |
|---|---|---|---|---|---|---|---|
| | Units | PSTA **Retent** Pond | LCTA Grab **Sample** | LCTA Comp **Sample** | PBTA Grab Sample | PBTA **Comp.** Sample | MDTA Grab **Sample** |
| Oil & Grease | mg/l | 0.51 | 0.91 | -- | 0.06 | -- | 7.35 |
| pH | ----- | 7.38 | 8.08 | -- | 7.67 | – | 7.30 |
| BOD | mg/l | 5 | 1 | 3.5 | 24 | 3.3 | 8.42 |
| COD | mg/l | 31 | 10.5 | 20 | 81 | 34 | 135.4 |
| TSS | mg/l | 11 | 14.5 | 19.5 | 50 | 20 | -- |
| TP | mg/l | 0.18 | 0.66 | 0.44 | 0.27 | 0.14 | 0.10 |
| TKN | mg/l | 2.76 | 0.08 | 0.20 | 0.83 | 0.43 | 1.51 |
| Nitrate + Nitrite | mg/l | 0.25 | 0.28 | 0.25 | 0.11 | 0.10 | -- |
| Fecal Coliform | col/100ml ml | 100 | 980 | -- | -- | -- | 866.3 |
| Sulfate | mg/l | 1.01 | 4.0 | 4.85 | -- | -- | -- |
| Sulfide | mg/l | 0 | 0 | 0 | – | – | -- |
| Surfactants | mg/l | 0 | 30.26 | 26.7 | – | 0.1 | -- |
| Iron | mg/l | 1.1 | 0.22 | 0.14 | -- | | – |
| Zinc | mg/l | 0.12 | 0.045 | 0.06 | -- | -- | |
| Lead | mg/l | - | - | - | -- | – | 0.025 |
| Chromium | mg/l | -- | -- | – | -- | -- | 0.006 |
| Total Phenolics | µg/l | 125 | 10.1 | – | – | – | – |
| Methl Chloride | µg/l | 2 | 0 | 0 | – | – | -- |

The stormwater quality test results for the four facilities tested are shown in Table 1. In general, each facility used different sampling techniques and tested for different pollutants. One facility tested their retention pond water, three facilities tested grab samples, and two facilities tested composite samples. In general, all of the facilities tested for oil and grease, pH, BOD, COD, TP, and TKN. Other water quality pollutants tested by some of the facilities included TSS, fecal coliform, sulfide, sulfate, surfactants, metals, and US EPA priority pollutants. Since the facilities tested for many different types of pollutants, only those detected were listed in this study. Overall the MDTA runoff quality was much lower than the quality of the other facilities. This lower quality may be attributed to the larger facility size. The MDTA provides more than six times the number of passenger trips provided by the PSTA, which is the second largest facility tested. The MDTA also has more than seven times the number of operating vehicles as PSTA, and the facility area is more than twice as large. The MDTA generally performs the same operations and stores that same materials as the other facilities. Therefore, the size of the facility is probably responsible for the lower runoff quality. These results were compiled to allow for a comparison with the NURP data, the State of Florida Class III Surface Water Standards, and Median Florida Stream Quality data. The ranges of water quality values for each pollutant were determined for all of the sampling types. Mean water quality values were calculated for the grab and composite samples.

The interpretation of the comparison results are dependent on the sample technique considered. Composite samples are typically representative of the quality of the water that will be released into the surface waters. Therefore, the composite mean values will be more representative of the true pollution problems and will be used more heavily than the grab sample mean values when comparing them with the standards. The grab sample represents the first flush of stormwater which typically contains the largest portion of pollutants. Therefore, this sample has the most potential to do harm, and these concentrations represent the initial pollutant concentrations that must be treated by the use of Best Management Practices (BMPs). The grab sample values will be used when composite values are not available. The values from both types of sample techniques are used to evaluate potential problems. The retention pond values are not necessarily representative of the stormwater runoff quality and thus these values are only used in the ranges of the compiled data for comparison with the standards.

It is observed that the stormwater runoff quality from transit facilities is very similar to the stormwater runoff quality of the NURP sites across the nation. In some cases the transit facilities runoff quality values were even much lower than that of the NURP data, suggesting that the runoff quality from these facilities may be slightly better than the other areas represented by the NURP data. In general, transit facilities seem to have the same stormwater runoff quality problems as other similar areas across the nation. However, when comparing the stormwater runoff quality data with the State of Florida Class III Surface Water Standards or the Median Florida Stream Quality Data it is obvious that some quality problems exist. The potential problem pollutants seem to be BOD, COD, TSS, TP, Nitrate + Nitrite, Fecal Coliform, Surfactants, Lead, Zinc, and Total Phenolics.

The BOD grab sample average of 11.1 mg/l and the composite sample value of 3.4 mg/l are much larger than the median Florida Stream Quality value of 1.5 mg/l, indicating possible problems. The COD grab sample mean value of 75.6 mg/l is almost twice the median Florida Stream Quality value of 46 mg/l. The composite sample value of 27.0 mg/l does not indicate any problems; however, the grab samples values and the overall ranges are so large that it can be interpreted that COD does pose a potential threat. The TSS grab and composite sample values of 32.25 mg/l and 19.8 mg/l are greater than the Florida Stream Quality value of 6.5 mg/l, indicating that TSS may pose a potential threat. The Total Phosphorus grab and composite sample mean values of 0.34 mg/l and 0.29 mg/l are much greater than the Florida Stream Quality value of 0.09 mg/l. The range of values also suggest that Total Phosphorus is a problem. The grab and composite Nitrate + Nitrite sample values of 0.19 mg/l and 0.18 mg/l are slightly higher than the Nitrate + Nitrite Florida Stream Quality value of 0.07 mg/l. Therefore, Nitrate + Nitrite pollutants pose a small threat to surface water quality. Fecal Coliform appears to be an extreme problem with a grab sample average of 43,490 col/100 ml. This value is much greater than the median NURP values of 2 1,000 or 1,000 col/100 ml (depending on the temperature), the Florida Water Quality Standard of 200/400 col/100 ml, and the Florida Stream Quality value of 75 col/100 ml. These problems seem to come specifically from the Metro Dade agency, and the comparison shows that Fecal Coliform does pose a serious threat. The Surfactant grab and composite sample values of 30.26 and 13.4 mg/l are much greater than the Florida Water Quality Standards value of 0.5 mg/l. These values show that Surfactants are an extreme problem. The Lead grab sample value of 0.025 mg/l is significantly greater than the Florida Water Quality Standards value of 0.0056 mg/l, indicating serious problems. The zinc composite sample value of 0.06 mg/l is only slightly greater than the Florida Water Quality Standard of 0.059 mg/l, suggesting that zinc may potentially pose some water quality problems. Finally, the total phenolics value of 0.010 mg/l in the grab sample is much greater than the Florida Water Quality Standards value of 0.001 mg/l, signifying a potential water quality problem.

With the problem pollutants in the stormwater runoff from transit maintenance and storage facilities identified, BMPs can be developed to reduce these pollutants. It must be noted that the ranges and mean values for the water quality data received from the transit facilities give very limited information. Many of the water quality parameters listed were not tested by all of the facilities, and not all of the facilities performed grab sample tests and composite sample tests. For these reasons, along with the fact that data from only four facilities was available, this data presented above is a good estimate of the potential stormwater runoff problems, but it is not conclusive. More runoff testing needs to be completed, more facilities need to be tested, and the testing from facility to facility needs to be more consistent to draw conclusions. However, the above data does allow for a general idea of potential problems, it does help to determine that the EPA is justified in including these facilities in the NPDES program, and it does help to determine which BMPs would be most helpful at transit facilities.

### Stormwater Best Management Practices For Transit Facilities

Since it has been determined that stormwater runoff pollution is a problem at transit facilities, it can be assumed that the use of Best Management Practices (BMPs) at these facilities would be beneficial to prevent and treat stormwater runoff. Using the results of the water quality data obtained from other industries together with the characteristics of transit facilities obtained from surveys in this study, the BMPs most applicable to transit facilities were identified and evaluated.

Using nonstructural BMPs to improve stormwater quality requires that the source of the pollution be the main concern. Once the source of pollutants is determined, BMPs can be used to decrease the quantity of pollutants from the source. Using the typical characteristics oftransit facilities in Florida, it was determined that specific areas of the transit maintenance facilities will tend to produce the most pollutants. The main areas at transit maintenance and storage facilities that could potentially produce most of the stormwater runoff pollution problems include the maintenance areas and the storage area. The fuel areas, chemical storage areas, wash areas, and painting areas may also contribute to stormwater runoff pollution.

The maintenance area at a transit facility has a large potential for stormwater pollution problems. Maintenance facilities perform three different types of service: routine preventive maintenance, repairs, and inspections. The performance of routine maintenance and repairs largely depends on the use of greases, oils, and solvents, as well as, other fluids necessary to insure proper performance of the vehicles. These materials provide the largest source of potential stormwater runoff pollutants.

This potential for pollution was determined by considering the steps involved in the maintenance activity, the materials necessary for the maintenance activity, and the amount of material used. Some of the oil and grease, lead, and zinc found in the stormwater runoff quality results may originate from these maintenance procedures shown above.

The storage area of a maintenance facility is usually an open area directly subjected to all stormwater and stormwater runoff. The largest source of pollution in this area of the facility is the parked/stored vehicles. Most vehicles usually have minor leaks. These vehicles leak oils, greases, and fluids from the engine onto the ground which collect until stormwater runoff carries them off the facility lot. The vehicles also collect dust, dirt, pollutants from the engine, and other airborne particulates as they travel. These pollutants adhere to the vehicles until they are washed off by the stormwater and end up in the runoff. This area is probably the largest source of stormwater pollutants from transit facilities. The problem pollutants from this area most likely consist of oil and grease from the leaking vehicles, BOD, COD, TSS, nitrites + nitrates, lead, and zinc.

Various other areas of the maintenance facility that may contribute to stormwater runoff pollution include the fuel areas, chemical storage areas, and wash areas. Painting activities, and fertilizing activities are also large contributors to stormwater runoff pollution. Leaking or spilled fuel and chemicals will often end up on the ground, as will detergents, dirt, and dust from the vehicle wash area. Paint removal, stripping, sanding, and painting produces large amounts of pollution. The paint, paint thinners, dust, rust, and old paint from stripping and sanding can be very hazardous, causing many stormwater runoff problems. Fertilizing grassy areas of the facility can cause large amounts of nutrients to enter the stormwater runoff and cause water quality problems. Finally, if the facility is paved with asphalt, worn and broken asphalt can contribute pollutants to the stormwater runoff. These areas most likely contribute BOD, COD, phosphorus, fecal coliform, surfactants, TSS, and TKN to the stormwater runoff.

**Best Management Practices**

Most of the non-structural BMPs investigated are fairly easy to implement and are very applicable to transit maintenance facilities based on the problem areas listed above. BMPs include:

- Planning - Planning and education are probably the most important of the non-structural BMPs. Essential are a site map designating all of the areas listed above (maintenance areas, storage areas, fuel areas, chemical storage areas, etc...) that have a large potential for pollution.
- Good Housekeeping -These practices consist of: proper cleaning of all areas after work and the immediate proper cleaning of all spills; using care when handling all materials and chemicals; developing spill prevention and response procedures; storing all fluids, materials, and chemicals properly; and frequent inspections to insure that these measures are being implemented.
- Maintenance Procedure Controls - Performing all of the maintenance under a covered area and using drip pans and underground floor drains will prevent pollutants from spreading. Keeping the waste fluids separated from one another, and storing them in separate storage containers will insure that the pollutants can be recycled or disposed of properly. All stored waste fluids should be placed in marked barrels on concrete slabs under cover to insure that they do not leak or come into contact with rain.
- Parts Cleaning Controls - Use of non-hazardous terpene solvents or biodegradable non-chlorinated solvents. Solvent sinks that recycle the solvent are also very helpful; bake ovens can be used to clean parts. All parts cleaning should be done in one area of the facility and solvents should be standardized so that a minimum number of solvents are used.
- Fueling Station Controls - Techniques include installing fuel overflow basins, installing overflow detection devices, instructing employees not to top off the fuel tanks, and protecting the fueling areas from rain by having them covered. Sorbents can also be used to clean up fuel spills rather than rinsing the spills and spreading the pollutants further. Finally, routine inspections of the fuel stations can insure that the stations are working properly and not leaking.
- Painting Controls - Using tarps, vacuums, enclosed outdoor paint areas, and drip pans when sanding, stripping, and painting objects will decrease the amount of pollutants leaving the area. Plastic media, dry ice, and water jets can be used to strip the paint instead of chemicals and thinners. Using high transfer paint guns will decrease the over-spray of paint. Finally, the use of non-toxic and water-based paints should be maximized.
- Vehicle Washing / Cleaning Controls - Vehicle washing should be done in a separated area of the facility. The cleaning water should flow into a self-contained bay where it is treated. Cleaning water should not end up as runoff. Phosphate-free biodegradable detergents should be used when possible.
- Sorbent Use - All spill and leaks should be cleaned immediately with sorbents, rags, or mops. Spills should never be rinsed with a hose.
- Preventative Monitoring - All the previously mentioned areas of the facility should be inspected on a routine basis to insure that the BMPs are being implemented. Parked and stored vehicles should be frequently inspected for leaks, and drip pans should be used if leaks occur.
- Education - Once the entire stormwater pollution prevention plan has been developed, all employees should be educated about the program and trained properly, and worker incentives should be implemented..

The purpose of structural BMPs is to treat polluted runoff. Unlike nonstructural BMPs, these activities do not prevent stormwater runoff pollution, but instead these BMPs treat the already polluted runoff coming from the facility. The structural BMPs evaluated for transit maintenance facilities are extensive. The overall pollutant removal level was determined for each BMP by using mean values of the removal rates for each BMP obtained from this extensive literature search.

Each BMP was evaluated based on its applicability to transit maintenance and storage facilities. **To** be applicable to transit maintenance and storage facilities, the BMPs must have reasonably high removal rates of all of the problem pollutants found at these facilities. The other important BMP characteristics that were considered in this evaluation include land requirements, quality control, maintenance level, and cost. Typically, transit facilities need BMPs with low land requirements, that provide quality control, require low maintenance, and have a low cost. The overall most important BMP characteristics are the pollutant removal rates. Since most of the pollutants found at the transit maintenance facilities were only slightly higher than the comparison standards, a removal of 25 to 50 percent would accomplish the necessary pollutant reduction. Therefore, all BMPs with a low overall pollutant reduction capability were rejected. The remaining BMPs were ranked in each characteristic category. These rankings were averaged, and any BMP that had an overall above average ranking was accepted, and the others rejected.

Using the stormwater runoffwater quality results from the Florida transit facilities presented earlier, the accepted BMPs were then rechecked to insure that the chosen BMPs removed the necessary individual pollutants as needed by the transit maintenance facilities. The most applicable structural BMPs for transit maintenance facilities were determined to be swales, porous pavement, dry retention ponds, vegetated filter strips, extended dry detention ponds, wet retention ponds, wet detention ponds, infiltration trenches, and wet detention ponds with vegetation.

## CONCLUSIONS AND RECOMMENDATIONS

The U.S. EPA requires that all stormwater runoff discharges from transit maintenance and storage facilities be permitted by an NPDES stormwater permit. The U.S. EPA relied on little or no stormwater quality test results specifically from these facilities in classifying them as problem areas. However, based on the characteristics of transit in facilities in Florida, it can be stated that transit maintenance and storage facilities have many characteristics and participate in many activities that have the potential to cause stormwater runoff problems. The most influential characteristic at these facilities that affects stormwater runoff is the large amount of impervious area present. The most influential activities at these facilities include participating in vehicle repair, vehicle painting, vehicle washing, vehicle fueling, and storage of materials such as fuel, oils, lubricants, grease, solvents, and other chemicals. Finally, by the analysis of stormwater runoff quality results from some of these facilities, it can be concluded that stormwater runoff pollution problems do exist at transit maintenance and storage facilities and the U.S. EPA is justified in including these facilities in the NPDES stormwater program. The pollutants posing potential problems include BOD, COD, TSS, TP, Nitrate -!-Nitrite, Fecal Coliform, Surfactants, Lead, Zinc, and Total Phenolics. The best way to prevent and treat these pollutants is through the use of a combination of nonstructural and structural Best Management Practices. The nonstructural BMPs decrease the pollution originating from the source, while the structural BMPs remove any remaining pollutants from the stormwater before discharging into surface waters. The most applicable nonstructural BMPs and structural BMPs for transit maintenance and storage facilities were determined and are recomended for use by transit facilities and are shown in Table 2.

| TABLE 2. BEST MANAGEMENT PRACTICES APPLICABLE TO TRANSIT MAINTENANCE FACILITIES | |
|---|---|
| NONSTRUCTURAL BMPS: | STRUCTURAL BMPS: |
| - PLANNING | - SWALES |
| - GOOD HOUSEKEEPING | - POROUS PAVEMENT |
| - MAINTENANCE PROCEDURE CONTROLS | - WET RETENTION PONDS |
| - PARTS CLEANING CONTROLS | - DRY RETENTION PONDS |
| - FUELING STATION CONTROLS | - EXTENDED DRY DETENTION PONDS |
| - PAINTING CONTROLS | - WET DETENTION WITH VEGETATION |
| - VEHICLE WASHING CONTROLS | - VEGETATED FILTER STRIPS |
| - PREVENTATIVE MONITORING | - INFILTRATION TRENCHES |
| - EDUCATION | |

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# FLORIDA LAKE REGION AND THE RESPONSE TO STORMWATER PERTURBATIONS: SOUTHERN LAKE WALES RIDGE VS. LAKE WALES RIDGE TRANSITION LAKES IN HIGHLANDS COUNTY

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## ABSTRACT

The two primary Florida Lake Regions, defined by Canfield et al., in Highlands County are the Southern Lake Wales Ridge (SLWR) and the Lake Wales Ridge Transition (LWRT) regions. Combined, these lakes represent approximately 19,000 acres of surface water in 39 public access lakes on the Ridge. SLWR lakes, characterized by sandy, low nutrient watershed soils that result in deep, clear, oligotrophic lakes, account for 14,000 acres in 20 lakes. LWRT lakes, with typically poorly drained, high nutrient, muck soils in their drainage that contribute to higher ambient nutrient levels and tannin stained water in naturally atrophic lakes, account for the remaining 5,000 acres. When predicting the response of these lakes to stormwater inflows, it might be expected that the SLWR lakes would be more sensitive to human induced perturbations than their more buffered companions. This study uses monthly ambient nutrient data from lakes of comparable size, watershed use and degree of stormwater abatement to test this hypothesis, prioritize lakes for stormwater abatement, and guide future work,

# IMPACTS OF ATMOSPHERIC DEPOSITION ON STORMWATER QUALITY

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## ABSTRACT

The goal of this project was to contribute to our knowledge regarding the influence of atmospheric deposition on surface water quality, particularly nitrogen loads in stormwater runoff. The project had two major objectives: estimating the total nitrogen loads in stormwater runoff contributed by atmospheric deposition versus other sources for sampled urban/residential basins in the Tampa Bay Watershed; and estimating the retention rates of nitrogen for these basins.

Stormwater sampling from two urban residential study watersheds utilizing automated sampling equipment allowed determination of non-point source nitrogen loadings using standard techniques of composite sampling. Wet and dry atmospheric deposition sampling at the nearby Gandy Bridge Intensive Atmospheric Deposition site allowed inputs from those sources to be quantified.

The study concluded that about 28 % of the non-point source (stormwater runoff) nitrogen loading is directly attributable to wet atmospheric deposition alone, with the remainder coming from the watershed. The total atmospheric deposition contribution to the Bay via non-point source discharge is likely greater but occurs through indirect watershed processes that could not be completely quantified during the study.

## INTRODUCTION

Understanding the relative contributions of atmospheric sources versus land-based sources of pollutant loads to Tampa Bay strengthens the basis for informed management actions currently being defined through the TBNEP Tampa Bay Comprehensive Conservation and Management Plan (CCMP) and provides information needed to implement a Nitrogen Management Strategy.

The project consisted of two major tasks: the collection of field data and the analysis of those data. Measured precipitation volume and quality data were obtained from the Gandy Bridge site, and measured stormwater quality data were collected from two locations near the Gandy Bridge site,

The data were used to estimate the proportion of the atmospheric deposition nitrogen loading delivered to the study site that contributes to nonpoint source loadings, and the proportion of the atmospheric deposition nitrogen load that is retained (attenuated) in the watershed. Additionally, the proportion of the total nonpoint source nitrogen load from the drainage basin that is directly or potentially attributable to atmospheric deposition was estimated. **A** coarse mass balance was prepared to place the atmospheric inputs and stormwater discharges in perspective.

Previous estimates of atmospheric deposition directly to the surface of Tampa Bay yielded a contribution of 27% of the total nitrogen load to the bay over the 1985-1991 time period (Zarbock et al., 1994). Given the relative importance of this load in comparison with the total nitrogen load to the bay, it was determined that a more accurate estimate of atmospheric deposition of nitrogen to the bay was necessary.

## METHODS - STORMWATER SAMPLING PROGRAM

The objective of the stormwater sampling program was to measure input rainfall amounts, runoff discharge amounts, and to conduct chemical analyses of the runoff discharge to allow calculation of pollutant loads. Watershed sampling sites were selected based on the basin meeting a defined set of criteria, The watershed selection criteria were:

- **Drainage basin *size***
- **Homogeneity of land use**
- **Proximity to Gandy Bridge Atmospheric Deposition Monitoring Site**
- **Avoidance of backwater conditions**
- **Sufficient flow for sample collection**
- **Accessible sampling site within the basin**

To accommodate the criteria, the project team selected two basins located within the nearby Norma Park area. The City of Tampa previously conducted a drainage study of the Norma **Park** area, which includes portions of south Tampa at and adjoining the Gandy Bridge site. The study provided a detailed assessment of subbasin delineation, surface water drainage patterns, drainage infrastructure, land use and soils characteristics, and other information that proved useful during the study.

**Site 1 (Bay Vista):** This sampling site was located east of Manhattan Avenue and west of Dale Mabry Highway, south of Euclid Avenue at Bay Villa Avenue. The stormwater drainage pattern is generally from east to west, then south toward Bay Villa Avenue. The sampled basin encompassed approximately 54.0 acres and consists of approximately 95% residential **and 5%** open area associated with **an** institution (school). The residential land use is of similar uniform density (single family with **5** - **8** lots per acre). Site 1 is approximately 1.5 miles from the Gandy Bridge site and is relatively far inland. Tidal inflows were not observed during field reconnaissance. The sampling site was located in a manhole with an 18 inch pipe in a functional stormwater conveyance. The site was secure and access was available to locate instruments and to drive a vehicle close to the site. Based on simple calculations using the SCS Runoff Curve Number Method, a 54.0 acre basin would generate approximately 4,500 cubic feet of runoff during a 0.5 inch event. This would be sufficient to reach the four inch depth of flow in the pipe desired for sampling, so the flow criteria were met.

**Site 2 (Fair Oaks):** This sampling site was located east of Westshore Boulevard and west of Manhattan Avenue, at the intersection of Trask and Lawn Avenue. The stormwater drainage pattern is generally from east to west then north of Fair Oaks to a main storm drain. The sampled basin encompassed approximately 17.9 acres and consists entirely of residential land use. The residential land use is of similar uniform density (single family with 5 - 8 lots per acre). Site 2 is approximately 0.8 miles from the Gandy Bridge site and is relatively far inland. Tidal inflows were not observed during field reconnaissance. The sampling site was located in a manhole with an 18 inch pipe in a functional stormwater conveyance. The site was secure and access was available to locate instruments and to drive a vehicle close to the site. Based on the SCS Runoff Curve Number Method, a 17.9 acre basin would generate approximately 1,500 cubic feet of runoff during a 0.5 inch event. This would be sufficient to reach the four inch depth of flow in the pipe desired for sampling, so the flow criteria were met.

**Stormwater Sampling Methodology**

Storm water samples were collected at both sampling sites from May 1997 to January 1998 using ISCO automated samplers. Each sampler recorded water level (measured by bubbler), and was activated to collect a minimum sample (100 ml) based on flow intervals. Prior to sampler installation and programming, a stage-discharge relationship (rating curve) was developed for each sampling site through manual flow and water level measurement. Samples were composited to obtain one flow-weighted composite sample for each runoff event. This limited analytical costs while providing a good representation of the total load from individual runoff events.

**Sampled Stormwater Parameters**

The objective of this study was to investigate the influence of atmospheric deposition of nitrogen on stormwater runoff quality, and to estimate the relative proportion of nitrogen from atmospheric sources that is retained in the watershed and that enters the runoff stream. For these purposes, composite stormwater runoff samples were analyzed for TKN, $NO_2/NO_3$, and specific conductance. The nitrogen species when summed yielded a reasonable estimate of TN concentration, and the specific conductance helped to identify potential contamination of the stormwater stream by other sources. All sampling was conducted in strict accordance with the EPA approved Quality Assurance Project Plan.

In some cases, runoff discharge from longer storm events continued into a second day (typically due to either very large events or multiple episodes of rainfall over an extended time). Additional composite samples were also collected on the second day for better estimation of concentrations representative of the entire discharge period. Determining the total nitrogen load in stormwater runoff was a straightforward calculation of concentration times runoff volume but relied on the key assumption that the composite sample was representative of the entire discharge volume. The measured concentrations obtained from the composite samples were assumed to be representative of the entire discharge period for the purpose of calculating loads. In the instances where stormwater discharge continued for a second day, the analytical results from the two Composite samples were weighted in proportion to the observed rainfall for each day to develop representative concentrations and loadings for the entire period of discharge.

**METHODS - ATMOSPHERIC SAMPLING PROGRAM**

The objective of the atmospheric deposition data collection and analysis for this project was to determine the amount of total nitrogen loadings to the watershed of the bay resulting from direct deposition to the watershed.

Data collection at the Gandy site and the meteorological station began in August 1996. Analyses of wet deposition samples yielded values of ammonium, chloride, sulfate, potassium, magnesium, specific conductance, orthophosphate, nitrate, sodium, calcium, and pH. In addition to the wetfall samples, rainfall amounts were taken from an on-site rain gauge. Wetfall samples were collected at least once weekly, and often more frequently.

For collection of data needed for calculation of the dry deposition of nutrient species to the bay, and, as we assume, to the watershed, the meteorological site in Tampa Bay provides input to the NOAA buoy model (Valigura, 1995) to determine deposition velocities of particulates from 1-2mm and for nitric acid (gaseous). The relevant physical parameters are wind speed, air temperature, water temperature, and relative humidity. The dry deposition sampling apparatus consists of a dual flow-through system containing annular denuders, for gaseous components measurement, and a nylon filter system, for collection of particulates. Sampling is done for a 24-hour period every six days, with a pumping rate of 10 liters/minute over the 24 hours. The samples are analyzed for gaseous and particulate nitrate, sulfate, and ammonia,

Wet deposition sampling is done following the protocols developed by the National Atmospheric Deposition Program (NADP) Atmospheric Integrated Research Monitoring Network (NADP/AIRMoN). A wet bucket collects rainfall samples, and samples of greater than 10 ml are sent to the Central Analytical Laboratory **(CAL)** of the Illinois State Water Survey, where the samples are analyzed utilizing the same methods as those used by the NADP/AIRMoN program.

**Dry** deposition is sampled using the denuders and filter packs for determination of gaseous and particulate nitrogen concentrations in the atmosphere. The denuders and filter packs are sent to QST (formerly Environmental Science and Engineering) for analysis.

Data reduction is straightforward. For wet samples that span several days between collections, the measured wet deposition is evenly distributed over the days for deposition calculation. To determine the wet deposition of nutrient species, the concentrations of various nitrogen species are determined, and the total mass **flux** due to wetfall is the product of the chemical concentration in the rainfall, the rainfall depth, and the surface area of the watershed.

Determination of dry deposition is more involved. Concentrations of various nitrogen species in the atmosphere are determined, and deposition velocities for the various nutrient components to the bay, and thus, by the assumptions of this effort, to the watershed, are determined utilizing the Buoy model developed by NOAA. The NOAA model uses as input meteorological data collected near the intensive deposition sampling site.

For the purposes of calculating dry deposition, the measured concentrations taken every six days are allowed to represent the concentrations on the day of sampling, and on the previous 2.5 days and the following 2.5 days. The concentrations of the various chemical species in the atmosphere are then multiplied by the appropriate deposition velocity, the surface area of the watershed, and the time period over which the deposition velocity is calculated, to determine the total flux of each nutrient species.

The sum of the wet mass flux and the dry mass flux of nitrogen species to the bay and its watershed represents the deposition of nitrogen due only to those nitrogen species converted by the annular denuders to nitrate and ammonium, in addition to the particulate forms of nitrogen collected by the nylon filter pack and the nitrate and ammonium from the wet deposition.

Atmospheric concentrations of nitrogen species were determined from data collected every six days at the Gandy site for August 1996 through December 1997. Meteorologic data were collected for **the** same time period, and used to determine dry nitrogen deposition **fluxes** to the surface of the bay (Pribble and Janicki, 1998), and to the land surface following the assumptions of this study.

## ANALYTICAL APPROACH RESULTS

To relate atmospheric deposition of nitrogen to nitrogen loading in stormwater runoff, a determination of nitrogen deposition for each stormwater runoff event was made. In addition, dry nitrogen deposition, which may be **an** additional component of stormwater runoff nitrogen loading to the bay, was estimated. The wet, dry, **and** total atmospheric deposition of nitrogen was then compared to the stormwater nitrogen loading from the watersheds.

To determine nitrogen concentrations in rainfall events that resulted in stormwater discharge at the two watersheds, stormwater samples collected **from** the watersheds were date-matched with wet atmospheric samples from the Gandy Site. Because the rainfall amounts differed between Gandy and each study site, measured concentrations of nitrogen from the Gandy Site wet samples were applied to the rainfall amounts observed at the two sites, so that a total wet atmospheric deposition of nitrogen was determined for each watershed. The concentration of nitrogen in the rainfall was assumed to be the same at all locations. The total number of date-matched samples for Site 1 was 17, with 11 date-matched samples from Site **2.**

**Dry** atmospheric deposition of nitrogen was estimated by applying the cumulative dry nitrogen deposition, **as** calculated from the Gandy Site data and the meteorological data (Pribble and Janicki, 1998), from the end of the rainfall event back to the end of the preceding rainfall event.

The 'Stormwater Runoff Loading' is derived by applying the stormwater concentration to the stormwater volume to generate a total mass loading and then dividing by the watershed area to generate **an** area-weighted loading.

The overall watershed relationship of wet atmospheric deposition load and stormwater discharge load is illustrated graphically in Figure 1. The linear best-fit equation is $L_{SW} = 0.9118 * L_{AD}$, with an $r^2$ of 0.78. Here, $L_{SW}$ is the stormwater area-normalized nitrogen loading (lb/acre) and $L_{AD}$ is the area-normalized atmospheric wet deposition of nitrogen (lb/acre).

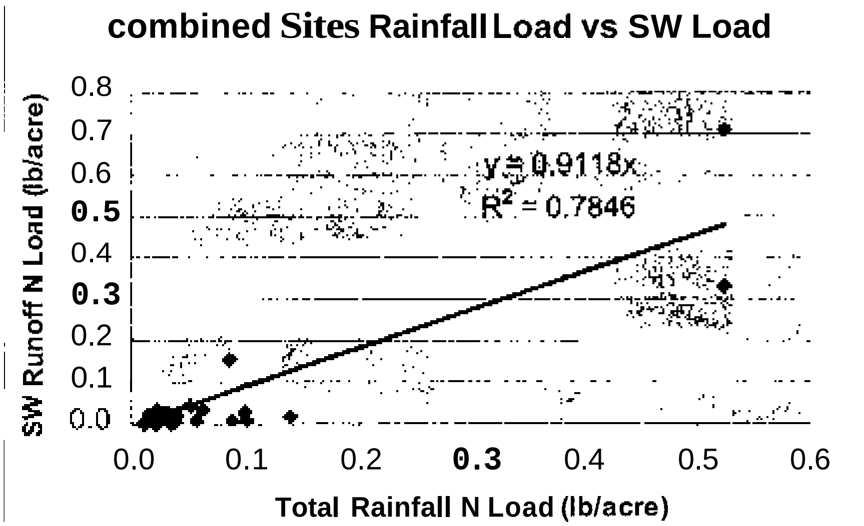


Figure 1 Combined Sites Rainfall vs Stormwater N Loads

The slope of the best-fit line for the two watersheds combined, 0.91, represents the apparent overall transfer coefficient of the atmospheric load, or that fraction of wet nitrogen deposition accounted for via the stormwater nitrogen loading, although the nitrogen in the stomwater loading is not necessarily the same nitrogen as was deposited in rainfall.

The methods used to determine nitrogen concentrations in the rainfall on the two watersheds resulted in only some of the actual measured rainfall events being utilized for this analysis (those that occurred simultaneously with rainfall events at the Gandy site that were analyzed for concentration data). Assuming that the total annual nitrogen loadings to the two watersheds are similar to that estimated at Gandy, and that approximately 91% of the nitrogen deposited via rainfall is represented in the stormwater runoff from the two sites, total annual stormwater nitrogen loadings from the two watersheds are estimated at 352 mg/m$^2$ (3.14 lb/acre). **An** alternate estimate of the annual stomwater load based on scaled-up stormwater discharge data (50 inches normal rainfall total / 30 inches study rainfall total **x** study loading) yields approximately 4 lb/acre-year. The extrapolated annual estimates should be used with caution because they do not take into account possible variations at other times of year.

The overall input-output relationship described above indicates that all stormwater discharge could be numerically accounted for by total wet atmospheric loads. This overall relationship may indicate that the watershed is an equilibrium system in which the discharges are controlled by the 'excess' inputs from wet atmospheric deposition. Alternatively, the apparent relationship may simply be coincidence.

We know, however, that this one-to-one relationship cannot hold true during a particular event because the rainfall that does not discharge from the basin is physically prevented from surface discharge by infiltration into the soil or by impoundment. The remaining rainwater discharges as runoff. On the assumption that the wet atmospheric nitrogen load is similarly partitioned into a retained fraction and a discharged fraction, the relationship between the wet atmospheric nitrogen loads associated only with the portion of the rainfall volume that ultimately discharged as runoff was examined versus the stormwater discharge loads.

Runoff-Volume Normalized Rainfall Loading' considers the volume of runoff in the state that it entered the watershed as rainfall to determine the associated input loading. Placing the input load on this basis (only the input amount associatcd with the water that is ultimately discharged) allows us to evaluate the relationship between the nitrogen load that was initially present in the water when it entered the watershed and the nitrogen load that is present when the same volume of water is discharged as stormwater.

To reitcratc, thc relationship is labeled 'volume normalized' because it represents the input loading and discharge loading for the same volume of water, i.e. the rainfall runoff volume in the stale that it entered the watershed and in the statc that it left the watershed.

Combining events from both watersheds results in the relationship graphically illustrated in Figurc 2. The linear best-fit equation is $L_{SW} = 3.5456*L_{AD}+0.0082$, with an $r^2$ of 0.983. In this case, $L_{SW}$ is the stormwater area-normalizcd nitrogen loading (lb/acre) and $L_{ADRV}$ is the area-normalized atmospheric wct dcposition of nitrogen (lb/acre) normalized to the runoff volume.

If the axes are reversed, the relationship indicates that atmospheric wet deposition directly accounts for an overall 28% of the nitrogen appearing in the stormwater discharge load. The rclationship shown represents the direct wet atmospheric nitrogen contribution indicating that the remaining nitrogen load is derived from the watershed during the runoff process.

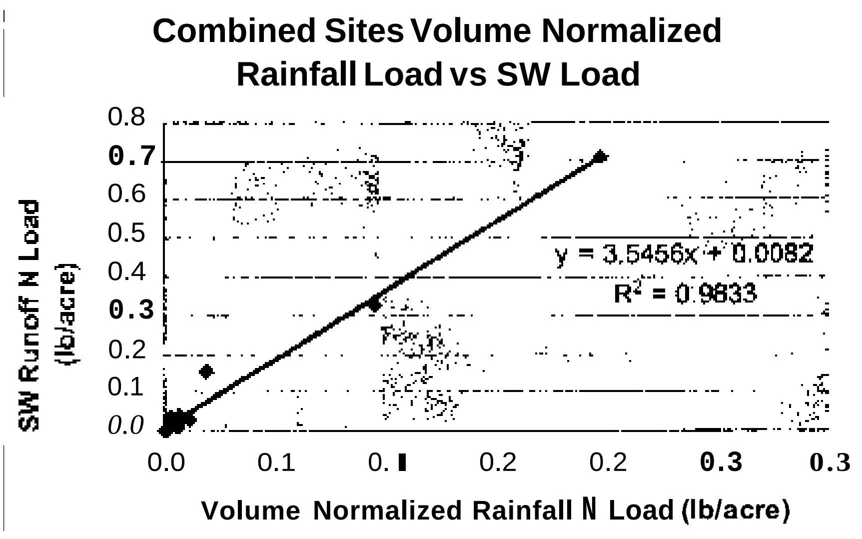


Figure 2 Combined Site Normalized Rainfall vs Stormwater N Loads

In an attempt to relate stormwater runoff nitrogen loads to possible cumulative effects of total (wct + dry) atmospheric nitrogen deposition, analyses similar to those dcscribed above were complctcd for thc sum of wet and dry atmospheric deposition. Daily dry deposition values, as cstimatcd from atmospheric nitrogen concentrations and air-to-water deposition velocities, were summed between rain events at the watersheds, then added to the wet deposition of nitrogen for each event. These estimated total atmospheric deposition nitrogen loads did not account for any of the various differences which may result in different *dry* deposition rates of nitrogen over land **and** water (as discussed previously).

Total (wet + dry) atmospheric nitrogen deposition values were only estimated for those events for which both wet and dry deposition values were available. The combination of the data from both watersheds showed no apparent relationship between stormwater loading of TN and total (wet + dry) atmospheric deposition, The $r^2$ from the best linear fit between these data from the combination of both watersheds is only 0.04. Given these data and the lack of fit between them, no conclusions may be drawn **as** to the relationship between total (wet + dry) atmospheric deposition of nitrogen and nitrogen loading in stormwater.

The assumption that dry deposition of nitrogen to a terrestrial environment is the same as that to the surface of Tampa Bay is not supported or rejected by this analysis. **Any** dry deposition of nitrogen to the surfaces of the terrestrial environment may be assimilated by vegetation, and thus not available for entrainment in the stormwater runoff. Likewise, however, nitrogen particles reaching the surfaces of the watersheds through dry deposition may not remain within the watersheds, but may be resuspended and displaced. Neither course may be determined from this analysis.

## CONCLUSIONS

The study examined the potential contributions of wet and dry atmospheric nitrogen deposition on stormwater quality both separately and in combination, but succeeded only in identifying clear relationships for wet atmospheric inputs.

The stormwater discharge load may be entirely attributable to atmospheric wet deposition loads via both direct discharge and indirect cumulative watershed processes. On an overall numerical basis, nearly all of the annual watershed stormwater discharge loading can be accounted for by the annual atmospheric wet deposition nitrogen loading. If this overall relationship proves meaningful and is not simply a numerical coincidence, it implies that **an** equilibrium situation potentially exists within these watersheds **and** the primary driving force behind the nitrogen discharged in stormwater is the excess wet input from atmospheric sources. Under this scenario, the nature of the rainfall – runoff process dictates that only a fraction of the nitrogen discharge is directly attributable to the atmospheric source. The remainder of the stormwater nitrogen discharge could conceivably stem almost entirely from the wet atmospheric inputs through indirect time-lagged watershed processes that eventually result in removal from the watershed of this quantity of nitrogen as excess in **an** equilibrium system,

On an event-by-event mass throughput basis, approximately 15**- 20%** of the total annual rainfall volume and, by inference, 15 - 20% of the associated atmospheric wet deposition nitrogen loading is discharged from the basin immediately as runoff, The remaining 80 - 85% of the atmospheric wet deposition nitrogen input is assumed to be attenuated at least temporarily within the basin, entering the normal nutrient cycle and becoming indistinguishable from other elements of the watershed stores of nitrogen.

The stormwater discharge nitrogen loading from the study watersheds is composed **28%** from direct atmospheric wet deposition sources and 72% from in-watershed sources on average during any particular storm event. In-watershed sources during a given storm event include cumulative amounts of previous dry atmospheric deposition and the non-discharged wet atmospheric deposition from previous events. No clear relationship could be identified between these cumulative retained atmospheric nitrogen sources and stormwater discharge loadings during this study.

If the relationship identified for the study watersheds holds true in other watersheds, particularly in similar urban residential areas, the relationship to determine the directly-attributable atmospheric wet deposition contribution to total stormwater discharge loads is:

Percentage directly-attributable atmospheric wet deposition load = (Watershed Runoff Coefficient x Atmospheric Wet Deposition load / Stormwater Discharge load) x 100

The results of the study are directly applicable to similar older urban residential areas with less than lush yards (i.e moderate to low levels of maintenance). The study results for direct contributions of wet atmospheric nitrogen deposition should be transferable to other vegetated areas with reasonable success by following the general rule outlined above.

The conclusions derived from the collected data during this project support the standard understanding of terrestrial nitrogen processes. The majority of the stormwater loading derives from in-watershed sources that include previously deposited atmospheric nitrogen rather than from direct atmospheric sources for the urban residential watersheds studied. Atmospheric nitrogen sources that are not discharged immediately contribute a small amount to the overall watershed standing crop and are thus subject to dissolution, transport and discharge during future storm events.

The primary recommendation is that similar studies should be conducted in other Tampa Bay area watersheds that represent the other major land uses contributing significant non-point discharges to the Bay. The atmospheric deposition program remains in place and data continues to be collected, providing the opportunity to gain knowledge of the atmospheric deposition - stormwater discharge relationship in additional watersheds. Future studies should span at least a full year of sample collection to better address issues of variability by season. Alternative methods of dry deposition collection should be explored that better simulate the interaction of substrate and washoff as experienced in the terrestrial environment. **A** more complete serial data collection would allow greater opportunity to identify possible relationships between cumulative *dry* deposition, cumulative non-discharged wet deposition, and subsequent stormwater discharge loading.

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# LITTLE LAKE JACKSON STORMWATER RUNOFF ANALYSES

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## ABSTRACT

Little Lake Jackson is located in Highlands County in the southern portion of the City of Sebring. Recurring algal blooms and accompanying poor water clarity in Little Lake Jackson prompted a diagnostic feasibility study of the lake, which was funded jointly by the Peace River Basin Board of the Southwest Florida Water Management District, Highlands County, and the City of Sebring. As part of this study, an analyses of surface water runoff was completed. Most surface water runoff enters Little Lake Jackson through the primary inflow drainage canal located in Sub-basin 1. The most intensive sampling was performed at the mouth of this inflow. At this site concentrations of orthophosphorus, total phosphorus, and ammonium were significantly greater within storm event samples than within base flow samples ($\alpha$=0.003, 0.007, and 0.02, respectively, Wilcoxon-Mann-Whitney rank-sum test). Significant correlations (Spearman correlation) were observed between discharge (log transformed) and the following nutrient concentrations (log transformed): discharge and orthophosphorus rho=0.855, p=0.001, n=14; discharge and total phosphorus rho=0.893, p=0.001, n=14, discharge and ammonium rho=0.828, p=0.001, n=14. Concentrations of orthophosphorus, total phosphorus, ammonium, and total nitrogen measured within the primary inflow were unusually high during one of the largest storm events (occurred on 10-08-96). Concentrations during this particular storm event appear to be primarily associated with fertilizer application within the Sebring Municipal Golf Course. Approximately five tons of fertilizer were applied to the City golf course, one day prior to the storm event.

## INTRODUCTION

Little Lake Jackson is located in Highlands County in the southern portion of the City of Sebring (Figure 1). The lake which comprises only 156 acres, is small in comparison to its drainage basin or watershed which comprises approximately 1048 acres.

### Water Quality

Little Lake Jackson is a moderately eutrophic lake. The mean concentrations for total nitrogen, total phosphorus, and chlorophyll a measured during the study period were 1.29 mg/L, 0.029 mg/L, and 30.5 *u*g/L, respectively. The mean FTSI value was 54.6, Both the mean and median N:P ratio

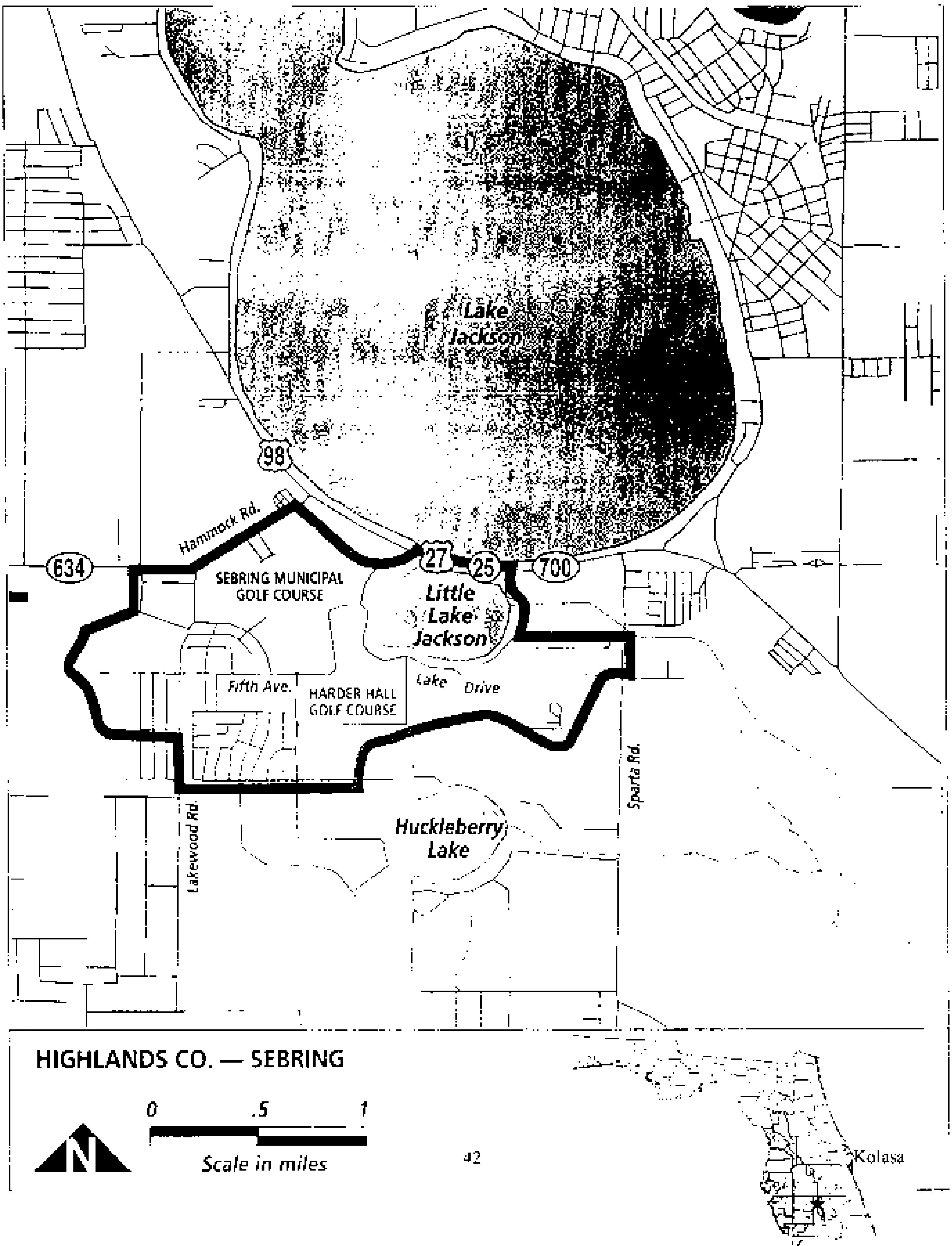



Figure 1. Little Lake Jackson General Location Map

indicated that the lake was phosphorus limited (126 and 50, respectively). Although Little Lake Jackson has average water quality when compared to many other lakes throughout Florida (FDEP database, Freidemann and Hand 1989), Little Lake Jackson has poorer water quality than most lakes in Highlands County.

Recurring algal blooms and accompanying poor water clarity in Little Lake Jackson have raised concerns from lake front homeowners and Sebring citizens for the past decade. Questions emerged regarding the possible degradation of water quality of the lake. Although recent water quality data were available, the long term historical condition of the lake was not known since little historical data were available. With intentions of finding solutions to the recurring problems within the lake and evaluating possible shifts in water quality, a diagnostic feasibility study was nominated by the Highlands County Board of County Commissioners and the City Council of Sebring as a cooperative funding project to the Peace River Basin Board of the Southwest Florida Water Management District. The study began in October 1995 and ended in October 1997. The study included **an** assessment of the lake, as well as its watershed. This paper presents the findings of the runoff assessment, which was one of the components of the watershed assessment.

## Historical Changes

**A** comparison of 1952, 1974, and 1993 Soil Conservation Service (SCS)(available from the National Archives) aerial photographs for this region, revealed that the Little Lake Jackson watershed has changed dramatically over the last 50 years. The watershed which was mostly open land during the 1950's, has been almost completely developed as residential and recreational land uses (golf courses). Most of the drainage ditches and residential canals were constructed within the watershed during the 1960's and early 1970's. House construction progressed most rapidly during the late 1960's, 1970's, and 1980's. Numerous ponds were constructed during the construction of two golf courses during the early 1980's. Several unaltered depression wetlands, possibly cut-throat seeps, which are evident within 1950's aerial photographs are no longer evident within the aerial photos of the 1980's.

## Current Land Use

The Little Lake Jackson drainage basin comprises approximately 1084 acres. Most of the watershed has been developed as residential or recreational land uses, which respectively comprise approximately 63% and **2**1% of the entire surrounding watershed. Although the watershed has been extensively altered, an overall park-like setting has been created within this community as the result of its numerous recreational facilities, which include three golf courses and a community ball park. Many of the residential yards within the western portion of the watershed remain forested with large longleaf pine trees and other native plants remnant of the original plant communities of this region. In addition, a significant portion of the watershed has remained undeveloped. Approximately 10.5 % of the watershed is collectively comprised of pine flatwoods, shrub and brushland, upland coniferous, and open urban land.

## Land Use Types

Three separate drainage sub-basins were defined within this watershed during the study (Figure 2). Sub-basin 1 and 2 have similar land uses. Both are dominated by residential and recreational land uses (Table 1)(59.5 % residential and 25.2 % recreational within Sub-basin 1, and **69.7** % residential and **29.3** % recreational within Sub-basin 2). Land use within Sub-basin 3 was somewhat different. Although residential development comprised 61.5% of Sub-basin 3, recreational land-use was absent from this sub-basin. In addition, several land uses were specific to Sub-basin 3. These include cropland and pasture, citrus crops, transportation land, and coniferous uplands. Additionally, this sub-basin contains the largest area of commercial property.

## Soil Types

Soil types within the Little Lake Jackson watershed primarily include the Satellite-Basinger-Urban Land Complex and the Basinger, St. Johns, and Placid Complex, which respectively comprise 50 % and 29% of the soils within the watershed. These two dominant soil types are described as ranging from somewhat poorly drained to very poorly drained (SCS 1989). According to the **SCS** (1989) the Basinger, St. Johns, Placid complex is found in lower areas of the ridge containing seeps, locally known as cutthroat seeps. **As** a result of their wetness, limitations occur for development within regions containing these soils. These include limitations for urban construction, and moderate to severe limitations for cultivated crops, citrus crops, and pasture crops. Installation of a proper drainage system is recommended to increase agriculture **and** urban development potential. In addition, septic field mounding is recommended for septic installation within these areas, since severe wetness inhibits absorption and filtering capabilities.

## Surface Water Drainage within Sub-Basins

The primary surface water inflow to Little Lake Jackson is located in Sub-basin 1, the largest sub-basin (497 acres), where a system of drainage ditches and canals converge and enter the western side of the lake through one main inflow canal (Figure 2). Unlike Sub-basin 1, there are no deep drainage canals or ditches located within Sub-basin 2 and Sub-basin 3. Runoff within Sub-basin 2 and sub-basin 3 primarily occurs as sheet flow which is conveyed through shallow swales. Continuous flow was observed through the main inflow canal within Sub-basin 1, as the result of the shallow water table within this watershed. Continuous flow was also observed within one of the upstream ditches which was excavated at a greater depth (approximately 12 feet total depth) than other ditches in this region,

Since most surface water drainage occurs within Sub-basin 1, stormwater sampling analyses were primarily limited to this basin. This study reports the findings of the stormwater analyses performed within Sub-basin 1.

## METHODS AND MATERIALS

### Location

Although two other locations were sampled within the upstream reaches of the primary inflow ditch located within Sub-basin 1, the results discussed within this report will be limited to the station closest to lake (Station 1 - primary inflow station, Figure **2).** This station was located at the drop-pipe structure just west of Golf View Road, approximately 300 feet upstream of the lake. This structure retains water at an elevation higher than that of the lake, prior to discharge. During the study (1995-1997) the lake elevation ranged from **0.55** feet to 3.81 feet below the structure. Pollutant concentrations measured at this appear to represent final pollutant concentrations entering the lake through this drainage system. Nutrient concentrations measured at this station will be used for determining nutrient loads entering the lake through the primary inflow (see Sub-basin 1 - Nutrient Load Analyses).

### Rainfall

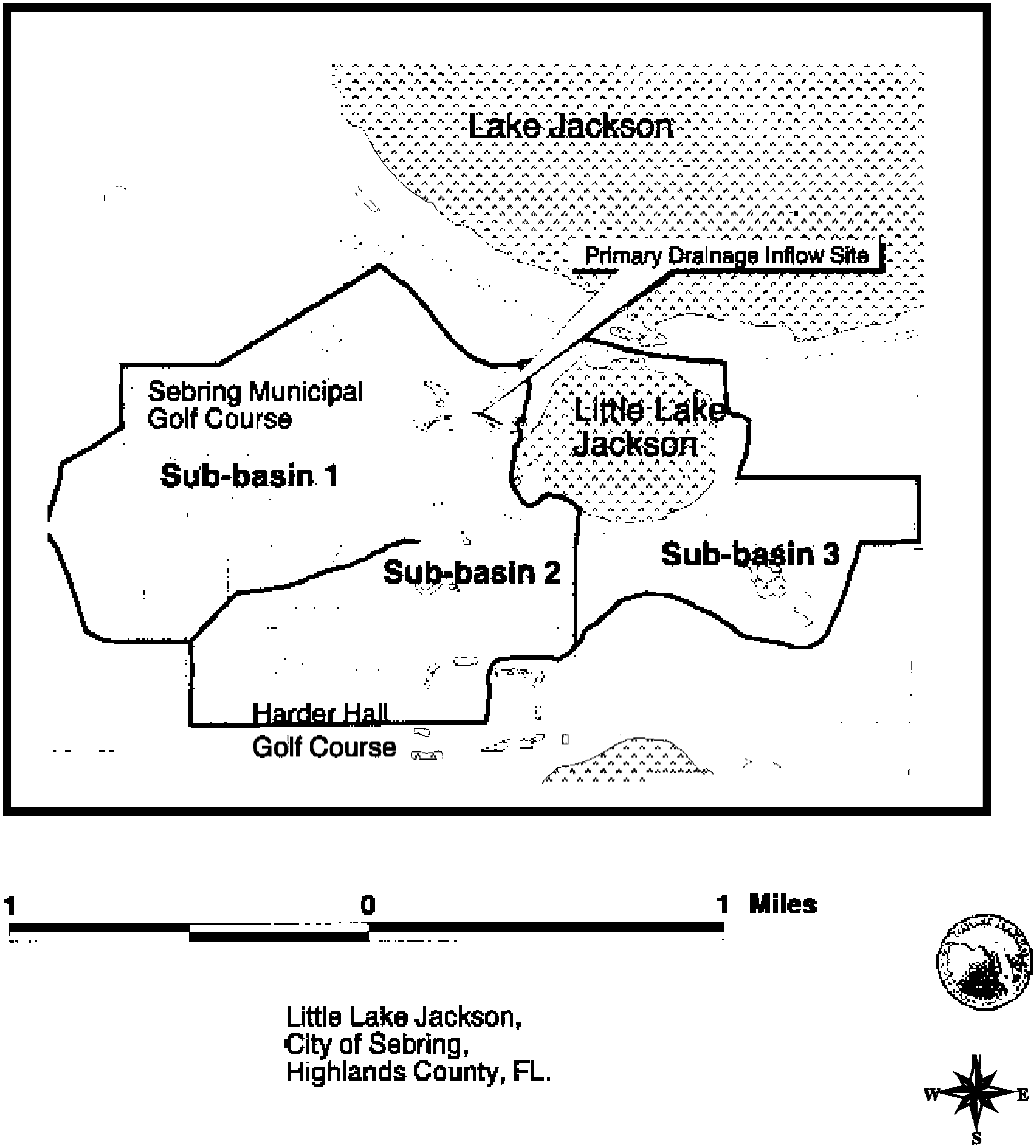
Daily rainfall totals have been collected since 1992 within the watershed by a local volunteer. The resident has generously donated his/her time as part of the volunteer data collection network of the District's Resource Data Department. In addition, during the study period, an electronic rainfall recorder was installed within Sub-basin 1, so that rainfall totals could be obtained at 15 minute intervals.

### Primary Inflow Hydrograph

**A** stream hydrograph can be used to characterize fluctuations of stream water level in response to runoff, which in turn is needed to establish a stormwater sampling procedure. **A** hydrograph was prepared for the primary inflow from water level readings collected from an automatic electronic elevation recorder (float and pulley), which was installed next to the drop pipe structure at the primary inflow station (station 1). The recorder was set to take automatic readings every thirty minutes. The deepest point on the invert of the drop pipe structure was used as the bottom depth reference point or the "zero" setting for the electronic recorder.

The stream stage hydrograph displayed storm peaks of long duration. For example, the upward slope or rise of the hydrograph caused by a single one inch rain event, occurred over approximately a **6** hour period. The downward slope of the hydrograph for this storm event lasted approximately 3 days. **As** a result of these peaks of long duration, grab sampling appeared to be feasible for this study. In contrast, grab sampling within flashy drainage systems is typically impractical, since sampling time windows are much shorter. Separate grab samples were routinely collected for base flow, which represent low flow or dry conditions. Base flow samples were verified by the examination of the inflow hydrograph. Samples were considered to represent base flow if they were collected during an extended period of stabilized stream levels. Four base flow samples were taken prior to the installation of the electronic elevation recorder. Since a hydrograph was unavailable

**Figure 2. Little Lake Jackson Drainage Sub-basins, Primary Drainage Inflow, and Inflow Sampling Site**



during this period these samples were verified by examining the daily rainfall record. Samples were considered to represent base flow if no rainfall was observed during the day of the sample collection, and the rainfall total was less than 0.20" for a total period of three days prior to the sample. Samples which could not be verified as base flow were eliminated from the data set. **A** total of nine qualifying base flow samples were collected,

**A** total of ten storm event samples were sampled, During three of these events, multiple samples were collected, so that different periods of the storm peak would be represented. Multiples samples are preferred since they provide a range of pollutant concentrations throughout a storm event. **A** single sample was collected during each of the remaining seven storm events. **All** rainfall data and all raw water quality data are available from the District if needed.

## RESULTS

**A** comparison of base flow nutrient concentrations and storm event concentrations is provided in Table 1. Little variation was observed between median base flow concentrations and median storm event concentrations for both organic nitrogen and total nitrogen (Table 1). Median and mean concentrations of ammonium, nitrite + nitrate, orthophosphorus, and total phosphorus were higher within storm event samples. Side-by-side boxplots of base flow concentrations and storm event concentrations are shown for each nutrient parameter in Figures 3A-3F.

Table 1. Summary of nutrient concentrations (mg/L) measured with base flow samples and within stormwater samples within the primary inflow of Little Lake Jackson, Sub-basin 1 (1995-1997).

| | $NH_4$ | $NO_3$ | ORG-N | TN | ORTH-P | **TP** |
|---|---|---|---|---|---|---|
| Base Flow | | | | | | |
| N | **9** | 9 | **8** | 8 | 9 | 9 |
| Mean | 0.137 | 0.087 | 1.40 | 1.63 | 0.152 | 0.172 |
| Median | 0.115 | 0.078 | 1.37 | 1.50 | 0.155 | **0.172** |
| Min | 0.029 | 0.004 | 0.685 | 0.88 | 0.102 | 0,123 |
| Max | **0.375** | 0.249 | 2.659 | 3.03 | 0.194 | 0.232 |
| Storm Events | | | | | | |
| N | 10 | 10 | 9 | 9 | 10 | 10 |
| Mean | 0.439 | 0.168 | 1.36 | 2.01 | 0.244 | 0.280 |
| Median | 0.315 | 0.120 | 1.31 | 1.81 | 0.210 | 0.250 |
| Min | 0.124 | 0.045 | 0.752 | 1.41 | 0.118 | 0,160 |
| Max | 1.72 | 0.569 | 2.15 | **3.22** | 0.650 | 0.701 |

Differences between base flow samples and storm event samples were greatest for ammonium, total nitrogen, orthophosphoms, and total phosphorus (Figures 3A,3D,3E, and 3F), while less variation was observed for organic nitrogen and nitrate + nitrite (Figures 3B and 3C). The Wilcoxon-Mann-Whitney rank-sum test (one-sided) was used to test whether storm event nutrient concentrations were significantly higher than base flow samples. Results showed that concentrations of ammonium, orthophosphorus, and total phosphorus during storm events were significantly greater ($a = 0.003$, 0.007, and 0.02, respectively) than concentrations measured within base flow samples, These differences were not found for nitrate, organic nitrogen, and total nitrogen (nitrate $\alpha = 0.135$, organic nitrogen $\alpha = 0.332$, total nitrogen $\alpha = 0.04$).

Nutrient concentrations of the primary inflow were compared to ambient concentrations of other flowing systems within the general local region, the Jackson-Josephine Drainage Canal and Little Charley Bowlegs Creek (Table 2). Orthophosphorus and total phosphorus concentrations measured within base flow and during most storm events (median storm) were similar to phosphorus concentrations measured within other systems (Table 2). Similarity between these sites suggests that phosphorus concentrations in these flowing systems may be associated with the surrounding soil types. **As** previously discussed (See Soil Types), black sands associated with cutthroat seeps and bayhead wetlands are common within the Little Lake Jackson watershed, Muck soils associated with bayhead wetlands are common along the Jackson-Josephine Drainage Canal and along the Little Charley Bowlegs Creek. Nutrients stored within muck soils may continually leach into surrounding groundwater, which in turn, may seep into surface water drainage ways. Phosphorus contributions from these types **of** soils appears to be a significant source of phosphorus within these systems. In addition, all of these systems are located within regions which frequently contain perched water tables. Dissolution of nutrients from these soils may increase when these soils become highly saturated.

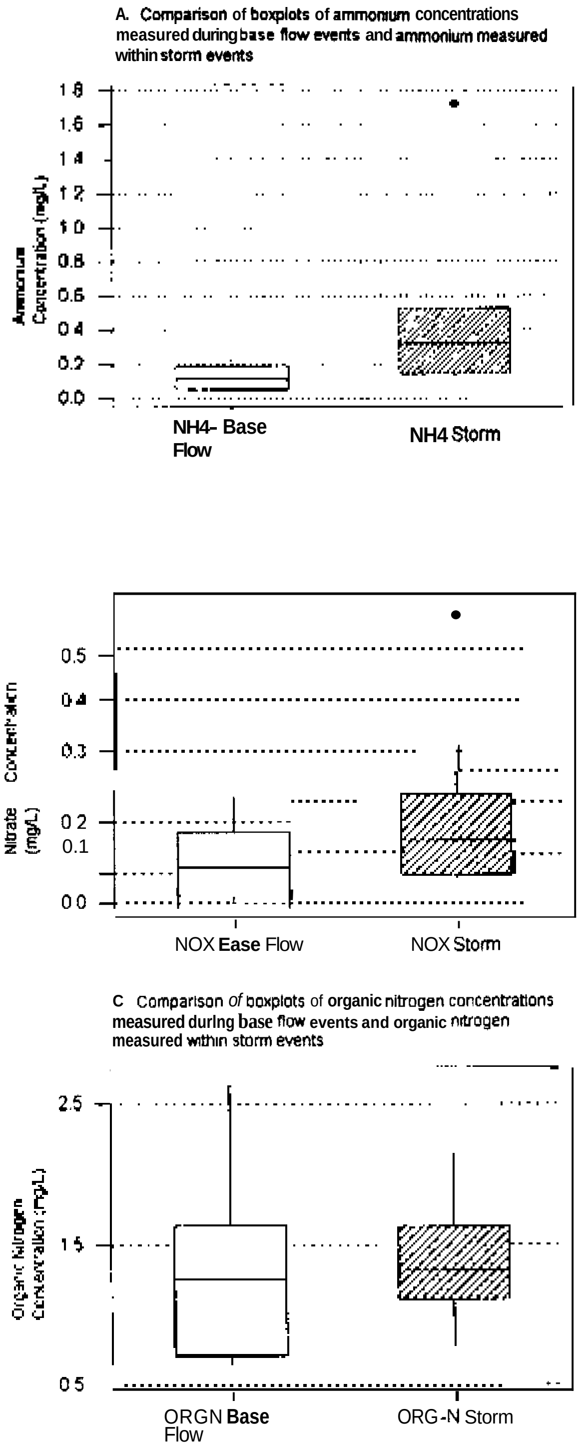
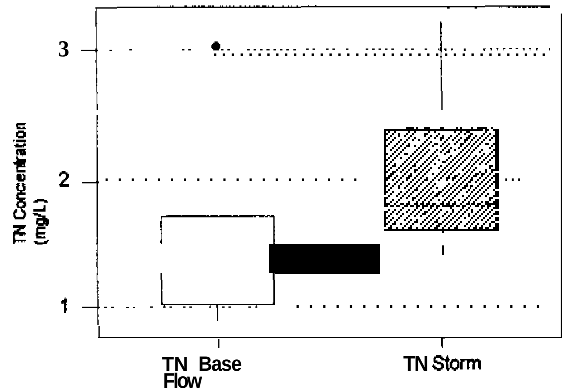
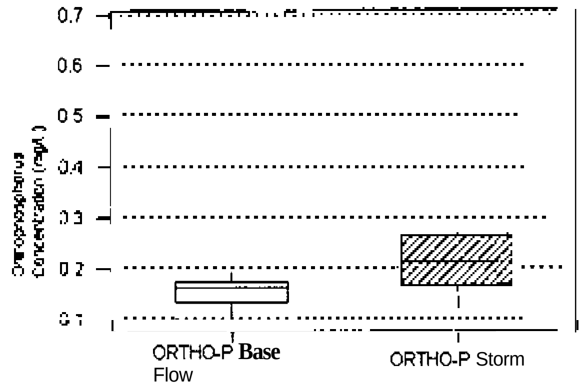



**Figure 3. Comparison** of **nutrient concentrations measured during base** flow **events to concentrations measured during storm events. Boxplots of ammonium (NH,) concentrations (3 A.), nitrate concentrations (3 B.), and organic nitrogen concentrations (3 C.) measured within base** flow **versus concentrations measured within storm events.**

D. Comparison of boxplots of TN Concentrations measured during base flow events and TN measured during storm events




E. Comparison of boxplots of orthophosphorus concentrations measured during base flow events and orthophosphorus measured within storm events




F. Comparison of boxplots of total phosphorus concentrations measured during base flow events and total phosphorus measured within storm *events*

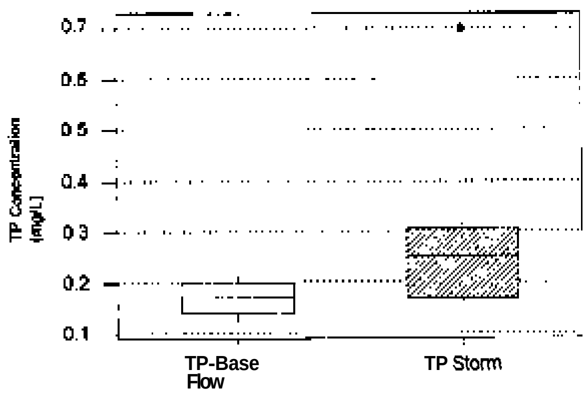



**Figure** 3.- Continued. Boxplots of total nitrogen concentrations (3 D.), orthophosphorus concentrations (3 E.), **and** total phosphorus concentrations (3 F.) measured within **base** flow **versus** concentrations **measured** within storm events.

Table 2. Comparison of median nutrient concentrations of the primary inflow drainage canal of Little Lake Jackson (Sub-basin 1) to concentrations measured within nearby flowing systems. Concentrations are expressed as mg/L.

| | N | ORTH-P | TP | $NH_4$ | TN |
|---|---|---|---|---|---|
| LITTLE LAKE JACKSON PRIMARY INFLOW | | | | | |
| Median Base Flow | 9 | 0.155 | 0.172 | 0.115 | 1.5 |
| Median Storm Event (1995-1997) | 10 | 0.210 | 0.250 | 0.315 | 1.8 |
| Maximum Storm Event Conc. (Mean for 10-08-96 event) | 2 | 0.650 | 0.701 | 1.725 | 3.2 |
| REGIONAL DATA | | | | | |
| Little Charley Bowlegs Creek Highlands Hammock State Park (1995) | 2 | 0.174 | 0.206 | 0.112 | 1.5 |
| Jackson-Josephine Canal Sparta Rd. Bridge (1995-1996) | 4 | 0.160 | 0.264 | 0.342 | 2.6 |
| Jackson-Josephine Canal Structure 2 (1995-1996) | 2 | 0.099 | 0.106 | 0.422 | 1.9 |

Although phosphorus measured within base flow and during most storm events appears normal for the region, concentrations measured during one of the heaviest storm events (event on 10-08-96) were unusually high (Table 2 - average orthophosphorus 0.650 mg/L, average total phosphorus 0.701 mg/L ). Elevated phosphorus concentrations carried within storm runoff, may be related to a number of factors, including both natural sources and anthropogenic sources. Higher concentrations of phosphorus are expected during heavy storm events due to factor such as increased erosion, however, other storm events of similar magnitude which were also sampled, yielded lower concentrations of phosphorus. Anthropogenic sources appear to be associated with these concentrations. The major anthropogenic sources within Sub-basin 1 are fertilizer and septic tank leachate. According to the Manager of the City Golf Course approximately five tons of fertilizer were applied to the course one day prior to the storm event in which these Concentrations were measured (event on 10-08-96). The timing of the fertilizer application suggests that it may have been the primary factor associated with the measured concentrations of this event.

Given the nature of the soils within Sub-basin 1, concentrations of ammonium measured within base flow samples of the primary inflow appear normal for the region. **As** displayed in Table 2, a median concentration of 0.115 mg/L for base flow concentrations was similar to the median concentration measured within the Little Charley Bowlegs Creek (0.112 mg/L) (Table 2). A median concentration of 0.315 mg/L for storm events may be somewhat typical of this region as well. The median ammonium concentration measured within sites within the Jackson-Josephine Canal ranged between 0.342 - 0.422 mg/L. Although these concentrations are somewhat high, soils within this region undoubtedly influence water quality within the Jackson-Josephine Canal. The canal was excavated through an area dominated by muck soils. Organic compounds which leach from muck soils and enter ground water or surface waters release ammonium and other forms of nitrogen as they decompose. Anaerobic conditions or conditions which promote decomposition were measured during most sampling events within this canal. These conditions included low dissolved oxygen concentrations, and elevated concentrations of iron and ammonium.

Unusually high ammonium concentrations (greater than 0.5 mg/L) were measured within three storm events within the primary inflow. Concentrations as high as 2.1 mg/L were measured during one of the heaviest storm events (3 inch rainfall event on 10-08-96)(Table 2). The maximum ammonium concentration of the U.S. EPA (1986) chronic-exposure criteria for natural surface waters within normal ranges of pH and temperature is 2.1 mg/L. Concentrations of this magnitude suggest that anthropogenic sources were definite contributors of ammonium loading during this event. Like phosphorus, anthropogenic sources of ammonium within Sub-basin 1 primarily include fertilizer and septic effluent, both of which probably contribute to ammonium loading within this sub-basin. Although ammonium may be released naturally from soil organic matter through such processes as decay and dissolution, these natural soil decay processes usually occur gradually (SWFWMD 1997). As previously discussed, a large quantity of fertilizer was applied within the City Golf Course one day prior to the event in which these concentrations were measured (event on 10-08-96). The timing of the fertilizer application suggests that it was a primary factor associated with the measured concentrations of this event.

## Discharge Curve and Discharge Hydrograph

The water levels displayed by a hydrograph are ultimately used to determine a stream discharge curve, which in turn, is used to calculate nutrient loads. Determining a stage discharge curve for the primary inflow ditch within Sub-basin 1 was challenging due to obstructions within this conveyance system. Thick vegetation (paragrass) located downstream of the drop pipe structure blocked stream flow. Culverts within this region were clogged with sediment making standard discharge calculations difficult. The drop pipe structure located to the west of Golf View Road has settled over the years and has shifted to an approximate 80° angle with the water surface. The two sets of slotted boards within the structure have shifted as well, causing the structure to change from a rectangular shaped weir to a structure with two separate triangle shaped outlets. Obstructions downstream often caused tail-water conditions at the structure. Accurate discharge rates could not be obtained by applying a standard weir discharge equation to this structure.

A stage discharge curve was created for the primary inflow by measuring stream velocity and stream area throughout a range of flow at the drop-pipe structure. Stream velocity was measured with a Marsh McBirney ® water current meter (Model 201D). Discharge rates (cubic feet per second) were calculated by multiplying stream area by stream velocity. Since downstream vegetation was inhibiting normal flow and creating tail-water conditions at this structure, nuisance vegetation had to be removed prior to measurement of velocity recordings. Fortunately, personnel from the Highlands County Aquatic Weed Control volunteered to spray the nuisance vegetation with aquatic herbicides. Free-fall conditions were generally achieved at the structure once the invasive vegetation was removed.

The discharge rating curve was calculated by plotting discharge rates against stream depth at the invert structure and then finding a predictive equation which appeared to have the best fit for these plots, Hourly electronic water elevation recordings were then applied to the predictive equations so that hourly discharge rates could be calculated. Hourly discharge rates were then used to calculate daily discharge rate averages. The maximum average daily discharge rate in stormwater samples were collected was approximately **7** cfs (event of 10-08-96). Average daily base flow discharge dropped as low as 0.1 cfs.

**Relationships between Nutrient Concentrations and Discharge**

Significant correlations (Spearman correlation) were observed between discharge (log transformed) and the following nutrient concentrations (log transformed) measured within both base flow events and storm events: discharge and orthophosphorus rho=0.855, p=0.001; discharge and total phosphorus rho=0.893, p=0.001, discharge and ammonium rho=0.828, p=0.001. Plots of discharge and concentrations of orthophosphorus, total phosphorus, and ammonium are shown in Figures 4A., 4B., and 4C., respectively.

**Nutrient Load Calculations**

Hourly discharge rates were used to calculate total hourly runoff volumes. Average daily runoff volumes were then calculated and used to determine daily nutrient loads by multiplying the daily discharge volume by the appropriate nutrient concentrations. Two different nutrient concentrations were applied to the discharge curve. The median base flow nutrient concentration was applied to days which were characterized as base flow discharge. The median storm event nutrient concentration was used to calculate nutrient loading for all days which were characterized as storm or post storm discharge. **A** total yearly load was calculated from summing these daily loads. A **summary** of yearly nutrient loads (April 1996 - April 1997) is provided in Table 3a.

Separate nutrient loads were also calculated for the storm event in which the peak nutrient concentrations were measured (Oct **8,** 1996, **3**" rain event). In addition, based on this additional loading, a new total annual load was calculated (Table 3b). Most nutrient concentrations measured during this particular storm were shown **as** outliers within boxplots of storm event nutrient data (Figure 3A, 3B, 3E, and **3F).** Separate nutrient loads were calculated for this particular event by using the concentrations and flows specific to this event (Table 3b). The loads for ammonium, nitrite + nitrate, orthophosphorus, and total phosphorus comprised large portions of the total yearly load (Table 3b). The ammonium load was the largest, comprising 45 percent of the annual load (Table 3b). When this event was included within the calculations of the total yearly load, a 51 percent increase in yearly load was observed for ammonium. The total yearly load for orthophosphorus increased by approximately 17 percent, while both the total yearly load for nitrate +nitrite **and** total phosphorus increased by approximately 15 percent. **A** percentage of the total yearly load was calculated for this separate storm event (Table 3b).

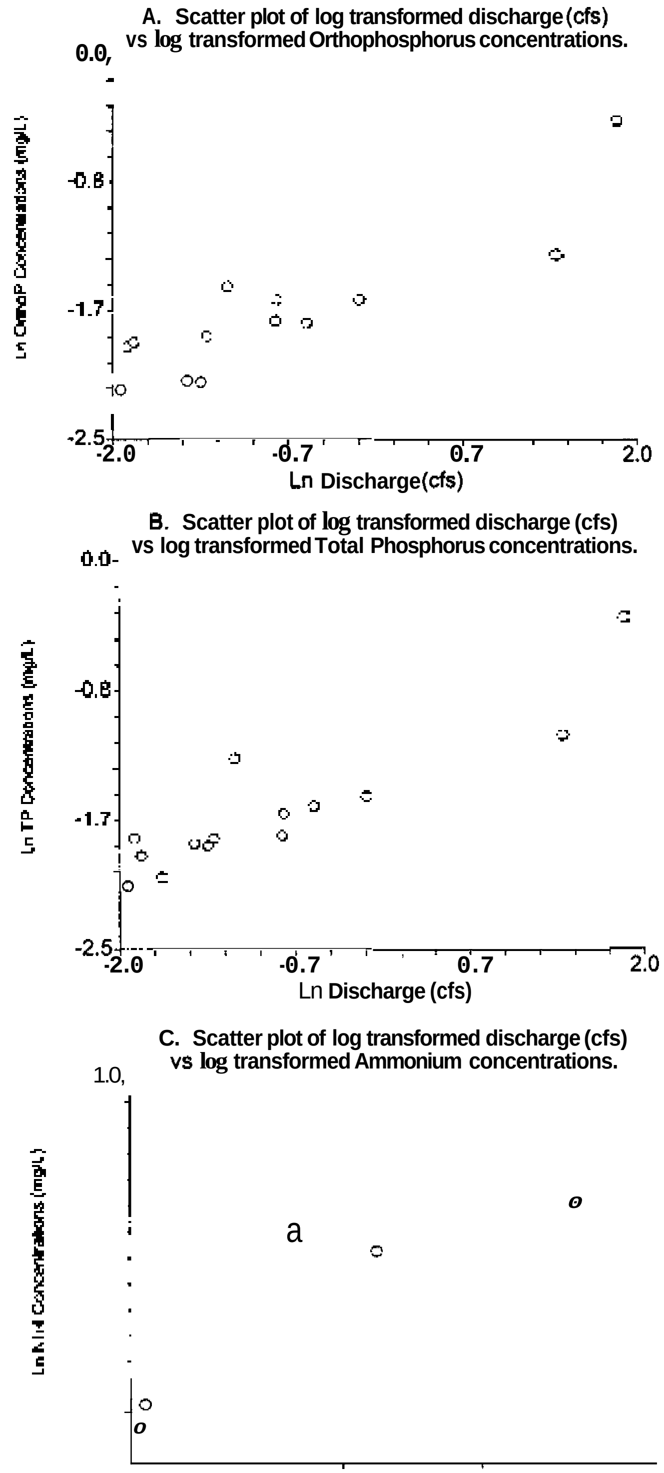



**Figure 4. Scatter plots of log (ln) transformed discharge and log transformed nutrient concentrations (A. Orthophosphorus, B. Total phosphorus, C. Ammonium). Nutrient data are from both base-flow and storm event samples.**

Table 3a. Nutrient loads calculated for the primary inflow to Little Lake Jackson, Sub-basin 1, based on the median base flow concentration and the median storm event concentrations. Separate loads and percentages are shown for base flow and storm events. Total loads are reported as kg/year, lbs/year, kg/acre/year, and lbs/acre/year.

| | $NH_4$ | $NO_3$ | ORG-N | TN | ORTH-P | TP |
|---|---|---|---|---|---|---|
| Base Flow (kg) | 7.2 | 7.3 | 151 | 160 | 14.3 | 16.2 |
| Percentage | 21.7 | 43.4 | 60.2 | 53.8 | 44.9 | 44.4 |
| Storm Events **(kg)** | 25.9 | 9.6 | 99.9 | 137.3 | 17.6 | 20.3 |
| Percentage | 78.3 | 56.6 | 39.8 | 46.2 | 55.1 | ***55.6*** |
| Total kg/year | 33.1 | 16.9 | 251.0 | 297.2 | 31.9 | 36.5 |
| lbs/year | **73.0** | 37.3 | 553.3 | 655.2 | 70.4 | 80.4 |

Table 3b. Nutrient loads calculated for peak storm event on Oct. 8, 1996 (containing maximum nutrient concentrations) for the primary inflow to Little Lake Jackson, Sub-basin 1. The overall loads for all base flow events and all storm events were recalculated to include the peak storm event.

| | $NH_4$ | $NO_3$ | ORG-N | TN | ORTH-P | TP |
|---|---|---|---|---|---|---|
| Storm Event Containing Peak Conc. | 23.3 | 4.2 | 23.6 | 42.7 | 8.3 | 9.0 |
| of $NH_4$, TN, ORTH-P, and TP | 44.9 | 21.5 | 9.2 | 13.6 | 22.2 | 21.4 |
| Base Flow (kg) | 14.5 | 9.9 | 173.8 | 190.3 | 19.6 | 21.8 |
| Percentage | 27.9 | 50.8 | 67.9 | 60.6 | 52.6 | 51.9 |
| Storm Events (kg) | 37.4 | 9.5 | 82.2 | 123.8 | 17.6 | 20.2 |
| Percentage | 72.0 | 49.2 | 32.1 | 39.4 | 47.4 | 48.1 |
| Total kg/year | 51.9 | 19.4 | 256.0 | 314.1 | 37.2 | 42.0 |
| lbs/year | 115 | 42.8 | 564.3 | 692.6 | 82.1 | 92.3 |
| kg/acre/year | 0.10 | 0.04 | 0.52 | 0.63 | 0.07 | 0.08 |
| lbs/acre/year | 0.22 | 0.09 | 1.2 | 1.4 | 0.17 | 0.18 |

**As** previously discussed, the peak nutrient concentrations measured during the storm event on Oct. **8,** 1996 (3" rain event), appear to be primarily associated with fertilizer application within the Sebring Municipal Golf Course. Approximately, five tons of fertilizer (15-5-1*5)* was applied over the 100 acre golf course one day prior to the large rain event (Oct 8, 1996, 3" rain event). Although the golf course incorporates weather forecasts into their fertilizer application procedure, this large rain event was not forecasted. The timing of their fertilizer application suggests that golf course fertilizer was a significant contributor of nutrients during this particular storm event. Approximately 1500 lbs of total nitrogen, 750 lbs of ammonium, and 187 lbs of nitrate was distributed over the golf course. The total nitrogen load estimated from the discharge hydrograph (Table 4b) for this event for the primary surface water inflow was approximately 94.1 lbs. The ammonium load estimated from the discharge hydrograph was approximately 51.4 lbs. The nitrate load estimates from the discharge hydrograph was approximately 10 lbs. for this particular storm event.

Approximately 500 lbs of phosphorus was applied over the City Golf Course during the application. The estimated total phosphorus load and orthophosphorus load delivered to the lake through the primary inflow were approximately 20 lbs. (estimation from stage discharge curve).

## DISCUSSION

### Estimated Nutrient Loads to Land Surface Based on Land Use Information

Although fertilizer application within the City Golf Course was suggested as a major nutrient contributor during one the heaviest storm events, land use data suggested that residential fertilizer may be the largest nutrient contributor. Estimates were prepared for nutrient loads applied to the land surface from potential nutrient sources within Sub-basin 1 (Table **4).** These sources included septic effluent, residential fertilizer, golf course fertilizer, and rainfall. These were prepared by applying general annual loading rates acquired from literature for each specific nutrient sources and then applying those to the appropriate land use data for Sub-basin 1. Some of the literature from which general loading rates are obtained are noted in Table 5. Residential fertilizer application appears to have the greatest potential for applying the largest nitrogen loads. At least one half of the nitrogen applied to the land surface within fertilizer is probably delivered as ammonium nitrogen, which is typically the dominant form of nitrogen within most common fertilizers. Like nitrogen, the primary source of phosphorus appears to be fertilizer, with fertilizer application within residential land use as the greatest potential contributor (Table 4).

Although calculations of generalized nutrient loads suggest that residential fertilizer is the largest contributor of nitrogen and phosphorus, water quality **and** discharge data collected from within the primary inflow suggests that fertilizer application within the golf courses poses a greater threat for bulk nutrient loads carried within surface water runoff. Unlike the residential areas, fertilizer application within the golf course is performed as a bulk application over a large area during a narrow time frame. **As** suggested from data collected within this study, the risk of large nutrient loads is of course greatest when applications are applied prior to large rain events.

Table 4. Estimated annual nutrient loads to land surface for Sub-basin 1 within Little Lake Jackson watershed, Highlands County, Florida. Loads shown are for direct land surface application rates.

| | TN (tons/yr) | $NH_4$ (tons/yr) | $NO_3$ (tons/yr) | TP (tons/yr) |
|---|---|---|---|---|
| Septic Effluent [1] | 4.7-6.7 | 0-6.7 | 0-4.7 | 1.6 |
| Residential Fertilizer [2] | 14.3-24.4 | 7-12 | 1.8-3.0 | 3.2-5.2 |
| Golf Course Fertilizer [3] | 8.2 | 4.1 | 1.0 | 2.7 |
| Rainfall [4] | 1.7 | 0.2 | 0.7 | 0.1 |
| TOTAL | 29-41 | 11-23 | 3.5-9.4 | 13.5-17.5 |

[1] - (Canter and Knox 1985, Cogger and Calie 1984, Henigar and Ray 1990, **Sikora** *et al.* 1976, SWFWMD 1990, SWFWMD 1990)
[2] - (Florida Cooperative Extension Service **1990)**
[3] - (Personal communication with staff **of the** Sebring Municipal **Golf** Course)
[4] - (SWFWMD Laboratory - unpublished data)

Although anecdotal evidence (personal communication with the Golf Course Manager) suggested that golf course fertilizer appeared to be a large nutrient contributor during the one the heaviest storm events (10-08-96) anecdotal evidence is not definite. In addition, since other storm events of similar magnitude were sampled, it appears that high nutrient loads that occurred on 10-08-**96** should be treated as an isolated event. Determining the association of golf course fertilizer with this separate event does not imply that golf course fertilizer is the dominant source of nutrients within all other storm events, or within base flow.

During the course of the study an idea was developed for a stormwater sampling protocol which may be helpful for distinguishing between inorganic nitrogen sources within runoff samples collected within this watershed, or possibly for other watersheds. The sampling protocol would basically involve first performing field testing runoff samples for concentrations of nitrate and ammonium. Either **an** electronic probe could be used (Hydrolab or YSI® ) or a field test kit such as a Hach® kit specific for ammonium and nitrate. If unusually high concentrations of nitrate or ammonium were detected, then additional samples would be collected for analyses of stable nitrogen isotopes ($\delta^{15}N_{AIR}$). Stable nitrogen isotope tests are helpful for distinguishing between inorganic sources such as fertilizer, and organic sources of nitrogen such as septic effluent or animal wastes. If the tests indicate that inorganic sources such as fertilizer are involved then fertilizer application schedules should be closely evaluated. These tests when combined with reviewing copies of the golf course fertilizer schedules would provide a strong indication of whether or not golf course fertilizer was a major contributor. Reviewing the fertilizer schedule is essential since residential fertilizer is also applied within this sub-basin. The absence of fertilizer application within the golf course may indicate residential fertilizer as the primary nutrient contributor. Distinguishing between these major nutrient contributors would be helpful so that target BMP's could be developed for this watershed.

## ACKNOWLEDGMENTS

This study was funded as part of the overall greater Little Lake Jackson Diagnostic Feasibility Study, which was a cooperative funding project between the Peace River Basin Board of the Southwest Florida Water Management District, Highlands County, and the City of Sebring. District staff from several different departments have assisted with the various components of this project, such as the water sample analyses provided by staff of District's chemistry laboratory. Bob Hoffman (City Administrator) from the City of Sebring and Lisa Burley from Highland County have provided contract management and administrative support. Special thanks goes to Jim Higgins (City Golf Course Manager) for his dedicated work and invaluable information provided during the study.

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# REMOVAL OF GROSS POLLUTANTS FROM STORMWATER RUNOFF USING LIQUID/SOLID SEPARATION STRUCTURES

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## ABSTRACT

Gross pollutants in stormwater runoff generally consist of litter, debris and coarse sediments. Litter is typically defined as human-derived material, including paper, plastic, metal, glass and cloth. Debris is typically defined as any organic material transported by stormwater runoff, such as leaves, twigs and grass clippings. Coarse sediments are defined as inorganic particulates. The discharge of gross pollutants to surface waters can threaten wildlife and aquatic habitats, can produce unpleasant odors and attract vermin, and can be aesthetically unpleasing. Many gross pollutants cannot be sampled by traditional automatic samplers and, as a result, gross pollutants are often overlooked when evaluating the impact of stormwater runoff on receiving waters.

During **1998-1999,** evaluations were conducted for the City of Orlando, the City of Winter Haven, and the City of Atlantic Beach related to the removal of gross pollutants from stormwater runoff using liquid/solid separator technologies. Based on information found in the literature and information obtained from technology manufacturers, removal efficiencies were compared for four separate liquid/solid separator technologies, including Vortechnics, Stormceptor, CDS Technologies, and traditional baffle boxes. The evaluation considered removal efficiencies for litter, debris, and coarse sediments; estimated installed cost; and operation and maintenance requirements. Based on removal efficiencies for coarse sediments, removal efficiencies were estimated for common stormwater constituents, including total nitrogen, total phosphorus, total suspended solids, BOD, and heavy metals. Based on typical fractions of particulate matter in runoff, liquidlsolid separators are capable of removing approximately 20-50% of nutrients and heavy metals under ideal conditions. Limitations of liquidlsolid separators must be understood when considering these systems for retrofit applications.

While performing the evaluations, it became apparent that there is insufficient field data to accurately predict the removal efficiencies for various gross pollutants contained in stormwater runoff in the United States. Additional field studies should be performed to develop accurate removal efficiencies for liquid/solid separator technologies.

# REMOVAL OF MICROBIAL INDICATORS FROM STORMWATER USING SAND FILTRATION, WET DETENTION, AND ALUM TREATMENT BEST MANAGEMENT PRACTICES

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## ABSTRACT

Stormwater runoff is often contaminated by a number of microbial pathogens and can contribute significantly to the degradation of valuable water resources. Several best management practices (BMPs) are used throughout the U.S. for stormwater treatment, however, little research has evaluated their effectiveness for the removal of microorganisms. In this study, indicators and surrogates of microbial pathogens (total and fecal coliform bacteria, MS2 coliphage, and a **3** µm fluorescent bead representing the pathogenic protozoa, ***Cryptosporidium parvum)*** were used to challenge sand filtration, wet detention, and alum coagulation treatment systems using simulated storm events. Significant ($p < 0.05$) reductions in total and fecal coliform bacteria, MS2, and bead concentrations were observed between inflow and outflow samples for each of the BMPs. On a few occasions, however, concentrations of total coliform bacteria, turbidity, and total suspended solids were greater in outflow samples than inflow samples. Using flow-weighted sampling techniques, load reduction estimates were calculated for each treatment system. Removal efficiencies for beads were consistently greater than 90% while MS2 coliphage removal was consistently greater than **80%** for all three treatment systems. Removal efficiencies for total and fecal coliform bacteria varied widely with total coliform removal values consistently less than 70% while fecal coliform values ranged from ***65*** to 100%. Removal efficiencies using sand filtration were generally high for turbidity, MS2, and beads but not for total or fecal coliforms. Wet detention using the current regulatory standard of a 5-day bleed-down period provided consistently high removal efficiencies for fecal coliform bacteria, **MS2** and beads and had the greatest TSS removal of the three treatment systems. Overall, alum coagulation (dose = 10 mg/L) provided greatest removal efficiencies for total and fecal coliform bacteria, **MS2** coliphage, and turbidity under semi-controlled conditions using jar tests. Recommendations for optimizing current stormwater treatment systems for the removal of microorganisms are addressed and include the use of a multiple treatment (treatment train) approach,

## INTRODUCTION

Research evaluating pollutant removal efficiencies for stormwater treatment systems (best management practices or BMPs) have focused primarily on physical and chemical contaminants such as total suspended solids, nutrients, and metals (Urbonas, 1994). Relatively little information has been collected regarding treatment efficiencies of BMPs for the removal of microbial pathogens (O'Shea and Field, 1992) - organisms known to be present in stormwater (Qureshi and Dutka; 1979) and which pose serious health risks to high-risk groups including the elderly and immunocompromised.

To date, little research has been conducted to evaluate the effectiveness of current regulatory criteria for stormwater treatment systems in the removal of human microbial pathogens. This information will become more critical as several alternative sources of drinking water are developed in Florida including the diversion and storage of stormwater runoff and treated wastewater to recharge depleted aquifers, rivers, and lakes (Bishop, 1992; SWFWMD, 1995). In Florida, state regulations (Chapter 17-40, Florida Administrative Code [F.A.C.]) recommend that stormwater treatment systems achieve an annual average of **80** percent pollutant load reduction. This standard is based primarily on the removal of heavy metals and nutrients (nitrogen and phosphorous) and does not specifically address microbial pathogens, Although standards for bacterial indicators (total and fecal coliforms) exist for surface waters, there are no maximum contaminant levels for a wide range of specific waterborne pathogens including other species of bacteria (***Clostridium***, ***E. coli, Salmonella***, ***Klebsiella***), viruses (hepatitis a, Coxsackie, rotavirus), and protozoa (*Cryptosporidium*, *Giardia*) that can cause human disease.

This study was conducted to determine removal efficiencies for bacteria, viruses, and protozoa using three stormwater treatment technologies used in Florida: **an** above-ground sand filter, a wet detention pond, and alum coagulation. Indicator organisms (total and fecal coliforms, coliphage, and fluorescent beads representing ***Cryptosporidium*** oocysts) were used as surrogates for the broad spectrum of human pathogens which may be present in stormwater. Removal efficiencies were calculated based on comparisons of total inflow and outflow loads of seeded microbial indicators. Effluent concentrations for total and fecal coliform bacteria and turbidity (no standards exist for coliphage or protozoa) were compared with Florida's Surface Water Quality Standards (Chapter 62-302) to determine the extent each BMP could treat stormwater to meet regulatory goals.

## METHODS

Three BMPs located in the Tampa Bay area were evaluated during this study and included an above ground sand filter, a wet detention pond, and **an** alum injection system. The sand filter stormwater treatment facility treats runoff from a 2.73 ha (6.75 ac) light commercial/urban drainage basin in the city of Madeira Beach. Treatment occurs as the stormwater percolates through one of three rectangular sand filter chambers each composed of approximately 1 m of clean creek gravel and sand. After traveling through a gravel underdrain and perforated drainage pipe, the treated stormwater is discharged to an adjacent residential canal which is tidally connected to Boca Ciega Bay. Three seeded trials were performed between September 1995 and November 1996. During each sampling event, approximately 5,160 L (1,365 gal.) of residual stormwater which had accumulated in the holding tank were pumped onto one of the three sand filters. Adjustable weirs were installed so that stormwater could be diverted to any or all three of the filters. Prior to the first trial (Trial 1), the northernmost filter bed had been isolated from stormwater inflows and was challenged in an unsaturated condition. The filter was then left open and challenged several months later (Trial 2) in a saturated condition (a few hours after being used to treat unseeded stormwater). The middle bed was later challenged once (Trial 3) in a saturated condition.

The second BMP consisted of two experimental wet detention ponds originally constructed at the Southwest Florida Water Management District's (SWFWMD) Tampa Service Office to evaluate pollutant removal efficiencies of chemical constituents using conventional wet detention methods (Cunningham, 1993). Two 0.06 ha (0.15 ac) ponds with depths of 1 m (3.3 ft) and 2.75 m (9.0 ft) were constructed to meet Chapter 40D-4, Basis of Review, Florida Administrative Code guidelines to compare the effect of pond depth on stormwater treatment efficiency. In order to simulate various storm events, water was pumped from the Tampa Bypass Canal into the western end of each pond through a series of 10 cm diameter underground PVC pipes. Vertical pipes were used to drain the ponds at the outfall. Two separate trials were performed representing different treatment or bleed down periods. In the first trial, a 14-day residence time was achieved by adjusting the outflow valve so that the initial outflow rate was limited to approximately 13 L/min. In the second trial, a 5-day residence time was simulated by increasing the discharge flow rate to 36 L/min.

The third BMP involved alum coagulation jar tests carried out using stormwater collected from drainage ditches which collected runoff from two heavily urbanized watersheds. Bench-scale jar tests were employed during two separate trials which included a relatively high dose of alum (600 mg/L) versus a lower dose typically employed for stormwater treatment (10 mg/L). The first collection site was in Pinellas Park, Florida, upstream of an existing in-line alum treatment system consisting of an alum injection system and downstream settling pond. The point of collection was located at a channel which drains an approximately 33 ha (83 ac) residential/light commercial watershed. An average dose of 10 mg/L concentration of aluminum sulfate ($Al_2(SO_4)_3 \cdot 18H_2O$) was determined by previous jar tests to be optimal for pollutant removal at the site (ERD, Inc., 1995).

Approximately 16 L of stormwater was pumped into each of four (4) 20 L capacity plastic containers from a collection point upstream of a settling pond and existing alum injection system. Three of the four containers were dosed with 160 ml of industrial-grade liquid alum to simulate a high dose treatment of approximately 600 mg/L concentration. The fourth container was used as a control to measure the effects of natural die-off and settling of the microbial indicators. A second trial was performed using water sampled from a large creek (Hamilton Creek) draining an 184 ha (460 ac) urbanized watershed in downtown Tampa near the Lowry Park Zoo. During this trial, a lower dose of alum was added at a concentration of 10 mg/L expressed as $Al_2O_3$.

During each of the three BMP challenges, high titers of MS2 coliphage and 3.0 µm fluorescent beads were simultaneously mixed with raw stormwater. The concentrations of beads and viruses used for seeding experiments were adjusted to ensure that adequate numbers of each surrogate could be recovered for analysis using a relatively small outflow sample volume. Total and fecal coliforms were not seeded during any trial since background concentrations were sufficiently elevated for influent-effluent comparisons. Total coliforms, fecal coliforms, MS2 coliphage, fluorescent beads, and turbidity were all measured in the inflow and then at evenly-spaced intervals during the drawdown period in the outflow. Total inflow loads were calculated based on the total volume challenged in each system times the average concentration of each parameter. Outflow loads were calculated by summing each of the outflow sample concentrations times the corresponding volume discharged between sampling events.

Temperature (°C), pH (s.u.), and conductivity (µS/cm) were measured *in-situ* using a Hydrolab®. Aqueous samples were analyzed according to Standard Methods for the Examination of Water and Wastewater (APHA, **1995).** Total and fecal coliform samples were collected in sterile 500 ml Nalgene® bottles and analyzed within ***6*** hours using the membrane filtration method. Several serial dilutions were filtered to ensure that a valid colony count could be expressed numerically. Too numerous to count (TNTC) results were not acceptable since removal efficiencies could not be calculated using a non-numerical value. Confirmation of total and fecal coliform colonies were made using the Enterotube® multitest system (Roche Bioscience), an accepted methodology by the Florida Department of Health (M. Rials, pers. comm.). MS2 and bead samples were collected in sterile 50 ml polyethylene tubes and analyzed at the Department of Marine Science lab on the University of South Florida campus in St. Petersburg. All samples were stored on ice prior to analysis.

The fluorescent latex beads (Fluoresbrite® beads, Polysciences, Inc.) used as surrogates to model the transport and fate of *Cryptosporidium* spp. were enumerated using the methods described by Paul et al. (1**995)** in their investigation of on-site sewage disposal systems in the Florida Keys. The beads used in this study were similar in size ($3.0 \pm 0.1\,\mu m$ in diameter) and density to *C. parvum*, were relatively inert in aqueous solutions, and have been used as tracers in both environmental contamination assessments (Harvey, 1989; Paul et al., 1995; Dr. Joan Rose, University of South Florida, pers. comm.) and cytometry studies.

Geometric means were calculated for all inflow **and** outflow values for microbial indicators. Arithmetic means were calculated for all physical parameters. Log removal values were calculated based on log concentration differences between inflow and outflow samples. Removal efficiencies were calculated using mass balance equations for each of the four indicators and **TSS.** For turbidity, removal efficiency was calculated by the difference between mean inflow and outflow concentrations, dividing by the inflow concentration, and multiplying by 100%. Loading values were known for fluorescent beads since none of the treatment systems had been exposed to this tracer prior to the study. For MS2, background samples were collected from the source water prior to seeding to determine ambient coliphage concentrations. The geometric mean of these values was then multiplied by the total volume of water entering the treatment system to estimate an ambient loading value which was then added to the known seed load to calculate a total inflow load. Effluent concentrations were also compared with Florida's Surface Water Quality Standards to determine if individual treatment methods could meet regulatory standards. Results of these comparisons are presented as the percent of samples which exceeded the water quality standard.

Whenever possible, parametric statistics (ANOVA) were used to compare concentrations between inflow and outflow samples for each of the microbiological indicators. Due to wider than expected variations in bacterial and coliphage concentrations, non-parametric analyses (Kruskal-Wallis Test) were used in cases where the assumption of homogeneity of variance could not be met even after log transformation of the data. Post-hoc comparisons were made using either the Kruskal Wallis Z test or Fisher's LSD test, depending on whether non-parametric or parametric analyses were used, respectively. Correlation analyses were performed using simple linear regression. All comparisons were considered significant at the ***95%*** confidence level and were analyzed using NCSS® software.

## RESULTS

### Sand Filtration

Removal efficiencies for the microbial indicators ranged from 59.4% to 99.5% and were greatest for the ***Cryptosporidium*** surrogate (fluorescent bead), followed by **MS2** coliphage, fecal coliforms (FC), and total coliforms (TC) (Table 1). Differences between removal efficiencies for the four indicators were not significant ($p > 0.05$). For each trial, concentrations of the four microbial indicators and turbidity were all significantly ($p \leq 0.05$) lower in the outflow than in the inflow except for total coliform comparisons in Trial **3.**

Table 1. Mean concentrations, log removal (based on concentrations), and removal efficiencies (based on loads) **for** indicator and physical parameters **from** the sand filter treatment system challenge. Data from all three trials were used for comparisons.

| Parameter | Inflow | Outflow | Mean Log Removal | Load Removal Efficiency |
|---|---|---|---|---|
| Turbidity (NTU) | **15.70** | 2.76* | | **82.4%** |
| **TSS (mg/L)**** | 19.27 | **5.63*** | | **7** 1.0% |
| Total coliforms (cfu/100 ml) | **2.44** x $10^4$ | **4.24** x $10^3$ | **0.88** | 59.4% |
| Fecal coliforms (cfu/100 ml) | 1.19 x $10^4$ | **1.19** x $10^3$* | 1.01 | **65.4%** |
| **MS2** coliphage (pfu/ml) | 2.10 x $10^5$ | 2.00 x $10^3$* | **2.02** | **87.7%** |
| *3* µm beads ***(Cryptosporidium*** surrogate) (beads/ml) | 1.94 x $10^5$ | **5.22** x $10^1$* | 3.57 | **99.5%** |

*statistically significant difference at the 95% confidence level between log transformed inflow and outflow concentrations. ** based on a single trial with multiple replicates.

Trends in TC concentrations were similar during Trials 1 and 2. Elevated values in the inflow (TO) generally (except for a few samples at the start and end of the treatment period) decreased below the Class III one-day maximum value of 2,400 cfu/100 ml in the outflow. During Trial 3, TC concentrations decreased only slightly after filtration and remained elevated above the Class III maximum during the entire treatment period, FC bacteria trends were nearly identical to TC values with the exception of fewer values exceeding the Class III maximum. A number of gram-negative bacteria were identified in both the inflow and outflow samples taken from the sand filter including several which are capable of causing human disease *(E. coli, Klebsiella pneumoniae,* and ***Salmonella enteritidis).*** None of the various bacterial species appeared to be removed differentially since most were present in both the inflow and outflow samples. *K. pneumoniae* was the most ubiquitous species and was found in both inflow and nine of ten outflow samples.

Turbidity and TSS values were elevated in all inflow samples and were reduced significantly during treatment. Trends in turbidity indicated relatively rapid removal except in Trial **3,** where a spike in turbidity and **TSS** occurred in the first outflow sample. Turbidity values in outflow samples during Trial 3 were greater than in Trials 1 and **2** despite having similar inflow concentrations. Log-transformed concentrations of TC ($r^2 = 70.1\%$), FC ($r^2 = 80.3\%$), MS2 ($r^2 = 42.4\%$), and fluorescent beads ($r^2 = 42.6\%$) were all positively correlated ($p \leq 0.05$) with turbidity.

Surface water quality standards ($\leq$ 29 NTU above background conditions) for turbidity in Class III waters were exceeded in only a single outflow grab sample but were never exceeded in any other inflow or outflow sample. TC and FC bacteria concentrations exceeded the Class III maximum value at the inflow (raw stormwater) during every trial. Outflow concentrations for TC exceeded the Class III (recreational waters: < 2,400 cfu/100 ml) one day maximum value in **43%** of all outflow samples. Outflow concentrations for FC exceeded the Class III (< 800 cfu/100 ml) one day maximum value in 40% of all outflow samples (Fig. 11). When analyzed by sand saturation conditions, TC exceeded Class III standards in 65% of outflow samples using a saturated sand filter and 0% using an unsaturated filter. FC concentrations were exceeded in ***55%*** of outflow samples in saturated filter conditions and 10% in unsaturated conditions. Of the **six** parameters, only turbidity, MS2 and the ***Cryptosporidium*** surrogate were reduced sufficiently to meet the State's **80%** reduction goals.

## Wet Detention

The mean inflow and outflow concentrations, removal efficiencies, log removal values, and statistical significance of comparisons between inflow and outflow concentrations for the wet detention ponds are presented in Tables 2 through 5. Removal efficiencies were typically greater for fluorescent beads followed by **MS2,** FC, and TC. Differences between inflow and outflow concentrations were significantly different for turbidity, **TSS,** TC and FC, MS2, **and** beads during the 5-day shallow trial. Concentrations of turbidity, **TSS, and** TC were significantly greater in the outflow than the inflow during the 5-day deep trial, however, FC, MS2, and beads were all significantly reduced. During the 14-day shallow trial, only turbidity, **TSS, and** bead concentrations were significantly lower in the outflow compared to the inflow.

Table **2. Mean** concentrations, log removal (based **on** concentrations), and removal efficiencies (based on loads) for indicator and physical parameters from the 5-day shallow wet detention pond challenge.

| **Parameter** | **Inflow** | **Outflow** | **Mean Log Removal** | **Load Removal Efficiency** |
|---|---|---|---|---|
| Turbidity (NTU) | 1.23 | 0.86 | | 30.3% |
| Total Suspended Solids (mg/L) | 1.42 | **0.28*** | | 99.8% |
| Total coliforms (cfu/100 ml) | $1.14 \times 10^3$ | $2.41 \times 10^{2*}$ | 0.67 | 64.0% |
| Fecal coliforms (cfu/100 ml) | $2.29 \times 10^2$ | $5.48 \times 10^{0*}$ | 1.62 | 98.2% |
| MS2 coliphage (pfu/ml) | $9.25 \times 10^4$ | $1.13 \times 10^{3*}$ | 1.91 | 93.9% |
| 3 µm beads ***(Cryptosporidium*** surrogate) (beads/ml) | $3.72 \times 10^2$ | $1.23 \times 10^{0}$* | 2.48 | ***99.5%*** |

Table 3. Mean concentrations, log removal (based on concentrations), and removal efficiencies (based on loads) for indicator and physical parameters from the 5-day deep wet detention pond challenge.

| Parameter | Inflow | Outflow | Mean Log Removal | Load Removal Efficiency |
|---|---|---|---|---|
| Turbidity (NTU) | 1.13 | 4.32* | | -281.2% |
| Total Suspended Solids (mg/L) | 1.67 | 4.21* | | -81.4% |
| Total coliforms (cfu/100 ml) | $6.80 \times 10^2$ | $3.03 \times 10^3$* | -0.65 | -284.5% |
| Fecal coliforms (cfu/100 ml) | $1.59 \times 10^2$ | $2.42 \times 10^0$* | 1.82 | 88.5% |
| MS2 coliphage (pfu/ml) | $9.24 \times 10^4$ | $6.94 \times 10^2$* | **2.12** | 98.6% |
| 3 µm beads *(Cryptosporidium* surrogate) (beads/ml) | $3.08 \times 10^2$ | $2.61 \times 10^0$* | 2.07 | 99.0% |

Surface water quality standards ($\leq$ 29 NTU above background conditions) for turbidity for Class III waters were never exceeded in either mean inflow or outflow concentrations for any of the wet detention pond trials. TC concentrations exceeded Class III maximum values in 33% of inflow samples from the four pond trials. TC concentrations exceeded the Class III maximum value in 83% and 60% of outflow samples from the 5-day and 14-day deep pond trials, respectively, and in 40% of outflow samples from the 14-day shallow pond trial. FC concentrations exceeded the Class III maximum value in 42% of inflow samples from all four pond trials. FC concentrations exceeded the Class III maximum value in 40% of outflow samples during the 14-day shallow pond trial and in 60% of outflow samples for the 14-day deep pond trial but did not exceed the one day maximum value in outflow samples for either of the 5-day trials.

Table 4. Mean concentrations, log removal (based on concentrations), and removal efficiencies (based on loads) for indicator and physical parameters from the 14-day shallow wet detention pond challenge.

| Parameter | Inflow | Outflow | Mean Log Removal | Load Removal Efficiency |
|---|---|---|---|---|
| Turbidity (NTU) | 3.80 | 2.38* | | 37.4% |
| Total Suspended Solids (mg/L) | 3.56 | 0.96* | | 72.2% |
| Total coliforms (cfu/100 ml) | 4.34 x 103 | $4.82 \times 10^2$ | 0.96 | **4.2%** |
| Fecal coliforms (cfu/100 ml) | $2.08 \times 10^3$ | $4.44 \times 10^1$ | 1.67 | 76.4% |
| MS2 coliphage (pfu/ml) | $7.07 \times 10^3$ | $6.96 \times 10^2$ | 1.01 | 88.9% |
| 3 µm beads *(Cryptosporidium* surrogate) (beads/ml) | $1.88 \times 10^2$ | $2.33 \times 10^0$* | 1.91 | 99.1% |

Table 5. Mean concentrations, log removal (based on concentrations), and removal efficiencies (based on loads) for indicator and physical parameters from the 14-day deep wet detention pond challenge.

| Parameter | Inflow | Outflow | Mean Log Removal | Load Removal Efficiency |
|---|---|---|---|---|
| Turbidity (NTU) | 3.83 | 4.12 | | **-7.5%** |
| Total Suspended Solids (mg/L) | 3.40 | 2.22* | | 73.3% |
| Total coliforms (cfu/100 ml) | 3.51 x 103 | 3.53 x 103 | -0.003 | 31.9% |
| Fecal coliforms (cfu/100 ml) | $1.57 \times 10^3$ | $1.53 \times 10^2$ | 1.01 | 69.2% |
| MS2 coliphage (pfu/ml) | $6.95 \times 10^3$ | $1.90 \times 10^2$* | 1.56 | 94.7% |
| 3 µm beads *(Cryptosporidium* surrogate) (beads/ml) | $1.85 \times 10^2$ | $2.11 \times 10^0$* | 2.33 | 99.5% |

Of the six parameters, TSS, FC, MS2, and beads were reduced sufficiently to meet the 80% reduction goals during the 5-day shallow pond trial. Fecal coliforms, MS2, and beads were reduced sufficiently to meet the 80% reduction goals during the 5-day deep pond trial, however, turbidity, TSS, and TC were not. MS2 and fluorescent beads were the only parameters reduced sufficiently to meet the 80% reduction goals during both the 14-day shallow and deep pond trials.

**Alum Coagulation**

Removal efficiencies and log removal values for comparisons between alum and control samples are presented in Tables 6 and 7. Greatest reductions in the concentrations of TC and FC and turbidity occurred within 24 hours after the addition of alum in both the high and low dose trials with removal efficiencies often exceeding 97% for most microbial indicators. In the low dose (10 mg/L) jar tests, greater than 3-log reductions were observed for TC and FC and MS2 within the first 24 hours. After 48 hours, removal efficiencies (differences between the control and alum treated sample concentrations) for most parameters except TSS and fluorescent beads had declined.

During the low dose trial, turbidity and TSS concentrations were found at greater concentrations in the floc layer than in initial ($T_0$) water column concentrations prior to the addition of alum. Concentrations of TC and FC and beads in the floc layer were within 1-log unit of $T_0$ seeded concentrations. Greatest declines in MS2 concentrations occurred between $T_0$ and $T_{24}$ and then remained relatively low, even in the floc layer. TC and FC concentrations were significantly greater ($p \leq 0.05$) in the floc layer than in the water column 48 hours after the addition of alum. Bead concentrations were significantly greater in the floc than at $T_0$ or after 24 hours but not after 48 hours. Log-transformed TC concentrations were positively correlated with TSS and log-transformed bead concentrations were positively correlated with turbidity.

During the high dose (600 mg/ L) jar tests, greatest removal efficiencies occurred within 24 hours for turbidity and TC and FC while removal efficiencies for TSS, MS2, and beads were greater after 48 hours, Negative TSS and TC removal efficiencies were observed after 48 hours. Microscopic examination of undiluted $T_0$ and $T_{24}$ samples revealed floc materials in both control and alum treated samples. The appearance of alum floc may be a result of either the resuspension of floc material during sampling, a thicker than expected floc layer which extended into the sample collection area of the jar, or contamination of the source water from the full-scale alum treatment system located downstream (which may have unintentionally back-flushed alum upstream to the sample collection point for the jar tests). This phenomenon was also confirmed by elevated Al concentrations during the trial as well as greater than expected conductivity values.

Table 6. Removal efficiencies based on differences between concentrations of indicator and physical parameters in control versus alum treated samples for stormwater taken from Lowry Park and dosed at 10 mg/L alum.

| **Parameter** | **Time,** | **Time,,** | **Time,,** | **Log Removal After 48 Hours** |
|---|---|---|---|---|
| Turbidity (NTU) | 0% | 88.1% | 79.6% | |
| Total Suspended Solids (mg/L) | 0% | 74.1% | 84.4% | |
| Total coliforms (cfu/100 ml) | 0% | 99.9% | 98.5% | 1.8 |
| Fecal coliforms (cfu/100 ml) | 0% | 99.9% | 99.6% | 2.4 |
| MS2 coliphage (pfu/ml) | 0% | 99.9995% | 98.0% | 1.7 |
| 3 µm beads *(Cryptosporidium* surrogate)(beads/ml) | 0% | 96.4% | 98.2% | 1.8 |

Table 7. Removal efficiencies based on differences between concentrations of indicator and physical parameters in control versus alum treated stormwater samples taken **from** Pinellas Park and dosed at 600 mg/L alum.

| **Parameter** | **Time,** | **Time,,** | **Time,,** | **Log Removal After 48 hours** |
|---|---|---|---|---|
| Turbidity (NTU) | 0% | 50.0% | **7.6%** | |
| Total Suspended Solids (mg/L) | 0% | -59.3% | **-26.9%** | |
| Total coliforms (cfu/100 ml) | 0% | **33.3%** | **-3233.3%** | |
| Fecal coliforms (cfu/100 ml) | 0% | 100% | 100% | >2.0 |
| MS2 coliphage (pfu/ml) | 0% | 99.996% | 99.998% | 4.9 |
| 3 µm beads ***(Cryptosporidium*** surrogate)(beads/ml) | 0% | **81.2%** | 90.8% | 1.0 |

Surface water quality standards for turbidity ($\leq$ 29 NTU above background conditions) for Class III waters were never exceeded in any of the initial ($T_0$) nor subsequent samples after 24 and 48 hours. However, discharge of the concentrated floc would violate the Class III standards for turbidity. Both TC and FC bacteria concentrations exceeded the Class III one day maximum value in all (100%) $T_0$ samples during both the high and low alum dose trials. TC bacteria concentrations did not exceed the Class III maximum value in any of the 48-hour ($T_{48}$) low dose alum samples but did exceed the one day maximum value for 50% of all T„ and $T_{48}$ control samples and **33%** of all $T_{24}$ **and** $T_{48}$ high dose alum samples. FC bacteria concentrations did not exceed the Class III maximum value in any of the 48-hour ($T_{48}$) high or low dose alum samples but the one day maximum value was exceeded in 50% of all control samples. TC and FC concentrations in floc samples from both alum treated and control tests from the low dose trial would have exceeded Class III standards if discharged to a protected waterbody. **TSS,** TC and FC, **MS2,** and beads were reduced sufficiently to meet the 80% reduction goals during the 10 mg/L trial. TC and FC, **MS2,** and beads were reduced sufficiently during the 600 mg/L trial to meet the **80%** reduction goal.

Low dose alum coagulation treatment resulted in the greatest overall removal efficiency values for total and fecal coliforms and turbidity. **MS2** removal was greatest using alum treatment, but was also typically greater than 80% for all other BMPs. Removal efficiencies for beads were greater **than** 90% for all three treatment systems. The greatest bead removal (99.5%) was identical for sand filtration, 5-day shallow, and 14-day deep wet detention pond treatments. Greatest turbidity removal was achieved using the sand filter followed closely by alum treatment (low dose). Total suspended solids removal was greatest during the 5-day shallow pond treatment followed by alum treatment (low dose).

Removal of pathogenic microorganisms using any of the three stormwater BMPs in this study may result in a potential reduction in health risks from contaminated stormwater. A reduction in health risk can be determined if the inflow concentration of a particular group of pathogenic microorganisms is reduced substantially to pose little or no health threat to a person exposed (via direct ingestion) to waters discharged at the outflow. The calculations assume that water from the outfall of a stormwater treatment system is not diluted and the person ingests an average of 100 ml of water. An estimate of ingested dose was calculated by multiplying the outflow concentration by 100 ml and the difference between this value and the infective dose determined either a positive or negative reduction in risk,

For bacteria, a conservative estimate of 10 vegetative cells was used as the infective dose with inflow concentrations ranging from $1.0 \times 10^2$ to $1.0 \times 10^4$ cfu/100 ml. Positive reductions in risk only occurred during low inflow concentrations using alum treatment (at a dose of 10 mg/L). For enteroviruses, a range of concentrations from $1.0 \times 10^2$ to $1.0 \times 10^4$ pfu/ml at the inflow was used with an infective dose of 10 virions. For protozoa (specifically ***Cryptosporidium*),** a range of concentrations from $2.0 \times 10^2$ to $2.0 \times 10^4$ oocysts/ml was used with an infective dose of 132 oocysts. Positive reductions in risk for enteroviruses were only observed using alum treatment (all levels of contamination). For protozoa, reductions in risk were only observed using wet detention and sand filtration at low levels of contamination.

******Due to the limited space available for these proceedings, a great deal of detailed information has been omitted in this paper. Interested readers are encouraged to obtain a copy of the full report available from the Southwest Florida Water Management District in Brooksville, FL.***

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# SUMMARY OF THREE INNOVATIVE STORM WATER BMPS

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## ABSTRACT

With the implementation of EPA's Phase 2 storm water permitting program and the changing focus of local and State storm water regulations, water quality and pollutant removal are growing issues in the storm water field. In this regulatory environment, municipal managers must look at storm water management in a holistic manner and determine such factors as the area to be managed, the soil types and land uses that prevail in the drainage area, and other factors. Based on this initial survey, storm water managers must then evaluate which Best Management Practice (BMP) strategies are appropriate for their area. This determination should include an evaluation of storm water BMP performance in terms of both flow control and water quality benefits. For the most part, past BMP evaluations have focused on individual sites evaluated under widely varying conditions, making comparisons of BMP types difficult. This paper evaluates three types of innovative storm water BMPs - storm water wetlands; bioretention; and modular systems (i.e., the StormTreat™ system) - and compares them in terms of specific applications and performance data. These three BMPs combine the flow control benefits of other structural BMPs with the enhanced pollutant removal capabilities inherent in vegetation-enhanced storm water BMPs. They are particularly effective relative to other BMPs in nutrient removal, which may make them appropriate for specific storm water applications. By using this comparative approach with their site-or area-specific requirements in mind, storm water managers will be able to choose the right storm water BMP to fit their needs.

## INTRODUCTION

Recently implemented and impending storm water initiatives **and** regulations have placed new demands on storm water managers and those persons responsible for implementing and overseeing storm water programs. The **primary** federal regulatory drivers for the current storm water program are the Phase I and Phase II Storm Water Regulations, which, among other requirements, require regulated entities to acquire a National Pollutant Discharge Elimination System (NPDES) permit for their storm water discharges. Among the requirements for obtaining an NPDES storm water permit are requirements to implement controls of municipal wet weather runoff and to implement good housekeeping activities in municipal operations. Other direct regulatory requirements include the Total Maximum Daily Load (TMDL) program, initiated under Section 303 of the Clean Water Act, which requires the calculation of enforceable maximum limits on pollutant loading into water bodies, In addition to these direct regulatory requirements, various national, State, and local initiatives are focusing on the issues of watershed management and pollutant loading into water bodies. These initiatives are setting goals and enforceable standards for reducing the effects of contaminated runoff on receiving waters. The Clean Water Action Plan, which President Clinton initiated in February, 1998, includes goals for curbing polluted runoff and encouraging adoption of enforceable storm water controls. In addition, the plan encourages implementation of numeric criteria for nutrients (i.e., nitrogen and phosphorus) in water bodies by 2000. Other federal regulations include the Coastal Zone Act Reauthorization Amendments **(CZARA),** which require nonpoint source control programs for coastal areas, and the Clean Water Act Section 319 nonpoint source program, which established baseline requirements for nonpoint source management programs. Other examples can be found in more localized initiatives, such as the Great Lakes and the Chesapeake Bay programs. Overall, these regulatory requirements and clean water goals will require storm water managers to adopt measures to achieve effective and measurable reductions in pollutant loadings into receiving waters,

This goal can be partially accomplished through the implementation of structural and/or non-structural and programmatic storm water Best Management Practices, or BMPs. While the term "storm water BMP" often brings to mind only structural storm water mitigation "facilities," such as wet detention ponds or infiltration practices, it is important to remember that BMPs may include such programmatic practices such as implementing a spill prevention planning program or a catch basin cleaning program,

Structural BMPs can be effective methods for removing pollutants from storm water runoff and thus reducing pollutant loads into receiving waters. However, BMP applicability and effectiveness is site-specific, and BMPs must be selected, designed, and installed based on site-specific conditions such as drainage area, land use, soil types, depth to the water table, and other factors.

Ideally, storm water managers and municipal officials responsible for storm water programs would be able to draw on local data and experience to assess BMP applicability and effectiveness for specific projects. However, standardized, comparable data on BMP applicability and performance is lacking in many areas of the country. Under contract to Headquarters U.S. Environmental Protection Agency (EPA), Office of Wastewater Management, Municipal Technology Branch, Parsons Engineering Science, Inc. (Parsons **ES)** has prepared a series of Fact Sheets on storm water BMPs that summarize available national data on a variety of structural and non-structural BMPs and present it in a standardized format that makes the data easy to use and readily available for BMP comparison. This paper will explore a case study in which data from three fact sheets (Bioretention, Storm Water Wetlands, and Modular Systems) are examined in terms of their appropriateness for specific applications.

## DISCUSSION

In order to comply with new and existing storm water regulations, storm water program managers will be required to design, implement, and/or regulate and review storm water BMPs to reduce pollutant loading into receiving waters. While the federal NPDES Phase I and Phase II regulations require only the implementation of storm water BMPs for construction sites and new municipal development, the requirement to implement good housekeeping practices and the water quality goals of national and local initiatives will encourage the implementation of storm water BMPs in retrofit situations. The challenges in implementing storm water BMPs for an entire municipality are much different than for implementing BMPs at smaller sites. In addition, storm water BMP pollutant removal goals may be very different between a new development and a municipality encompassing many different land uses. For example, construction sites might be most concerned with sediment and erosion control and the problems associated with moving soils off-site, while municipalities may be forced to deal with nutrient removal issues from runoff from farmland or other areas using fertilizers, or metals or hydrocarbon removal from industrial areas. The result of these differences in BMP requirements is that municipalities will most likely require a BMP program consisting of various appropriate BMPs (which may include several different BMPs or multiple applications of the same BMP) to meet their local water quality goals.

In order to implement and/or oversee storm water BMP programs, storm water program managers must have the best and most recent information on storm water BMP applicability, design, performance, and cost. Unfortunately, literature and data on storm water BMPs is often hard to find and even harder to compare. Much of the literature on storm water BMPs has been generated by municipalities, state agencies, manufacturers, and industry groups. It often comes from pilot and evaluation projects and from demonstrations by manufacturers. In addition, most storm water BMP data comes from a small number of municipalities and regions, including the Northwest (particularly Washington and Oregon), the Upper Midwest (particularly Michigan, Minnesota and Wisconsin), the Chesapeake Bay area, and Florida. Much of the interest in storm water BMPs in these regions has been generated by local or regional clean water initiatives, such as the Chesapeake Bay and the Great Lakes programs. These initiatives have often spawned partnerships that focus on larger watershed issues, such as developing management plans to clean up receiving waters. These larger entities often apply for grants and demonstration project funding to evaluate specific storm water BMPs. For example, the Clean Water Act Section 319 nonpoint source control program has provided funding for nonpoint source control demonstration projects, which can include storm water BMP evaluation projects. Another example is the Rouge River Wet Weather Demonstration Project, in which EPA, the Michigan Department of Environmental Quality, the Wayne County Department of the Environment, The Rouge Remedial Action Plan Advisory Council, and other partners are evaluating wet weather controls for the Rouge River watershed. These projects often generate good data on BMP design, applicability, and cost. However, because these projects are intended to evaluate BMPs on a localized basis (i.e., under site-specific soil conditions, land use, and hydrology), extrapolation of pollutant removal performance data for other applications may or may not be valid. **A** second source of performance data is data from manufacturers; however, because there are no widely-accepted protocols for monitoring and evaluating pollutant removal performance, this data may be biased,

## Using EPA's Storm Water BMP Fact Sheets

**EPA's** series of Storm Water BMP Fact Sheets provide a good overview of storm water BMPs because they compile and standardize data for a number of structural and non-structural storm water BMPs. The Fact Sheets are presented in a standard format that includes information on design criteria for the BMP; data on BMP applicability, advantages and disadvantages, and performance; and cost data. Design criteria for a structural BMP can include information on sizing and placement of the BMP, alternative construction materials to fit the site, and optional features, while design criteria for a non-structural BMP can include information on designing and implementing a BMP program. Applicability data includes information on how and where the BMP can be used effectively and efficiently. Performance data includes data on pollutant removal efficiency and may or may not be expanded to include further analyses of the effects of sizing, and the frequency and intensity of runoff, on pollutant removal mechanisms and efficiency. Cost data can include both capital cost and operations and maintenance data.

The Storm Water BMP Fact Sheets can serve as an essential planning tool for storm water program managers who must get some basic information on BMP feasibility and design. They can also be a first step from which program managers can proceed to sources of more information. The paragraphs below describe each section of the Fact Sheets in more detail. Following these descriptions is a discussion of how the fact sheets can be used to compare BMPs top determine their applicabilities for different management scenarios.

## Description and Applicability

The first two sections of each Fact Sheet, titled "Description" and "Applicability," respectively, provide background information on the storm water BMP described in the fact sheet and how it is used. These data can include the history of the development of the BMP, the general scenarios in which it is used, and other basic descriptive information. **A** diagram that shows a generalized schematic of the way the BMP works is also usually provided.

## Design Criteria

The fact sheets also provide general design information that can help a storm water program manager to determine more specifically if a BMP is feasible for a specific application. Design information is crucial in ensuring that a BMP will function efficiently under specific site conditions. Important design factors include the site soil type, the depth to the water table, the depth to bedrock, site slope, and the watershed area to be served (NVPDC, 1992). For example, infiltration practices such as bioretention may not be practical in soils with greater than a 25 percent clay content. Bioretention is not recommended in areas where the water table is within 2 meters of the surface, or in areas where the slopes are greater than 20 percent. In contrast, the design criteria for constructed storm water wetlands are primarily concerned with sizing the wetland for the design storm and ensuring that the flow is circulated through the wetland and detained for a sufficient time for treatment to occur. Site soil types and the watershed area to be served are both crucial design factors for storm water wetlands. Because they are self-contained system, the design criteria for modular systems such as StormTreat™ primarily focus on the influent flow volumes and flow rates and the desired storage capacity, although they the climate and geographic area will also dictate the types of wetland plants that are appropriate for the system.

## Performance

The next section of the Fact Sheets provides pollutant removal efficiency data that can be used to compare BMPs to determine if they may be feasible methods to achieve a system's pollutant removal goals. It is important to remember that the goals of different storm water programs may be different. For example, the Minneapolis area and the Chesapeake Bay area of Virginia focus on nutrient removal (particularly phosphorous removal), while North Carolina requires 85% annual removal of suspended solids from their storm water BMPs. Therefore, different BMPs should be assessed based on the goals of the program.

While evaluating BMP performance is crucial in determining the overall feasibility of a BMPs for a specific application, it has also been one of the most difficult aspects of BMP evaluations. Traditionally, it has been difficult to efficiently compare and evaluate BMP performance data because most studies do not standardize the way they present their data. For example, the primary factors influencing a BMP's pollutant removal efficiency are its design, the local runoff characteristics, and the pollutant loading rates. These factors must be addressed during the discussion of the BMP's performance to put the performance data in context. The Fact Sheets are beginning to make this task easier. They present a comprehensive, unbiased picture of the BMP's pollutant removal efficiency, often by presenting compilations or comparisons of performance data from several different sources that show the range and variability of the BMP's performance.

In addition to the Fact Sheets, there are several other good sources of data on storm water BMP performance, including the "National Pollutant Removal Performance Database for Storm Water Best Management Practices," a database developed by the Center for Watershed Protection that attempts to standardize performance data from existing case studies, and the National Storm Water Best Management Practices database, being prepared by the American society of Civil Engineers **(ASCE)** for **EPA.**

## costs

The Storm Water BMP Fact Sheets can also be an excellent method to compare basic costs for storm water BMPs. BMP costs can include both capital and operations and maintenance costs, both of which are discussed in the fact sheets. Like BMP performance data, most of the cost data generated on storm water BMPs has been inconsistent, Some of the problems with the cost data for storm water BMPs include the facts that data may not break out capital costs from O&M costs, or that costs may not be broken out into definable units, like costs per acre or costs per unit. In addition, structural and non-structural BMPs will obviously have very different cost structures because the distinction between capital and O&M costs are not as distinct. The Fact Sheets standardize costs and often utilize tables that show design and sizing details to make cost comparisons easier.

Being able to compare the relevant costs of a BMP is a very important tool for storm water managers, especially when one can break the costs into costs per unit or costs per treated area. This will allow a much more valid comparison of the true costs of storm water management. In addition, there may be economies of scale that are not evident in the direct unit costs but that become evident as one compares the unit costs of one BMP versus its intended use. For example, while the costs for bioretention and the StormTreat™ system are substantially less than costs for a storm water wetland system, these latter BMPs are primarily intended to treat smaller runoff volumes from self-contained areas. Storm water wetlands may be the BMP of choice for a larger drainage area and may benefit fi-om an economy of scale in that there is a larger economic base in a larger drainage area from which to **support** the BMP. Thus, a storm water wetland may be a more cost-effective choice for larger entities like municipalities or townships.

## BMP Comparison

The following paragraphs describe an example scenario in which data from EPA's storm water BMP fact sheets for Storm Water Wetlands (**EPA** 832-F-99-043), Modular Systems (EPA 832-F-99-044), and Bioretention (EPA 832-F-99-012) are used to compare the feasibilities of different storm water BMPs for specific site applications.

The three BMPs discussed here (Storm Water Wetlands, Modular Systems, and Bioretention) are all systems in which flow is diverted to an area in which natural processes are used to remove pollutants. The first system, storm water wetlands, is a relatively common system on which there has been abundant research. Storm water wetlands can be either natural or constructed systems which are characterized by specific types of vegetation and often contain pools of standing water. Storm water wetlands remove pollutants through a complex series of physical, chemical, and biological processes, and they are often several acres in area. Storm water wetlands are often used as end-of-pipe BMPs that are used to treat storm water collected from large drainage areas.

While similar to storm water wetlands in that natural processes play an important role in pollutant removal, bioretention and modular systems for storm water treatment are otherwise very different. Bioretention is a practice pioneered by Prince Georges County, Maryland. A bioretention system consists of a grass buffer strip leading to **a** sand bed **and** a ponding area. The grass buffer strip and the sand bed slow the runoff and distribute it more evenly to the centralized ponding area, which consists of **an** organic surface layer and/or ground cover. Water **is** ponded to a depth of 15 cm and gradually infiltrates the soil or is evapotranspired. The system's mulch layer supports microbial activity that can help to degrade petroleum-based products and other organics, while the heavy metals and other pollutants may sorb to the underlying clay soils. The bioretention system may be modified by adding an underdrain beneath the sand bed, enabling filtered runoff to be diverted directly to a receiving water body. In contrast to storm water wetlands, bioretention is primarily used for smaller applications that receive sheet flow runoff, such as parking lots or other impervious areas. High flows may erode or flood the system, and thus it is not practical for end-of-pipe applications.

Modular storm water treatment systems, such as the StormTreat™ system, consist of a series of sedimentation chambers and constructed wetlands. As with bioretention, a modular system such as StormTreat™ is ideal for use in highly impervious areas such as commercial areas or industrial parks. However, unlike bioretention, storm water flow is piped directly into the StormTreat™ system and thus flow into the system can be controlled. The system works by first sedimenting and filtering particulate matter, and then releasing the filtrate to the root zone of a constructed wetland system. The system's wetland plants then take up nutrients and metals, while some pollutants are bound in the wetland soils. An adjustable valve controls the flow out of the system, and thus this system can also provide storm water volume control. Table 1 summarizes the general characteristics and the situations in which these BMPs are applicable.

Table 2 summarizes the pollutant removal efficiencies for these BMPs. As shown in the Table, both bioretention and the StormTreat™ system show high pollutant removal efficiencies for most pollutants, including TSS, nutrients, and metals. While pollutant removal efficiency rates may overall be lower for storm water wetlands, these differences may be the result both of the increased volumes treated by storm water wetlands and the differences in sampling and evaluation methods used in the different reports.

| TABLE 1 CHARACTERISTICS OF THREE INNOVATIVE STORM WATER BMPS | | | |
|---|---|---|---|
| | Storm Water Wetlands | Bioretention | StormTreat™ System |
| General Description | Natural or constructed system consisting of characteristic "wetland" soils and plants located in area where water coverage is characteristic. Pollutant removal is achieved through physical, chemical, and biological processes. | System consists of grass buffer strip leading to a sand bed and ponding area. Water gradually infiltrates soil or is evapotranspired. Mulch layer supports microbial activity to degrade petroleum-based products and organics, while heavy metals sorb to the underlying clay soils. | System consists of sedimentation chambers and constructed wetlands. System sediments and filters particulate matter, and releases filtrate to the root zone of constructed wetland system. Plants take **up** nutrients and metals, while some pollutants are bound in the wetland soils. |
| Applicability | Often used as "end-of-pipe" BMP for larger drainage areas. Very adaptable because wetlands are found in all regions of country. | Primarily used for smaller applications that receive sheet flow runoff, such as parking lots or other impervious areas. High flows may erode or flood the system, and thus it is not practical for end-of-pipe applications. | In-line system receives flow from drainage pipe. Flow control valves allow treatment of prespecified volumes. Ideal for use in parking lots or other impervious areas. |

| | Storm Water Wetlands | Bioretention | StormTreat™ System |
|---|---|---|---|
| Total Suspended Solids | 67 | 90 | 99 |
| Phosphorous | 49 | 70-83 | 90 |
| Nitrogen | 28<br>(as total N) | **68-80**<br>(as TKN) | 77<br>(as total dissolved N) |
| Metals<br>Copper<br>Lead<br>Zinc | <br>41<br>62<br>45 | <br>93-98<br>93-98<br>**93-98** | <br>Not reported<br>**77**<br>90 |

Different BMPs can also vary widely in cost. For example, costs for storm water wetlands include those for permitting, design, construction and maintenance, and construction costs range from $65,000 to $137,500 per hectare ($26,000 to $55,000 per acre) of wetland for an emergent wetland with a sediment forebay. Cost data for bioretention is not as readily available, but costs will include excavating and grading the site and adding planting soils and wetlands vegetation.

While costs for bioretention at a new site will be lower, costs of retrofitting existing sites have been reported to range from $6,500-$7,440 per bioretention area. In contrast, StormTreat™ systems are sold as pre-manufactured units costing $4,900 each, with installation costs running from $500-$1000 per unit. Additional costs include wetland plants, PVC piping, and fill gravel. These costs are estimated to be between $350 and $400 **per** unit for a total estimated cost **per** unit of $5,750-$6,300.

As illustrated by these comparisons, storm water wetlands, bioretention, and the StormTreat™ system should be used for very different applications. Bioretention and modular storm water treatment systems, such **as** StormTreat™, are often used to treat flow from self-contained areas, such as parking lots or office **parks.** In contrast, storm water wetlands are often used to treat runoff from larger areas. EPA's Storm Water BMP Fact Sheets can help storm water program managers in comparisons such as these as they evaluate storm water BMPs for use in their programs.

In conclusion, storm water program managers can utilize **EPA's** Storm Water BMP Fact Sheets to help them evaluate storm water BMPs. Numerous programs, including the Phase **I** and Phase II NPDES regulations, plus water quality initiatives, will require updates to existing storm water programs, including the implementation of storm water BMPs. For many areas, there is little local data to draw from directly in updating storm water programs, and while there is a large quantity of storm water BMP information available, it is often in differing formats and has different degrees of completeness. **As** the examples in this paper show, the Fact Sheets allow the comparison of storm water BMPs in terms of overall feasibility for use at a site, design considerations, pollutant removal efficiency, and cost. Storm water program managers can use storm water BMP information in a standardized format such as the EPA Fact Sheets to help define and meet their storm water management goals.

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# THE USE OF A CDS UNIT FOR SEDIMENT CONTROL IN BREVARD COUNTY

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## ABSTRACT

In July 1997, Brevard County's Stormwater Utility Program installed a new type of trash and sedimentation control device called a **CDS** unit. This was the first American installation using the continuous deflection separation **(CDS)** technology developed in Australia. **After** installation, autosamplers were placed upstream and downstream of the **CDS** unit and a years duration of sampling data have been collected. Monitoring has shown that the **CDS** unit has provided an average 52% removal efficiency for total suspended solids and 31% removal efficiency for phosphorus.

## INTRODUCTION

Stormwater sedimentation is a primary source of pollution to the Indian River Lagoon. Suspended solids and turbidity reduce sunlight penetration in the Lagoon which negatively impacts seagrass growth. Where land is available, detention ponds effectively reduce most of the suspended solids from stormwater flows. When land is not available; alternative less effective, treatment methods must be used.

The CDS technology was initially developed in Australia to provide an effective method for trash and solids removal from stormwater flows. The screening action within the unit provides for 100% removal of trash and particles down to 4700 microns. In addition, the unique circular design creates centrifugal action within the round concrete **box** which propels suspended solids to the center of the box and down into the storage chamber.

## METHODS

The location chosen for the CDS unit installation was along a ditch at the north end of Brentwood Drive, south of Port St. John. The drainage basin for this location was 24.87 hectors (61.45 acres) in area. This basin has Type A soils along a sand ridge. The land uses are 24.87 hectors (6.7 acres) of roadway (*US* Highway 1), 8.04 hectors (19.87 acres) of industrial park, 9.47 hectors (23.39 acres) of vacant land, and **4.65** hectors (11.49 acres) of commercial property. the industrial area has a permitted stormwater system. A significant land feature is a 2.02 hectors (*5* acre) dirt parking lot, 152.4 meters (500 feet) upstream of the site around the Corky Bells restaurant.

This parking lot has a steep slope and is composed of fine white base material. There is evidence of heavy silt buildup in the inlets and pipes downstream of this parking lot along US 1.

There is an earthen ditch running eastward 76.2 meters (250 feet) upstream from the project location. At the project site there is an existing 122 centimeter (48 inch) RCP driveway culvert in the ditch which discharges to a concrete channel running 152.4 meters (500 feet) eastward to the Indian River, The time of concentration to the site is 63 minutes with a 10 year flow of 1,557.2 L/sec (55 cfs) and mean annual flow of 1,177.9 L/s (38.2 cfs). In Brevard County, the 10 year storm is 20.1 centimeters (7.9 inches) of rainfall and the mean annual storm is 13.97 centimeters (5.5 inches) of rainfall. There is no base flow at this location.

A diversion weir 68.58 centimeters (27 inches) tall is placed in front of the 122 centimeter (48 inch) culvert so as to divert flows over 254.8 L/sec (9 cfs) around the unit. In 18 months of observations the water level has risen over the weir one time.

A 76.2 centimeter (30 inch) concrete pipe was constructed adjacent to the existing 122 centimeter (48 inch) pipe in order to divert flows to the CDS unit. The 76.2 centimeter (30 inch) pipe enters the CDS unit tangentially to start the circular flow within the unit.

The CDS unit (Figure 1) consists of three (3) circular, concrete chambers stacked on top of each other. The top chamber, where the water enters the unit, has a 1.524 meter (5 feet) inner diameter and is 188 centimeters (74 inches) tall. The middle chamber has a 2.44 meter (8 feet) inner diameter and is 127.54 centimeters (51 inches) tall. In the middle chamber is a 1.524 meter (5 foot) diameter stainless steel screen matching the walls of the top chamber. The screen has 4700 micron holes to filter larger materials. The bottom chamber has a 1.22 meter (4 foot) inner diameter by a 1.22 meter (4 foot) tall sediment sump.

Water enters the unit in a clockwise rotation. When the water passes through the screen it then flows counter clockwise between the screen and outer wall until it reaches a 76.2 centimeter (30 inch) concrete pipe. This exit pipe is again tangentially placed for smooth exit flows. The elevation of the exit pipe rises 96.52 centimeters (38 inches) from the lower chamber to the outflow channel downstream of the 122 Centimeter (48 inch) culvert. This rise in elevation keeps the normal water level in the unit near the top of the 2nd chamber at all times. There is no base flow at this location.

The top of the unit is flush with the surrounding ground and has a 0.91 meter (3 foot) square, lockable, stainless steel access cover. This feature allows for easy access with a vacuum truck for cleaning purposes.

The CDS unit was installed on July 17, 1997. Installation took two (2) days with the precast structures. A large crane was required to lift the chambers into place. A 4.57 meter (15 foot) deep hole was excavated to place the structure in.

In conjunction with the CDS unit installation, County forces cleaned the ditch upstream of the unit. Two (2) days latter a significant rainfall event occurred and 2,294 kilograms (6,600 pounds) of sediment from the upstream ditch were trapped in the unit. After that storm the ditch was reworked and sod was laid. The sod greatly reduced the volume of sediment washing into the unit.

Cleanouts were also performed on November 17, 1997, with 626.84 kilograms (1382 pounds) of sediment and 2.88 meters (34 cubic feet) of trash and debris, and again on May 6, 1998 with 998 kilograms (2200 pounds) of sediment. The solids removed from unit are taken to the Brevard County landfill for disposal. The volume of water stored in the unit is greater than the vacuum truck capacity so decanting is performed on nearby sandy soils to avoid a second trip to the landfill for disposal.

## Evaluation of The Cds Unit During Storm Events

The intent of the sampling was to evaluate the effectiveness of the **CDS** unit in removing pollutants from a storm event prior to discharging into the Indian River Lagoon.

Five storm samples were collected at the CDS unit between April **1998** and March 1999. The storm events were captured after *dry* periods ranging between 7 and 75 days. Protocol for this program dictated that if the sample collection devices (autosamplers) triggered at intervals of less than **3** days between storms the samples were to be discarded. This situation did not occur during the year period, and near drought conditions were observed in the sample area throughout most of the year-long monitoring program.

Rainfall was measured at the sampling site by a tipping bucket rain gauge; and additional rainfall data obtained from the Orlando Utilities Commission (OUC) power generating plant ***5.6*** km (3.5 miles) to the north of the **CDS** installation.

Review of the rainfall data collected indicates the majority of the water passing through this BMP was from precipitation falling on the upland, 18.72 hectares (46.25-acre), watershed. The variation noted in both coverage and amount of rainfall helps illustrate the localized nature of the storms occurring along the Lagoon coastline. During this sampling period, water flowing off the drainage basin contributed much more flow through the **CDS** unit than would have been expected based on the rainfall recorded at the sample site.

Samples were collected through the use of automated storm water samplers; one (1) at the inlet and another at the outlet pipe of the **CDS** unit. All samples, associated blanks, and duplicates were collected in accordance with our state certified Comprehensive Quality Assurance Plan.

The stainless steel intake strainers for the samples were mounted on the reinforced concrete pipe, slightly off center bottom, and both angled away from the flow. This was to prevent the strainers from becoming silted over by sediments and allow collection of representative water samples. Flow rates during the storm events was measured initially utilizing water level meters **(ISCO** bubbler type) in conjunction with a 90-degree V-notch weir and eventually replaced with a Doppler area-velocity flow meter to provide a more accurate flow assessment. Initially, two bubbler meters were installed with both bubbler tubes mounted on the upstream weir. However, this led to difficulties in estimatingjust when to trigger (time delay) the downstream sampler in order to collect samples from the same "plug" of water.

During the first three sample events, water levels recorded were correlated to flow, and the samples were manually cornposited to give a flow-weighted composite sample from each sampler. Both inlet and outlet sample sets were cornposited identically, in accordance with the EPA NPDES Stormwater Sampling Guidance Document (July 1992). Discreet samples were collected for the fourth and fifth events.

It was intended that the third sample event would include a mass balance calculation. The CDS unit sump **was** thoroughly cleaned utilizing a VAC-truck to assure that the material collected was a result of the one storm to be evaluated. Inlet and outlet stormwater composite samples were again collected, with the addition of a sediment (Table 1) and water column sample from the sump. Sediment depths were measures at five locations; four from the corners of the lid opening and once in the center. Based on a depth of 13.21 centimeters, a **sump** diameter of 1.22 meters (4 feet) and an estimated 1,410.6 kg/m3 **(88** lb/ft$^3$), (based on previous sediment weight evaluation) approximately 217.3 kilograms (479.2 pounds) of sediment was collected in the unit from storm three. Based on the concentrations measured, 126.07 grams (4.443 ounces) BOD 5, 33.587 grams (1.184 ounces) of metals, and 122.81 grams (4.33 ounces) of TKN were removed.

For this third sample event, the upstream, or intake flowmeter bubble tube was mounted on the 90-degree V-notch inlet weir as it was for previous sample events. The downstream bubbler, however, was moved and attached to the downstream discharge pipe. This change was necessary to account for the lag time between when the first sampler received flow (at the beginning of the storm) the time required to fill the sump 8,115 liters (2,144 gallons), and discharge to occur providing flow past the second sampler several minutes later. The problem encountered with this setup was that the upstream V-notch weir used to determine the **flow** was overtopped, allowing flow around and over it and preventing **an** accurate flow measurement. This led to disparity in the estimation of actual flow through the unit. Due to the questionable flow measurements, it was not possible to calculate the mass balance.

For the fourth sample event **an ISCO** Doppler area-velocity flow meter was mounted in the bottom of the outfall pipe of the **CDS** unit. Upon registering a water level rise of one inch, this unit triggered both upstream and downstream autosamplers. The autosamplers were synchronized to collect 2 bottle sets at the same times. With this methodology and placement, overtopping the weir, flow bypassing, and pressurization were no longer potential source of error. Since the samplers now triggered only when the **sump** was full, it was also somewhat easier to accept the premise of "what went in, must have come out".

Appropriate trigger points were selected in order to allow sufficient water depth for the velocity meter probe to operate properly. The Doppler area-velocity flow meter probes appear to function erratically when covered **by** less than one inch of water, **and** measurements taken when the water was at this depth are suspect. Two-bottle sample sets were collected at sampler initiation, and at 10-minute intervals during the storm. During previous sample excursions samples were manually composited. Due to the high suspended solids content, (heavy particles including sand) that rapidly settled in the sample container, it was questioned whether the composite samples were truly representative of the solids collected. Therefore, discrete **2** bottle sets collected every 10 minutes were sent to the laboratory without being cornposited.

For the fifth sample event, two-bottle sample sets were again collected at sampler initiation, and at 10-minute intervals during the storm, **As** with the previous sample event, sample sets were not composited but sent for analysis as ***6*** individual, 2-bottle sets. The sample bottles for bottle sets ***6*** were not collected, due to insufficient water to cover intake strainers, as the storm was not of adequate duration to keep the flow up for the time required to collect the last 10 minute sample. Because of numerous problems encountered in the previous storm event samplings, along with refinements in sampler setup **and** flow measurement, the **fifth** storm sample event is considered the most accurate determination of what pollutant reduction is provided by the CDS unit for that storm. The individual 2 bottle sets shows the variation in pollutant loadings throughout the storm event and the corresponding removal under the varying loads. Unfortunately, this was the lowest flow storm encountered, which may account for higher than normal removal efficiency. Maximum flow was estimated to be only **136** liters/sec **(2.16** gpm). The average pollutant reduction between inlet and outlet samples for this event was: BOD5 **53%,** COD **52.6%,** TP **36%, TSS** 56%, and Turbidity **74.8%**

Sample results are presented in tables 2 through **4** for the 5 sample events. Storm event **2** showed a **23%** reduction in turbidity but no reduction in the other parameters. Storm **4** showed an increase in most parameter concentrations between inlet and outlet that could not be attributed to resuspension due to a full sump due to the sump having been cleaned prior to the third event. Data for these two storms are therefore suspect. For events 1, 3, and **5**, the average removal efficiencies for those parameters that showed a reduction were: TSS 52%, Turbidity **46.9%,** BOD **34.2%,** COD 35%, and TP 30.6%

After each sample event field observations were also made of the appearance of the samplejars, each containing a water sample which had been collected at progressive ten-minute intervals throughout the storm flow. Outlet samples typically appeared to be less turbid **than** the corresponding inlet samples, and also had less sediment on their bottoms. An observation was also made of the water surface inside the CDS unit proper. There was typically a thick layer of floating grass and other vegetation, an oil sheen, glass and plastic bottles, plastic sheets and bits, seeds and nuts, sticks, and a surprising amount of Styrofoam cups and particles. Each sample event is discussed in further detail in the full report.

## CONCLUSION

While none of the sample events were a perfect combination of a good flow and everything working right, the data collected and observations made certainly indicates that the **CDS** unit is operating as intended **and** removing significant quantities of debris and suspended materials prior to discharge to surface waters. It was quite impressive to think that this trash and sediment would have been washed out into the lagoon during a normal rain.

The phosphorus removals observed for the **CDS** Unit, as with any BMP of this type, will not have a high degree of accuracy due to leaching of nutrients from grass, leaves, and other organic debris. If there are no base flows these leached nutrients will be washed out with runoff and skew sample readings.

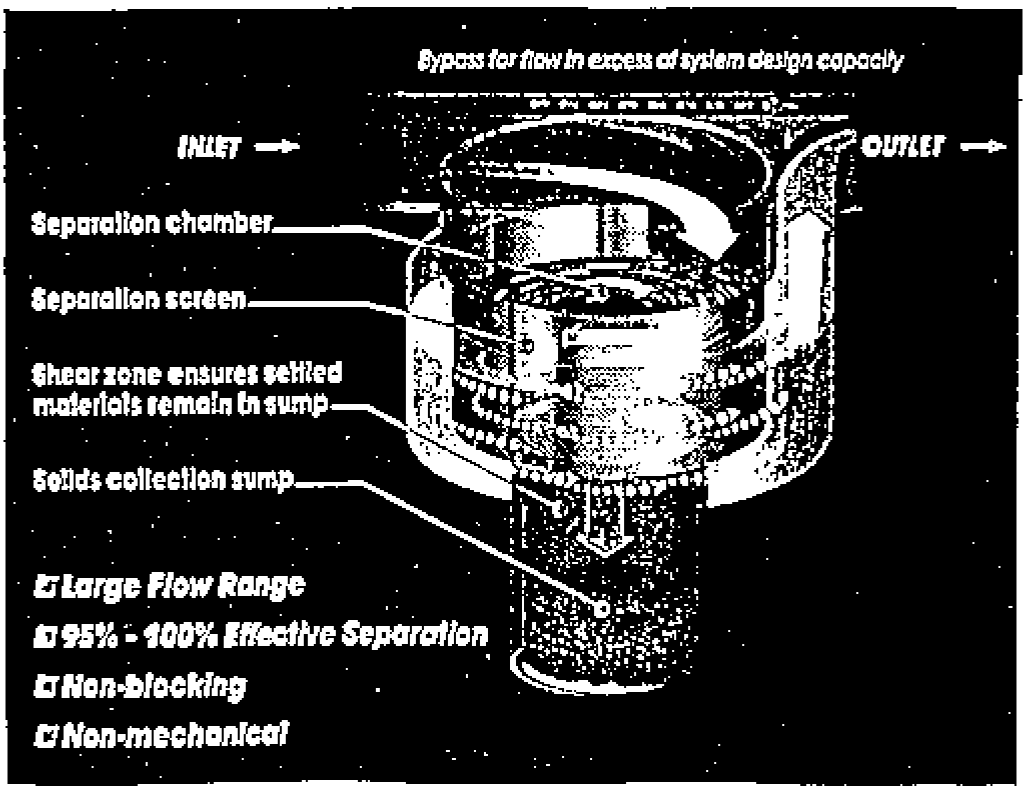
### Future Evaluations

More data is necessary to further evaluate this BMP. Due to the inherent inaccuracies in water quality sampling, additional determination of the efficiency of this **type** of BMP could be made by conducting a mass loading and sediment evaluation. Much of the sediment collected in this type of BMP is invisible to current testing techniques since it is comprised of large particles that roll along the bottom of the pipe. Yet known quantities of sediment are being collected. **A** previous study of baffle boxes resulted in the same conclusion. Future sediment analysis from the **CDS** unit could be compared to the baffle **box** data previously collected, Brevard County will be conducting a sediment evaluation at three **(3)** baffle **box** sites over the next twelve (12) months that will provide additional comparison. **As** time permits, Brevard County will also collect additional sediment data from he **CDS** unit.

Table I

| PARAMETER | Sediment **Grab** | Grab Duplicate | Average Value | Detection Limit | UNITS |
|---|---|---|---|---|---|
| **Arsenic** | 0.096 | 0.11 | 0.103 | 0.069 | Mg/Kg |
| Barium | 3.4 | 2.9 | 3.15 | 0.14 | Mg/Kg |
| Benzo(b)fluoranthene | 260 | ND' | 250' | 240 | Ug/Kg |
| BOD5 | 650 | 510 | 580 | 2.7 | Mg/Kg |
| Cadmium | 0.03 | 0.033 | 0.0315 | 0.014 | Mg/Kg |
| Chromium | 1.1 | 1.1 | 1.1 | 0.027 | Mg/Kg |
| Copper | 1.2 | 0.95 | 1.075 | 0.027 | Mg/Kg |
| Iron | 220 | *260* | 240 | 0.55 | Mg/Kg |
| Lead | 2 | *2.2* | 2.1 | 0.041 | Mg/Kg |
| Nickel | 0.4 | 0.36 | 0.38 | 0.069 | Mg/Kg |
| Silver | 0.16 | 0.059 | 0.1095 | 0.014 | Mg/Kg |
| Total Kjeldahl Nitrogen | 450 | *680* | 565 | 37 | Mg/Kg |
| Total Phosphorus | 79 | 230 | 154.5 | 9.2 | Mg/Kg |
| Zinc | 14 | 14 | 14 | 0.27 | Mg/Kg |

Figure 1



## Table 2

| Site: CDS STORM 1 | pH SU | Total Suspended Solids mg/l | Turbidity NTU | BOD5-Day mg/l | COD mg/l | Total Phosphorous mg/l |
|---|---|---|---|---|---|---|
| CDS Inlet | 7.6 | 220 | 180 | 28 | 150 | 1.4 |
| CDS Outlet | 7.4 | 110 | 100 | 23 | 110 | 1 |
| Change | 0.2 | 100 | 80 | 5 | 40 | 0.4 |
| Percent Reduction | 3% | 50% | 44% | 18% | 27% | 29% |

Maximum flow rate = 5.488 liters/sec (87 GPM, 0.19 cfs)
Storm Duration = 67 minutes
Rainfall @ OUC 0.254 cm (0.1 inch), @ SITE not recorded

| Site STORM 2 | pH SU | Total Suspended Solids mg/l | Turbidity NTU | BOD5-Day mg/l | COD mg/l | Total Phosphorous mg/l |
|---|---|---|---|---|---|---|
| CDS Inlet<br>CDS Inlet | 8.4 | 350 | 440 | 8.2 | 20 | 0.86 |
| Outlet | 8.2 | 350 | 340 | 8.2 | 20 | 0.86 |
| Change | 0.2 | 0 | 100 | 0 | 0 | 0 |
| Percent Reduction | 2% | 0% | 23% | 0% | 0% | 0% |

Maximum flow rate = 8.39 liters/sec (133 GPM, 0.3cfs)
Storm Duration = 68 minutes
Rainfall @ OUC 1.778cm (0.7 inch),.@ SITE 0.0762 cm (0.03 inch)

| Site STORM 3 | pH SU | Total Suspended Solids(mg/l) | Turbidity NTU | BODS-Day mg/l | COD mg/l | Total Phosphorous mg/l |
|---|---|---|---|---|---|---|
| CDS Inlet | 7.6 | 300 | 110 | 12 | 71 | 1.3 |
| CDS Outlet | 7.6 | 150 | 86 | 8.2 | 53 | 0.95 |
| Change | 0 | 150 | 24 | 3.8 | 18 | .35 |
| Percent Reduction | 0% | 50% | 21.8% | 31.7% | 25.4 | 27% |

Maximum flow rate = 149.75 liters/sec (2374 GPM, 5.29cfs)
Storm Duration = 113 niinutcs
Rainfall @ OUC 4.064 cm (I.6 inch), @ SITE 1.27 cm (0.5 inch)

# Table 3

| STORM #4 Set 1 @ initiation | BOD5-Day (mg/l) | COD (mg/l) | pH (SU) | Total Phosphorous (mg/l) | Total Suspended Solids (mg/l) | Turbidity (NTU) |
|---|---|---|---|---|---|---|
| | | | | | | |
| Inlet 1 | 2.1 | 2 | 8 | 0.32 | 690 | 99 |
| Outlet 1 | 5.4 | 2 | 7.8 | 0.19 | 320 | 120 |
| Change | +3.3 | 0 | -0.2 | -0.13 | -370 | +21 |
| Percent Reduction/Gain | +61% | 0*% | -3% | -41% | -54% | +18% |
| | | | | | | |
| Inlet 2 | 6.6 | 15 | 8.3 | 1.2 | 1400 | 1800 |
| Outlet 2 | 7 | 18 | 8.4 | 0.94 | 1600 | 1000 |
| Change | +0.4 | -3 | +0.1 | -0.26 | +200 | -800 |
| Percent +/- | +6% | +17% | +1% | -22% | +13% | -44% |
| | | | | | | |
| Inlet 3 | 6.7 | 25 | 8.2 | 1.2 | 830 | 530 |
| Outlet 3 | 6.7 | 24 | 8.3 | 1.5 | 550 | 430 |
| Change | 0 | -1 | +0.1 | +0.3 | -280 | -100 |
| Percent Reduction/Gain | 0% | -4% | +1% | +20% | -34% | -19% |
| | | | | | | |
| Inlet 4 | 6.3 | 45 | 8.1 | 1.6 | 330 | 200 |
| Outlet 4 | NT | NT | NT | NT | NT | NT |
| Change | Na | Na | Na | Na | Na | Na |
| Percent Reduction/Gain | Na | Na | Na | Na | Na | Na |
| | | | | | | |
| Inlet 5 | 5.6 | 33 | 8 | 1.6 | 290 | 300 |
| Outlet 5 | 6.4 | 30 | 8.2 | 1.6 | 170 | 260 |
| Change | +0.8 | -3 | +0.2 | 0 | -120 | -40 |
| Percent Reduction/Gain | +13% | -9% | +2% | 0% | -41% | -13% |
| | | | | | | |
| Inlet 6 | 6 | 39 | 7.9 | 1.6 | 220 | 120 |
| Outlet 6 | 6.3 | 33 | 8.2 | 1.5 | 270 | 230 |
| Change | +0.3 | -6 | +0.3 | -0.1 | +50 | +110 |
| Percent | +5% | -15% | +4% | -6% | +19% | +48% |

# Table 4

| Storm # 5 | BOD5-Day | COD | PH | Total Phosphorous | Total Suspended Solids | Turbidity |
|---|---|---|---|---|---|---|
| | (mg/l) | (mg/l) | (SU) | (mg/l) | (mg/l) | (NTU) |
| Inlet 1 | 4.6 | 68 | 7.8 | 0.23 | 49 | 16 |
| Outlet 1 | 4.0 | 18 | 7.9 | 0.18 | 11 | 4.3 |
| Change | -0.6 | -50 | +.1 | -0.05 | -38 | -11.7 |
| Percent Reduction/Gain | 13% | 74% | 1% | 22% | 78% | 73% |
| | | | | | | |
| Inlet 2 | 10 | 51 | 7.8 | 0.25 | 59 | 38 |
| Outlet 2 | 3.8 | 23 | 7.9 | 0.18 | 19 | 6.9 |
| *Change* | -6.2 | -28 | +.1 | -0.07 | -40 | -31.1 |
| Percent +/- | 62% | 55% | 1% | 28% | 68% | *82%* |
| | | | | | | |
| Inlet 3 | 13 | 55 | 8.2 | 0.3 | 23 | 23 |
| Outlet 3 | 4.7 | 33 | 7.6 | 0.18 | 21 | 12 |
| Change | -8.3 | -22 | -0.6 | -0.12 | -2 | -11 |
| Percent Reduction/Gain | 64% | 40% | 7% | 40% | 9% | 48% |
| | | | | | | |
| Inlet 4 | 9.9 | 53 | 9.2 | 0.35 | 39 | 61 |
| | | | | | | |
| Change | -6 | -24 | -1.5 | -0.17 | -24 | -53.8 |
| Percent Reduction/Gain | 61% | 45% | 16% | 49% | 62% | 88% |
| | | | | | | |
| Inlet 5 | 9.6 | 53 | 9.4 | 0.29 | 35 | *56* |
| Outlet 5 | 3.4 | 27 | 7.6 | 0.17 | 13 | 9.4 |
| Change | -6.2 | -26 | -1.a | -0.12 | -22 | -46.6 |
| Percent Reduction/Gain | 65% | 49% | 19% | 41% | 63% | 83% |
| | | | | | | |
| Average Percent Change | 53% | 52.6% | - % | 36% | *56%* | 74.8% |

**Maximum** flow **rate 0.136** liters/sec **(2.16** GPM, 0.005cfs)
**Storm Duration** =50 **minutes**
Rainfall @ OUC 1.016cm (0.4inch), @ SITE 0.5842cm (0.23inch)

# IMPERVIOUS COVER, BENTHIC COMMUNITY HEALTH, AND STORMWATER BMPS: IS THERE A RELATIONSHIP?

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## ABSTRACT

Research during the past ten years has indicated that benthic biological community health is adversely affected by as little as 10 to 15 percent impervious cover within a watershed. To assess the effectiveness of stormwater BMPs in protecting benthic communities at different levels of watershed imperviousness, sampling was performed in four locations around the country. This project examined several watersheds to identify the linkages between watershed conditions, specifically urbanization, and the habitat elements and biological responses. This paper will present the most current data associated with this project. Preliminary data analysis has revealed that measures of benthic macroinvertebrate and fish community integrity declined from the lowest levels of urbanization without exhibiting a threshold effect, although retention of natural riparian buffer partially ameliorated the decline of invertebrates. The study produced a set of conditions necessary to preserve the highest levels of biological integrity or avoid the lowest. It also is assessing the influence of structural and non-structural best management practices (BMPs) on the same ecological communities. Preliminary results demonstrate that retention of a wide, nearly continuous riparian buffer in native vegetation has greater and more flexible potential than other options to uphold biological integrity when development increases. Upland forest retention also offers valuable benefits, especially in managing any development occurring in previously undeveloped or lightly developed watersheds. Structural BMPs have less mitigation potential than the non-structural BMPs assessed and should not be regarded, as they so often are, as the leading or even the single strategy. Still, they have their place in management, especially in moderately and highly developed watersheds, to help prevent further resource deterioration, and, in dense networks along with non-structural means, in less developed basins of relatively high ecological value and sensitivity, None of the options is without limitations, and widespread landscape preservation must be incorporated if we are to keep the Nation's most productive aquatic.

**APPLICATION OF STORMWATER BMPS: PAST, PRESENT AND FUTURE**

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## ABSTRACT

BMP programs implemented in the State of Florida since the early 1980s have emphasized the use of technology-based design standards that facilitate communication between the design professional and the permitting agency, and result in a BMP application that has a presumed pollutant removal performance. The practical result of these programs, however, is that they produce environmental permits that allow new development to occur. As a result, most BMP designers have been rewarded by the receipt of an environmental permit, not by providing an innovative, cost-effective BMP design.

A different approach will be required in the future if we are to successfully address existing water quality problems, and meet the requirements of the Clean Water Act, as implemented through the Total Maximum Daily Load (TMDL) Program. Stormwater designers and local governments will need the tools and the motivation to develop and implement cost-effective BMP master plans that maximize pollutant removal for a fixed budget. This paper describes the need for new partnerships that must be formed between the regulatory, engineering and scientific communities, and local government, if desired advances in BMP planning, design and implementation are to take place in the future. Discussions are also provided that relate to future areas of growth.

## INTRODUCTION

In the past two decades, much has been accomplished in the State of Florida with regard to the design and implementation of reliable stormwater treatment technologies, popularly known as best management practices (BMPs). Since the early 1980s, new development in the State has had to provide for the treatment of stormwater runoff resulting from changes in land cover characteristics, such as grade changes and the addition of impervious surfaces. During this period, there has also been considerable progress made in the development of innovative stormwater treatment technologies, such as alum treatment, constructed wetlands, and inline sedimentation facilities, such as underground baffled vaults and swirl concentrators.

In more recent years, local governments have increasingly sought to design and construct retrofit BMPs that provide stormwater treatment for older urban areas built without adequate stormwater treatment facilities. These latter efforts have been fueled in part by participation in the National Pollutant Discharge Elimination System (NPDES) Stormwater Program by numerous municipal and county governments across the State of Florida; an increased awareness of the impact that non-point source pollution continues to have on our receiving water bodies; and the popularity of stormwater utilities, which provide a dedicated funding source for stormwater management that can address a broad range of issues, including stormwater treatment.

As the standard of practice for stormwater BMP application continues to evolve, the need for innovative and effective methods of stormwater treatment has never been greater, particularly as related to retrofit projects. Despite considerable progress in the development and application of reliable BMP technologies in the past two decades, further advancements are needed to achieve desired water quality improvements and/or prevent further degradation of Florida's surface waters. In particular, the Florida Department of Environmental Protection (FDEP) is currently in the early stages of implementing a Total Maximum Daily Load (TMDL) Program that will ultimately affect land development and stormwater treatment practices throughout the State. The goal of the TMDL Program is to implement the portion of the Federal Clean Water Act relating to the management and treatment of non-point sources of pollution, which are transported by stormwater runoff.

To fulfill the mission of the TMDL Program, advances are needed in the development and implementation of BMP master planning and design techniques that allow the identification of the most cost-effective stormwater alternative, as defined by maximizing the quantity of pollutants removed for a fixed expenditure (capital cost plus operation and maintenance costs), or minimizing cost for a fixed pollutant removal goal. Such planning methods can provide greater assurance that capital and maintenance funds are expended in the most appropriate manner; that the stated pollutant removal goals of the retrofit BMP project can be met; and measured progress can be made in regard to improving receiving water quality.

These advances, however, require the facility designer to understand the mechanisms by which pollutants are removed or transformed by BMPs, so that reliable predictions can be made regarding future BMP performance, as defined by percent of pollutants removed in the long term and the cost-per-unit quantity of pollutant removed. In addition, regulators, practitioners and BMP owners must be encouraged to try approaches that do not necessarily follow traditional BMP design paradigms, which primarily rely on technology-based ("cook book") standards that were essentially created to obtain an agency permit for new development.

**BMP Definition: "Best or Better?"**

The BMP definition included in the federal code is provided below.

"A ***best management practice (BMP) is a means of practice or combination of practices that is determined by a state (or designated area-wide planning agency) after problem assessment, examination of alternative practices, and appropriate public participation to be the most effective practicable (including technological, economic, and institutional considerations) means of preventing or reducing the amount of pollution generated by non-point sources to a level compatible with water quality goals."***

The above definition refers to the "most effective practicable (including technological, economic, and institutional considerations) means". Therefore, cost is a factor in defining a BMP, but so are technical and institutional issues. This definition also refers to the need to reduce pollution in order to reach downstream water quality goals. It is fair to state that the BMP permitting programs executed in the State of Florida since the early 1980's, which use technology-based standards, are consistent with the federal BMP definition provided above.

In a perfect world, however, the term "best management practice" would perhaps describe a treatment technology that is not only better than other technology options, but in fact is the "best" of all the technology options available. Were this true, a true BMP could only be determined following a comparison of potential stormwater treatment options, and a review of their ability to remove pollutants in the most efficient and cost-effective manner possible. Furthermore, this idealized BMP selection process would allow for performance variations due to site conditions and constraints, the pollutant(s) of concern, the percent of the total pollutant load targeted for removal, and projected operation and maintenance requirements.

We do not reside in a perfect world, however, and in general practice BMPs could perhaps be identified as "better" management practices since, although the practices adopted are typically better than some, there is seldom an attempt made to determine whether the treatment technology and design parameters selected for a specific application is indeed "best". In fact, most BMP designers rely on technology-based design standards and methodologies associated with obtaining the desired permit. Although this is a commendable goal, there is no assurance provided that the "best" treatment facility was actually provided.

**BMP Permitting Programs: What Have We Learned?**

The federal BMP definition provided above emphasizes the role that the state and/or designated area-wide planning agency (e.g., water management districts) have in the selection and design of BMP technologies. In the State of Florida, FDEP and the water management districts (WMDs) have obviously taken the lead in the design and implementation of stormwater BMPs, which has primarily occurred due to their role in the issuance of stormwater permits for new development. To facilitate implementation of these permitting programs, FDEP and the WMDs have adopted "technology-based" design standards that specify how the BMP should be constructed, and presumes that a desired level of stormwater treatment will be provided.

**As** a result, uniform BMP design standards, which emphasize the use of "better" treatment technologies, are typically applied by those seeking a stormwater permit, since the use of an innovative solution will likely delay or potentially prevent the issuance of the desired permit. Accordingly, even though stormwater treatment rules typically allow for innovation, there is little incentive to apply **an** innovative solution that may actually remove a greater fraction of pollutants and/or have a significantly lower cost-per-unit quantity of pollutant removed.

Accordingly, even though BMP permitting programs in the State of Florida have been highly successful in the issuance of permits for new development and the application of "better management practices", they have not promoted innovation on the **part** of BMP design professionals. Furthermore, these permitting programs generally do not provide BMP performance and cost data that could be used by the designer to identify the best treatment technology, such as long term pollutant removal Performance data, BMP construction and maintenance costs, and information as to how the BMP technologies actually remove pollutants.

Today, after nearly two decades of BMP permitting programs in the State of Florida, many BMP designers in this state still have a limited understanding of how stormwater BMPs actually remove pollutants, and few who can prepare a BMP master plan in a manner consistent with downstream water quality goals. This is because the typical BMP designer has been rewarded by his/her ability to obtain a permit using "cookbook" design criteria, not for their knowledge of actual BMP performance and cost.

## The Need For Future Partnerships

The purpose of this paper is certainly not to criticize the past performance and mission of the FDEP and WMDs in regard to their stormwater treatment permitting programs. FDEP and the WMDs have fulfilled their mission of bringing stormwater treatment to the mainstream, and significantly reducing pollutant loads originating from new development. The intent of this paper is to emphasis that partnerships must be formed between the regulatory, engineering and scientific communities, and local government, if desired advances in BMP planning, design and implementation are to take place in the future. Within the past year, the need for these partnerships has taken on an added importance given the goals and objectives of the FDEP TMDL Program.

To solve the serious surface water quality problems present in the State of Florida in a cost-effective manner, the BMP design profession, in association with the regulatory community, will need to advance to the next level of understanding and ability, particularly as related to retrofit BMP design and implementation. Improvement in BMP design methodologies, however, will require that future BMP permitting programs encourage appropriate understanding, behaviors, and innovation on the part of the BMP designer. The BMP design community, and BMP owners, must then respond with programs that provide proper designer training, and performance and cost data gathering.

Given the tremendous need in the State of Florida for cost-effective stormwater treatment facilities that maximize the pollutants removed for a fixed monetary budget, innovation can only occur if the BMP designer and the end user (city and county governments) participate in the development of BMP planning methods, design standards, and retrofit solutions. BMP implementation cost does matter, as defined by capital cost, operation and maintenance (O&M) cost, and ultimately the unit cost of pollutant removal (annual cost per pound of pollutant removed). Millions of dollars could potentially be spent in the next decade in the State of Florida to construct and maintain retrofit stormwater BMPs. If the ultimate goal is to obtain a cost-effective solution, the designer and the end user must have appropriate design tools, and sufficient latitude within the permitting structure, to reach this goal.

Accordingly, future advances in BMP design practice will require other entities, in addition to state agencies, to have a role in defining design standards and implementing innovative solutions, These entities include professional engineering societies and local government organizations, such as the American Society of Civil Engineers **(ASCE),** Florida Engineering Society (FES), Water Environment Federation (WEF), American Public Works Association (APWA), Florida Association of Stormwater Utilities **(FASU),** and the Florida Chamber of Commerce, Regulatory agencies must allow, and encourage, these entities to participate in the BMP planning, design and implementation process in **a** manner that will yield the "best" and most cost-effective solutions.

**Future Areas of Growth**

Future advancement in BMP master planning, design, construction and maintenance could occur in many areas, as suggested by the following discussions:

1. Master planning techniques exist that allow BMP performance (cost-effectiveness) to be related to treatment technology, BMP level of effort (e.g., pond size, swale length), and hydrologic and pollutant loading. Many of these approaches can be reduced to the application of nomographs and/or mathematical expressions readily applied by the BMP designer, which eliminates the need for **a** sophisticated modeling effort. Such design aids could be created for a region of the state that has relatively uniform hydrologic characteristics. Accordingly, BMP design nomographs could be created for the entire state in a manner similar to those created years ago by the Florida Department of Transportation (FDOT) to facilitate the design of roadway culverts. Such BMP nomographs could be developed and/or promoted by FDEP, and/or the WMDs.

2. BMP design professionals should obtain training in BMP pollutant removal mechanisms, and the effect that hydrologic and pollutant loading have on BMP performance. For instance: the fraction of pollutants removed by a BMP is a function of long-term rainfall/runoff patterns, pollutant type, and the presence of upstream BMPs. ASCE, FES, WEF and the WMDs can continue to provide seminars and other venues for the transfer of such information.

3. The application of retrofit BMPs should ideally be applied in a manner consistent with predetermined downstream water quality goals, as defined by the state and local communities. Obviously, the definition of these goals, and the methods used to influence and monitor their attainment, will largely define how retrofit BMPs are applied. Accordingly, BMP design professionals and local communities will need to remain involved in the ongoing definition of pollutant load reduction goals (PLRGs) and TMDLs.

4. **A** central database (e.g., **an** internet web site) should be established that can provide BMP designers, owners and regulators with important BMP design and maintenance information, such as performance and design data, capital costs, O&M costs, maintenance standards, etc. This BMP database could be maintained by a state agency (e.g., FDEP, WMD), or an organization of municipal **and** county governments (e.g., **FASU,** APWA).

5. BMP operation and maintenance personnel employed by municipal and county governments should receive training in proper BMP O&M techniques. Ideally, such personnel would receive certification demonstrating their participation in this training program **and** their knowledge of certain minimum standards.

## CONCLUSIONS

BMP permitting programs implemented in the State of Florida have been largely successful in bringing the treatment of stormwater runoff and the design of BMPs to the mainstream of stormwater management. Use of technology-based standards has been an effective means of communicating uniform BMP design criteria, and implementing permit programs that focused on new development. One result of these programs, however, is that BMP design innovation has not been promoted, since the main emphasis has been on obtaining the necessary permit for new construction.

Advances in BMP design and implementation that focus on the improvement of receiving water quality and the development of cost-effective applications, will require BMP designers, owners and regulators to apply innovative methods and work in partnership, This is particularly important given the developing FDEP TMDL Program, and the significant financial impact this program could have on land development and stormwater management practices throughout the state.

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# AN ASSESSMENT OF AN IN-LINE ALUM INJECTION FACILITY USED TO TREAT STORMWATER RUNOFF IN PINELLAS COUNTY, FLORIDA.

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## ABSTRACT

The Southwest Florida Water Management District, under its Stormwater Research Program, conducted a study to determine the feasibility of using an in-line alum injection facility as a stormwater treatment retrofit. To determine the effectiveness of the facility, primary efforts were directed to water quality and hydrologic data collection. An intensive flow-weighted storm event and monthly water quality and hydrologic data sampling program was conducted during a two-year period. Facility operation and maintenance problems experienced during the study emphasized the need for regulated operation and maintenance practices to assure proper and efficient facility operation.

The water quality constituents analyzed during this study included various forms of phosphorus **and** nitrogen, and several metals. Portions of these data were likely biased due to a back flow of alum in the inflow station samples. Individual storm data revealed event mean percent loads were reduced. Reductions were experienced in total phosphorus **(37.2** percent), ortho phosphorus **(42.7** percent), ammonia (24.5 percent) and nitrate-nitrite **(52.2** percent). Data indicate the alum facility could be effective in reducing phosphorus if properly maintained.

Comparisons were made of pre- and post-treatment data. Phosphorus concentrations measured downstream from the facility were generally lower and less variable after facility installation. **A** detailed analysis of the potential for aluminum toxicity to various fish and benthic species was conducted and potentially toxic concentrations of aluminum were measured.

## INTRODUCTION

Whole-lake applications of aluminum **(alum)** for phosphorus removal can be traced back to the 1960's (Jernelov, 1970). These treatments, performed primarily on northern lakes, have been shown to be effective in phosphorus reduction. Internal loads in lake bottom sediments can be the major source of phosphorus. Alum binds to phosphorus in the sediment, making it unavailable to the water column.

Several local governments in Florida have used alum for regional stormwater treatment. This treatment involves injecting liquid alum into stormwater flows before it discharges to a lake. Alum mixes with the runoff and binds to pollutants in the water column. The alum floc then settles to the lake bottom adjacent to the point of discharge providing additional treatment. Studies have demonstrated that these facilities can be effective for phosphorus removal.

The Southwest Florida Water Management District (the District) co-fund an in-line alum injection facility through the Surface Water Improvement and Management (SWIM) program. The facility was owned and operated by the Pinellas Park Water Management District. This project was designed as a demonstration project to assess the technical feasibility of using alum injection technology to treat stormwater runoff in an in-line system with limited storage volume to contain accumulated floc. SWIM requested that the District's Stormwater Research Program conduct a detailed study of the facility. The results of the study would determine whether an in-line facility could be effective in reducing loads to downstream areas while retaining the alum floc within the storage area. Additionally, the study was to conduct an environmental impact assessment. The results of this study were complicated by extended periods of alum facility shut-down and ineffective operation and maintenance practices.

The primary focus of this project was water quality. Intensive water quantity and quality data were collected upstream (inflow) and downstream (outflow) of the injection facility to calculate event mean concentration data and pollutant load reductions, Monthly pre- and post-facility construction water quality data were also collected. These data were collected at the inflow and outflow to the facility as well as stations further downstream to ascertain the effectiveness of floc containment and reveal any water quality trends. Potential toxic effects of aluminum to aquatic freshwater species were also addressed.

This paper is an abridged version of a final technical report (Carr, 1998). The report contains several aspects of the study not in this paper including rainfall characteristics, a comparison of pre- and post-treatment event mean concentration data, a comparison between predicted and measured constituent concentration percent reductions, a comparison on monthly water quality data to class III State standards, and a literature review of the environmental availability and chemistry of aluminum.

## MATERIALS AND METHODS

### Site Description

The study was conducted in the city of Pinellas **Park,** Pinellas County, Florida (Figure 1). The alum treatment facility was located at **43rd** Street immediately upstream of the third in-line infiltration pond (sump) furthest from Sawgrass Lake. Runoff from an **83** acre drainage basin flowed past the alum facility, through **a 1,128** meter section of underground culverts and the three infiltration ponds, then down **1,967** meters of open ditch to the mouth of Sawgrass Lake.

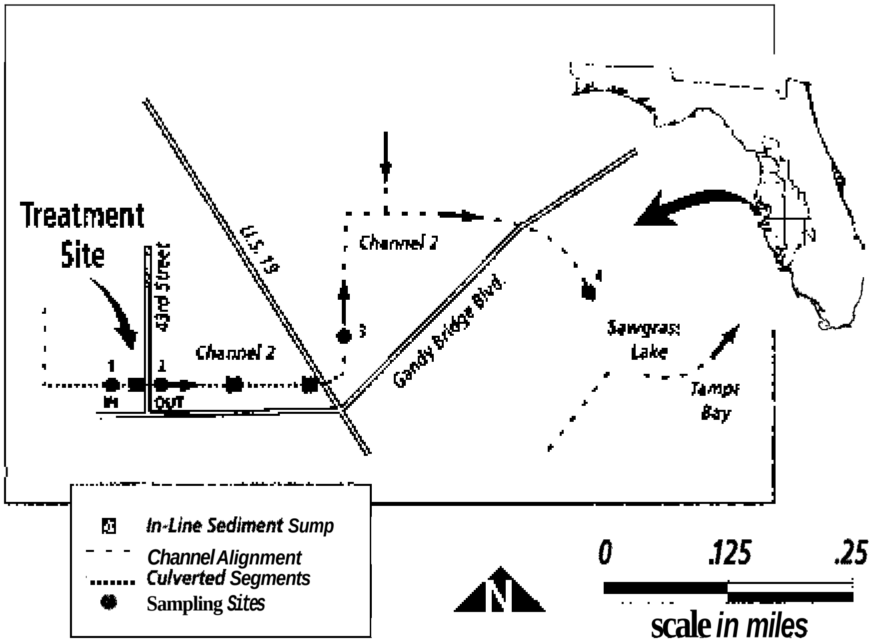


**Figure 1. Location of the alum treatment site in Pinellas Park, Florida.**

During a storm event, the inflowlpump station injected the alum just upstream of the first pond (Figure 2). The intent was to capture the alum floc in the pond. A fabric weir was constructed at the pond outflow to optimize storage volume for floc retention. At sufficient flow, the treated runoff would flow over the weir, continue **past** the outfall station on its way to Sawgrass Lake.

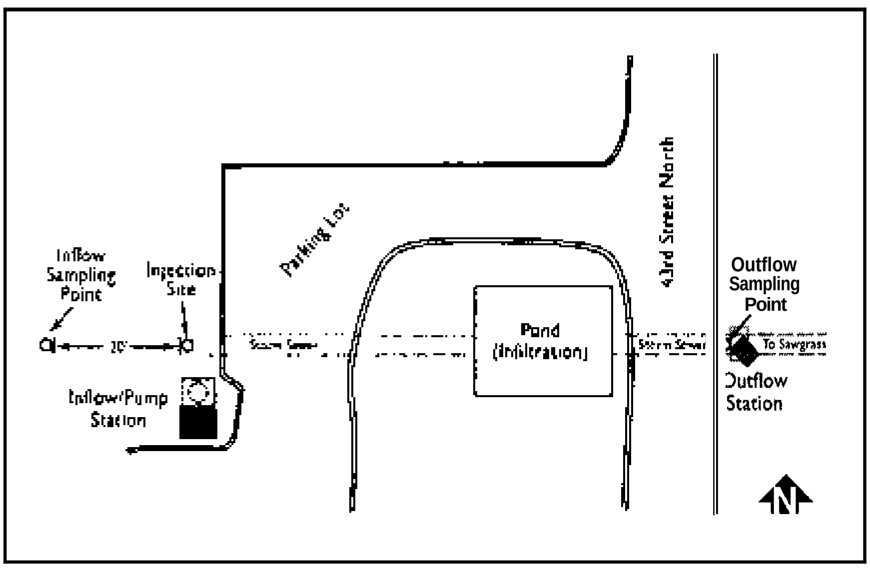


**Figure 2. Detail site map showing locations of structures and recording stations.**

## Hydrology and Water Quality

Stormwater inflow volume was measured in the 60-inch culvert using a Badgermeter 5000™ velocity meter located at the inflow station (Figure 2). The sensors were approximately 32 meters upstream of the pond. Outflow volume was measured at the outflow station using an Isco™ level "bubbler" type flow meter. A sharp crested rectangular weir was constructed at the outflow to facilitate flow calculations using this particular flow meter. The data loggers at each station measured flow data continually and recorded at 15 minute intervals. **A** Marsh McBirney™ velocity meter measured no flow from the infiltration system in the alum containment pond. Infiltration system data collection was abandoned early into the study.

Water quality samples were collected at the inflow and outflow for most storm events. The data loggers at each station triggered American Sigma™ refrigerated samplers to automatically collect flow-weighted, composite water quality samples. Storm event water quality parameters of greatest interest included total and ortho phosphorus, total aluminum, total copper, total zinc and pH.

Pollutant load reductions were used to determine the effectiveness of the alum injection facility. Mass loads at the inflow and outflow points were calculated by the event mean concentration (EMC) being multiplied by the runoff volume generated by the storm event and converted to grams. Each storm event that had complete flow and water quality data were included in the mass load calculations

Additional water quality grab samples were collected monthly at the inflow (site 1), outflow (site 2), and two stations downstream (sites **3** and **4)** (Figure 1). Monthly water quality parameters discussed in this paper include total and ortho phosphorus, total aluminum, dissolved monomeric aluminum (a fraction of dissolved aluminum), and pH. Pre- and post- alum facility operation concentration data were graphed for trend analysis. The aluminum data were compared to literature values shown to have adverse or potential toxic influence on aquatic freshwater species.

# RESULTS AND DISCUSSION

## Facility Operation and Maintenance Problems

The alum treatment facility experienced several periods of inactivity. These periods made evaluation of the alum injection facility problematic and diminished its treatment effectiveness. The facility became operational October 1995 and the start-up/training period extended through January **1996.** The facility was shut off from April **14,** 1996 to June 5, 1996 due to alum injection dosage problems. The facility was once again shut down from October **2,** 1996 to the last week in December **1996.** A lightning strike damaged several pieces of equipment including the flow meter June 26, **1997** and the facility remained inoperable until data collection was halted in September **1997.**

**A** critical component to the alum injection facility was the buffer system. This system was designed to inject sodium hydroxide simultaneously with the alum during storm events to regulate the pH of the water ($6.0 \geq pH \leq 8.0$). A potential for aluminum availability/toxicity to the biota can occur when pH is too high (>8.0) or low (<6.0). Problems were experienced with a buffer supply valve and the scale used to measure the quantity of the buffer available. The buffer scale was broken from June **3,** 1996 to September 19, 1996. During this time, **an** automatic facility shut down occurred (pH went below 6.0 **SU)** August 30, 1996 when the system ran out of the buffer during a storm event. The scale was found broken again on May 13, 1997 and was not repaired. A leak in a valve controlling buffer flow occurred December 20, 1996 and likewise was apparently never repaired.

## Storm Event Water Quality

Twelve storm events were successfully sampled using the composite flow-weighted water quality sampling method. These events had varying degrees of alum treatment due to the operation and maintenance problems during the events graphed in Figure **3.** The buffer system was ineffective during the 24Jun **96** and 7Apr97 events which resulted in pH values of **4.9 and** 4.0 **SU** respectively. The facility is designed to automatically shut down injection when pH goes below 6.0. This occurred during the 30Aug96 event. In addition to buffer system operation problems, the 26Apr97 event was under dosed when the alum tank became empty.

There seems to be an inverse relationship between phosphorus and aluminum (Figure 3). When aluminum is high (indicating **an** injection of alum), then phosphorus is reduced. Generally, the EMC of total phosphorus is high (greater than or equal to 0.15 mg/l) when low pH problems occur or when an event is under dosed. The data reveal that phosphorus is reduced when the alum facility functions properly.

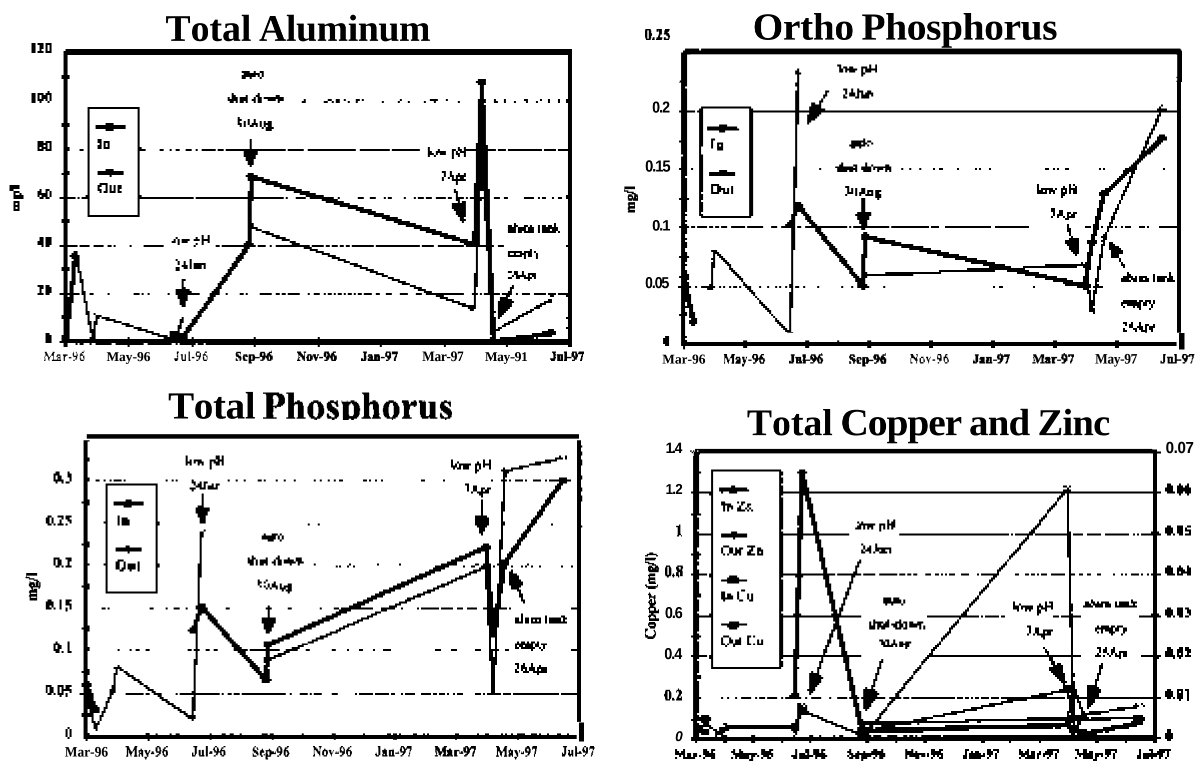


**Figure 3. Event mean concentration values at the inflow and oufflow for phosphorus, aluminum, copper and zinc.**

EMC copper and zinc values at the outflow were often higher than the inflow values (greater than or equal to 56 percent of these events). This likely occurred as the result of alum metal contamination. An aliquot of alum (sodium aluminate) was taken directly from the alum storage tank and mixed with de ionized water to a concentration of 18 mg/l alum (the alum treatment facility dosage was set at 15 mg/l). The laboratory analyses resulted in a 0.028 mg/l copper and 0.06 mg/l zinc. Therefore, the alum reasonably contributed to the EMC values of copper and zinc at the outflow.

## Load Reduction

Seven alum treated storm events were successfully sampled using the flow-weighted water quality method. Event load reduction calculations **were** performed on inflow and outflow data collected during storm events treated with alum, These load reduction calculations were used to determine the effectiveness of the alum facility.

The alum facility reduced individual storm event nutrient loads (Table 1). Total phosphorus loads were reduced in all but two storm events and had a mean load reduction of 37.2 percent. Ortho phosphorus loads were reduced in all but one storm event and had a mean load reduction of 42.7

percent. The primary intent of an alum treatment facility is to effectively remove phosphorus, not nitrogen. Nevertheless, mean ammonia percent load reduction was 24.5 percent (loads were reduced in all but one event) and mean nitrate+nitrite percent load reduction was 52.2 percent (loads were reduced in all seven events).

**Table 1.** **Mean percent load reduction for selected constituents of the treated storm events.**

| Constituent | Percent Reduction |
|---|---|
| Ammonia ($NH_3$-N) | 24.5 |
| Nitrate+Nitrite ($NO_x$-N) | 52.2 |
| Ortho Phosphorus | 42.7 |
| | |

## Monthly Water Quality

In addition to the storm event sampling, as many as 32 monthly water quality grab samples and field parameter data were collected. These monthly data were collected along the length of channel 2 (Figure 1). Twelve monthly pre-treatment and twenty monthly post-treatment samples were collected from April 1993 to September 1997. The alum facility became operational October 1995.

Monthly water quality sampling included analyses for total and ortho phosphorus. Phosphorus concentrations were lower and less variable after the facility was installed for stations 2, 3, and 4 (Figure 4) with one exception. The June 1997 data was likely elevated due to the 34.7 mm (1.4 inches) rain event that occurred the day before sampling. This change in downstream phosphorus occurred despite the fact that the data was collected between storm events and direct alum injection had not occurred. These results suggest the alum residual in the pond provides continual treatment regardless of facility operation.

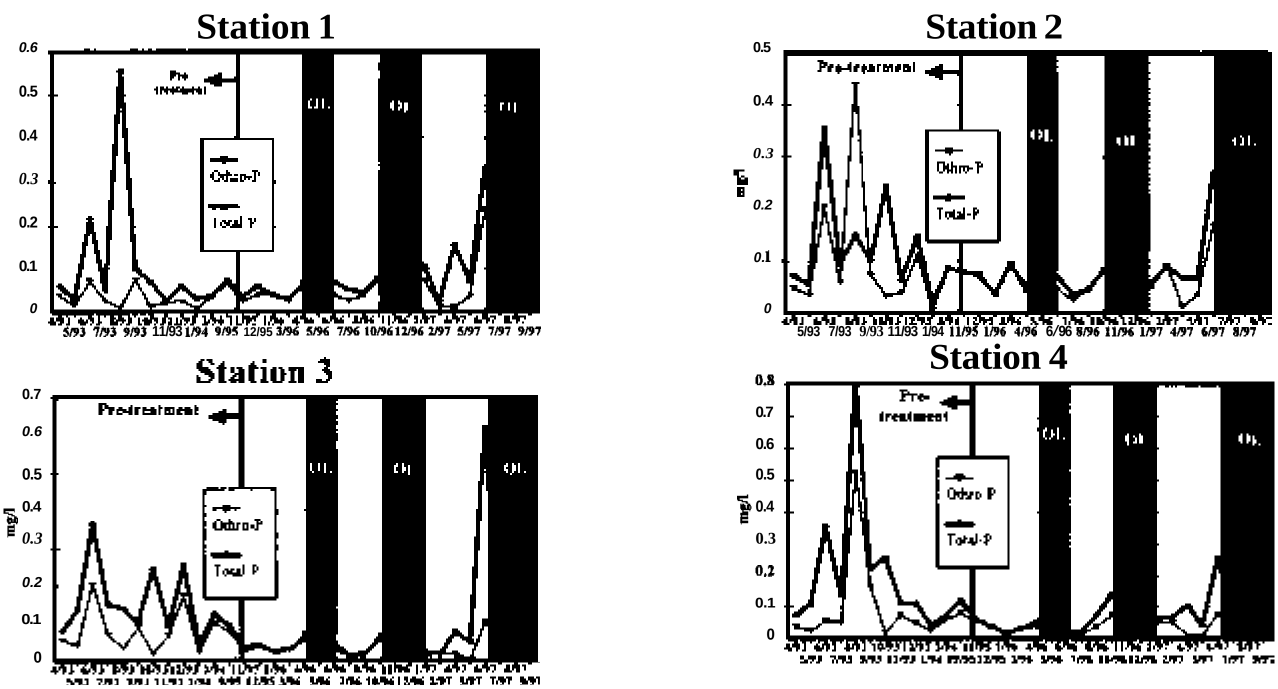


**Figure 4.** **Monthly ortho and total phosphorus concentration data from April 1993 to September 1997 at stations 1-4.**

With the exception of June 1997, post-treatment total phosphorus values during the rainy season seem to be tempered during facility operation and rose slightly when the facility was off-line (OL). Total phosphorus values would likely have been lower if facility operation was trouble-free. The data indicate the facility could be affective in reducing phosphorus if properly maintained.

## Potential Toxic Effects

DM-Al is a fraction of dissolved-Al, therefore, DM-Al value comparisons to dissolved-A1 should be considered an underestimation (Table 2). Hall et al., 1988 and Dominie, 1980 concluded that DM-Al concentrations become toxic when concentrations are greater than 0.1 mg/l and adversely affect some fish species, Comparably low levels of dissolved-A1 have been shown to adversely influence other fish and benthic organisms (**Cooke** and Kennedy 1981, Biesinger and Christersen 1972). The U.S. EPA established 0.75 mg/l dissolved aluminum as **an** acute toxic criterion and 0.09 mg/l as a chronic toxic criterion (USEPA, 1988).

DM-Al exceeded 0.1 mg/l four times at station 1 and three times at station **2** (Figure ***5).*** These concentrations can adversely affect golden shinners (Dominie, 1980) and can be toxic to striped bass larvae (Hall, et al., 1988). The February and April 1997 monthly DM-Al values at stations 1 and 2 (April only) were greater than 0.44 mg/l. ***Daphnia magna*** reproduction impairment was found to occur at these levels (Biesinger and Christensen, 1972). Station 1 and 2 April 1997 concentrations also exceeded the

| Reference | Form of Al | Conclusions |
|---|---|---|
| Biesinger and Christersen, 1972 | DM-Al | Daphnia magna (a zooplankton) reproduction was impaired 16% at 0.32 mg/l.. Populations were reduced approx. 60% after a 96hr exposure to 0.8 mg/l. |
| Cooke and Kennedy, 1981 | Diss. - Al | Established ≤ 0.5 mg/l as an upper limit based on rainbow trout toxicity. |
| Hall, et al., 1988 Dominie, 1980 | DM-Al | Becomes toxic when concentrations are >0.1 mg/l. |
| U.S. EPA, 1988 | Diss. - Al | Established toxicity criterion: acute = 0.75 mg/l, chronic = 0.09 mg/l. |

**DM-A1** concentration of 0.8 mg/l that reduced *Daphnia* population approximately 60% (Biesinger and Christensen, **1972).** DM-A1 concentrations at stations 1 (0.64mg/l) and **2 (0.92** mg/l) exceeded the USEPA toxic criteria once during the study (April **1997).** These DM-A1 levels clearly show a potential for alum treatment to adversely influence aquatic freshwater species near the point of injection.

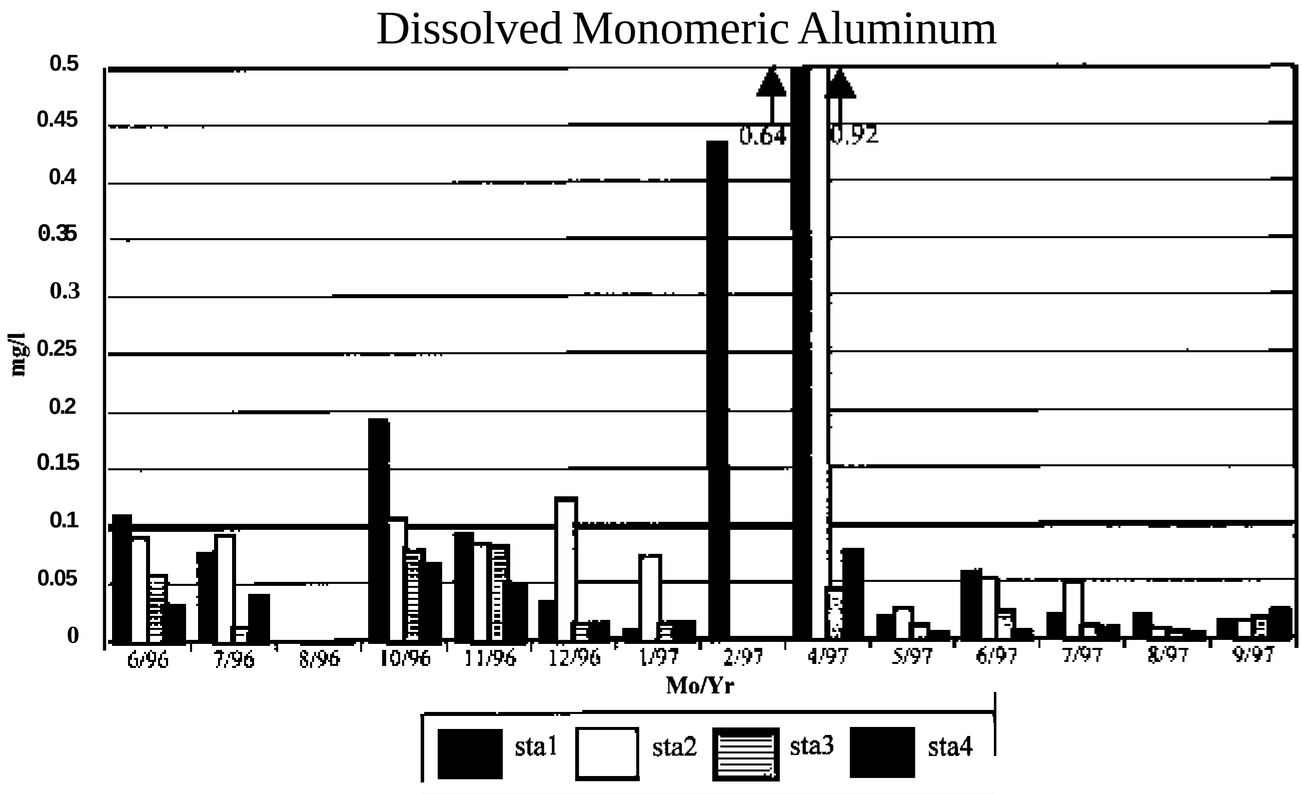


**Figure 5. Monthly DM-aluminum concentration data over time at stations 1-4.**

It was encouraging that the monthly DM-A1 concentrations at stations 3 and 4 were below possible toxic levels and avoided potential adverse effects to the aquatic environment. Unlike stations 1 and 2, stations 3 and 4 are viable ecosystems for numerous plant and wildlife species. Recall that both stations are located along the open channel portion of Channel 2 and in addition, station 4 is located at the mouth of Sawgrass Lake. The alum floc was generally restricted to the sediment pond, though the data indicates some migration to the downstream conveyance did occur. It should be noted, however, that improper operation and maintenance of the buffer system (including the buffer pH probe), could potentially lead to an acidic environment and an increase of dissolved-A1 concentrations downstream.

## CONCLUSION

This paper is an abbreviated version of a more inclusive technical report (Carr, 1998). Though the major findings were included in this paper, additional data collection and analyses were performed including: 1) data comparisons to class III State water quality standards, 2) comparisons of predicted and measured concentration percent reductions, and 3) inclusion of additional water quality analyses (i.e., metals, nutrients and total suspended solids).

### Facility Operation and Maintenance

Proper O&M is an important aspect of alum treatment facilities. The weekly and monthly observation schedule recommended by the alum facility design consultants is sufficient provided proper steps are taken to address problems as they occur. Further, it is suggested that the regulating agency take necessary precautions so that alum injection facilities are properly operated. Potential precautions include: 1) obtain assurance that sufficient funds will be available for repair/replacement of equipment, 2) require semiannual inspection reports by the facility design consultant or similarly qualified agent, and 3) require an alum treatment facility operator's certification with periodic renewal,

### Storm Event Water Quality

Several conclusions can be made from the event mean concentration data collected. Event mean concentrations (EMC) of total phosphorus and aluminum appeared to be inversely related. The concentration of phosphorus was reduced from the inflow to the outflow when the facility was maintained and operating properly. Zinc and copper values were often higher at the outflow than the inflow and were found to be alum contaminants.

### Load Reduction

The quality (EMC) and quantity (volume) of stormwater before and after alum treatment was used to evaluate the effectiveness of the facility (pollutant loads). Mean total phosphorus loads were reduced by 37 percent and ortho phosphorus loads were reduced by 43 percent. Mean ammonia and nitrate+nitrite loads were reduced 25 and 52 percent respectively. It is suggested that the pond storage volume used in this study was insufficient. More storage volume would also provide a longer residence time, which would likely improve the above load reductions.

**Monthly Water Quality**

Monthly phosphorus grab samples were conducted before and after the facility became operational. Stations included the inflow, the outflow, and two stations further downstream. Phosphorus values were generally lower and less variable after the facility began operations and was attributed to the alum treatment operations. The data suggests that the alum facility can be effective in reducing phosphorus when properly maintained.

**Potential Toxic Effects**

Data from monthly grab samples were analyzed for the monomeric species of dissolved aluminum to determine the potential affects of alum facility operations to aquatic freshwater species. Levels at stations 3 (roughly ¾ of a mile downstream) and 4 (roughly 2 miles downstream at the mouth of Sawgrass Lake) were consistently low. Potentially harmful levels were not measured at these sites. Harmful and toxic levels were measured at stations 1(inflow) and 2 (outflow), near the site of alum injection. Studies have shown that these levels can be toxic or adversely affect golden shiners, striped bass, rainbow trout and Daphnia magna (a zooplankton). The presence of these species at the study site was not in the scope of this study, but it is suggested that other fish and benthic species may be similarly affected. A study that would address the toxic effects of alum (including dissolved aluminum) on aquatic freshwater species of Florida would be of great benefit.

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# AN INNOVATIVE TAILWATER RECOVERY AND SEEPAGE-WATER INTERCEPTION SYSTEM

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## ABSTRACT

A tailwater recovery and seepage-water interception system (TRSIS) was installed on a farm near an environmentally sensitive area in Manatee County. The purpose of the TRSIS is to recover and reuse irrigation water and for water table management. Typical tailwater recovery systems include an open drainage ditch, a tailwater reservoir, a pump, and a pipeline. Open drainage ditches and tailwater reservoirs remove land from production and cause seepage and evaporation losses. They are impediments to field operations and harbor pests. This design substituted an underground perforated drain-tube and sump for the open drainage ditch and tailwater reservoir. The TRSIS has performed well, to date, and will be monitored through the 2001 spring irrigation season. Installation was accomplished under the Southwest Florida Water Management District's Agricultural Conservation Program (AgCP).

## INTRODUCTION

Balancing the need for water and its potential impact to hydrologically sensitive ecosystems is the dominate issue challenging both water resource regulators and agricultural water users in the Upper Myakka River Watershed (UMRW). Concerns about tree mortality and morbidity in the Flatford **Swamp** area of the UMRW resulted in the development of the Southwest Florida Water Management District's (District) Agricultural Conservation Partnership (AgCP) Program.

In response to the District's request for AgCP grower participation, Pacific Tomato Growers, Ltd. (Pacific) proposed to demonstrate a field-scale water conservation best management practice (BMP) using an innovative Tailwater Recovery and Seepage-Water Interception System (TRSIS). The objective of the TRSTS is to provide environmental benefits to the UMRW by providing a functional boundary between the farm and the adjacent riparian forested wetlands.

In 1994 more than 947,000 acres of commercial agricultural crops were irrigated in Florida using semi-closed seepage, or subirrigation, systems (Smajstrla, 1995). Most tomato farms in the District are irrigated using semi-closed seepage irrigation (Figure 1). With semi-closed seepage irrigation, water is pumped to the top of the field through hoses connected to buried pipes. Water travels by gravity to the bottom of the field via irrigation furrows. The water table is maintained at a depth just below the plant's root zone and water is supplied to the plant roots by capillarity (Smajstrla et **al.,** 1992).

**Figure 1. Typical semi-closed seepage irrigation system.**

Tailwater results as the field receives irrigation (Figure 2). BMPs for reducing and/or controlling tailwater include tailwater recovery, microirrigation (i.e., drip irrigation or fully enclosed seepage), and implanted reservoir tillage. Conventional tailwater recovery systems possess disadvantages that include considerable reservoir losses that may occur from deep percolation and evaporation. Waterlogged soils may result near the bottom of the field, near the tailwater reservoir, resulting in reduced crop yields. Tailwater reservoirs remove land from production and may impede farming operations. The TRSIS eliminated many of the disadvantages that have impeded wider use of tailwater recovery system.

**Figure 2. Tailwater at the end of an irrigation furrow.**

## MATERIALS AND METHODS

The TRSIS was installed adjacent to an existing **37** acre semi-closed seepage irrigated field (Figure 3). The area serviced is approximately 30 cultivated acres. The soil type is Myakka fine sand (Aeric Haplaquods), a typical high water table soil in that area.

Approximately 2,108 feet of perforated drain-tube was installed along the bottom (north) end of the field. This was in lieu of a conventional tailwater drainage ditch and reservoir. The drain-tube was installed using specialized laser-guided equipment to maintain design tolerance (Figure 4). The drain-tube was placed on a uniform 0.2 percent slope. The depth of the drain-tube ranged from 2.8 to 1.9 feet deep and was determined by the depth to the spodic horizon (Figure **5).**

Design of the TRSIS included the field measurement of saturated hydraulic conductivity (Ks) in situ in the presence of a water table. **A** simple device, developed by Rosa and Smajstrla, was used to determine the saturated hydraulic conductivity. The interception of seepage-water flow was estimated using the following equation derived by Dupuit (1**863)** on the basis of simplifying assumptions and is shown by Charny (1951) to be the analytically exact solution for flow through a horizontal drain.

$$q = \frac{kh^2}{21}$$

Interception of seepage water was verified using measurements from six water table observation wells and two automatic water level recorders (Figure **6).** The recorders were located midway (station 10+00) along the drain-tube alignment. One recorder was placed in the tomato bed near the bottom of the field; the other recorder was placed, down gradient, on the opposite side of the drain-tube near the property boundary. The recorders were approximately **60** feet apart. The recording interval was hourly. The six water table observation wells were placed, in pairs, at stations 5+00, 15+00, and in **an** adjacent field without a tailwater recovery system.

**A** portable **pump** station was constructed to reuse the tailwater and seepage-water recovered by the drain-tube. The portability of the pump station will enhance the economic feasibility of broader applications since one pump station can be shared between spring and fall crop sites. The drain-tube is connected to a sump housing the power unit's automatic controls. The automatic controls start, stop, and regulate engine speed, depending on the inflow rate of the drain-tube. This maintains the water table at the property boundary at a level consistent with natural conditions.

Tailwater and seepage-water will be pumped into **an** underground pipeline that will connect to the farm's irrigation system. **A** water meter was installed to totalize recovered tailwater and seepage-water volumes to be applied to the field.

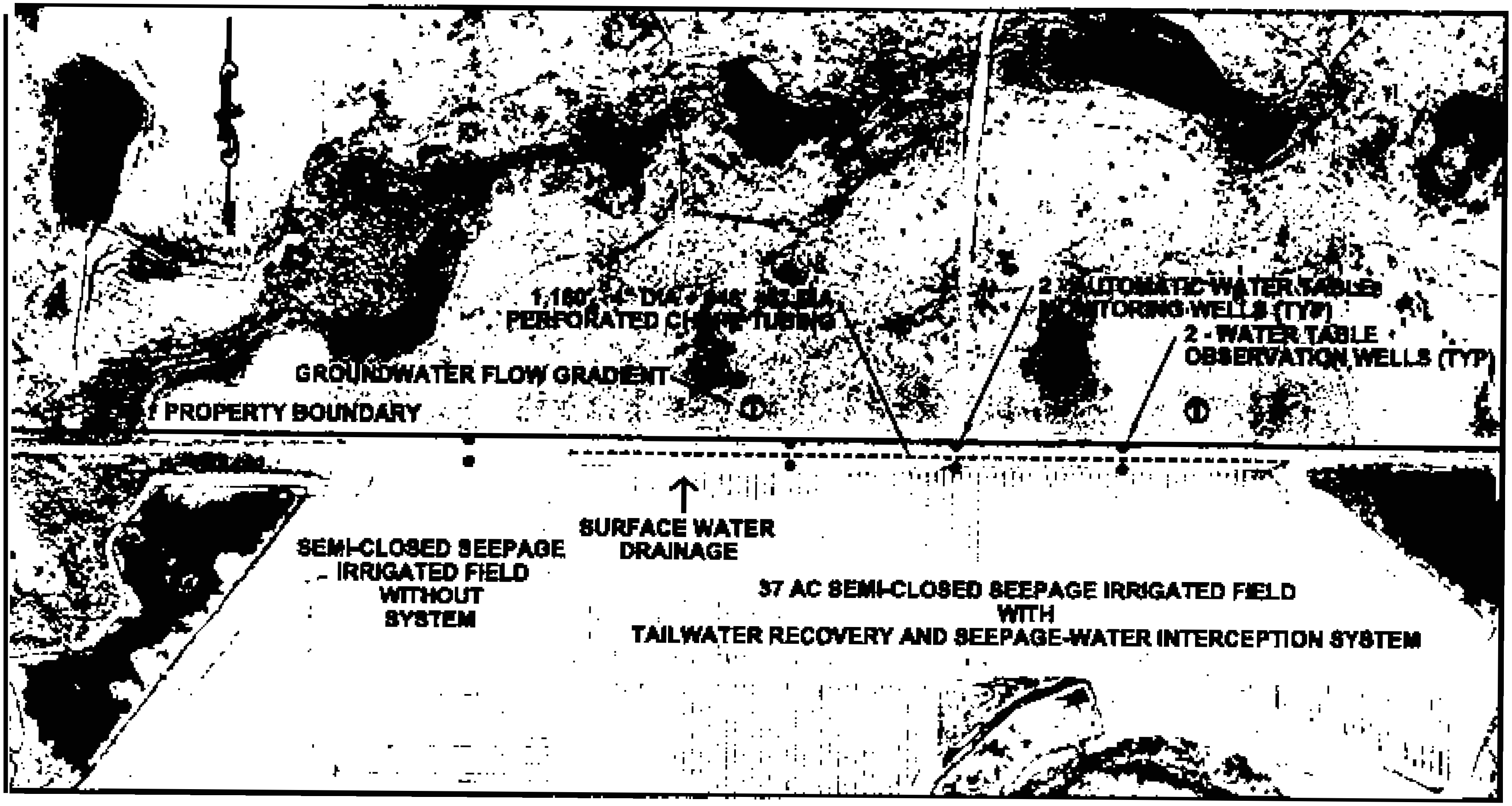


**Figure 3. Aerial Photograph.**

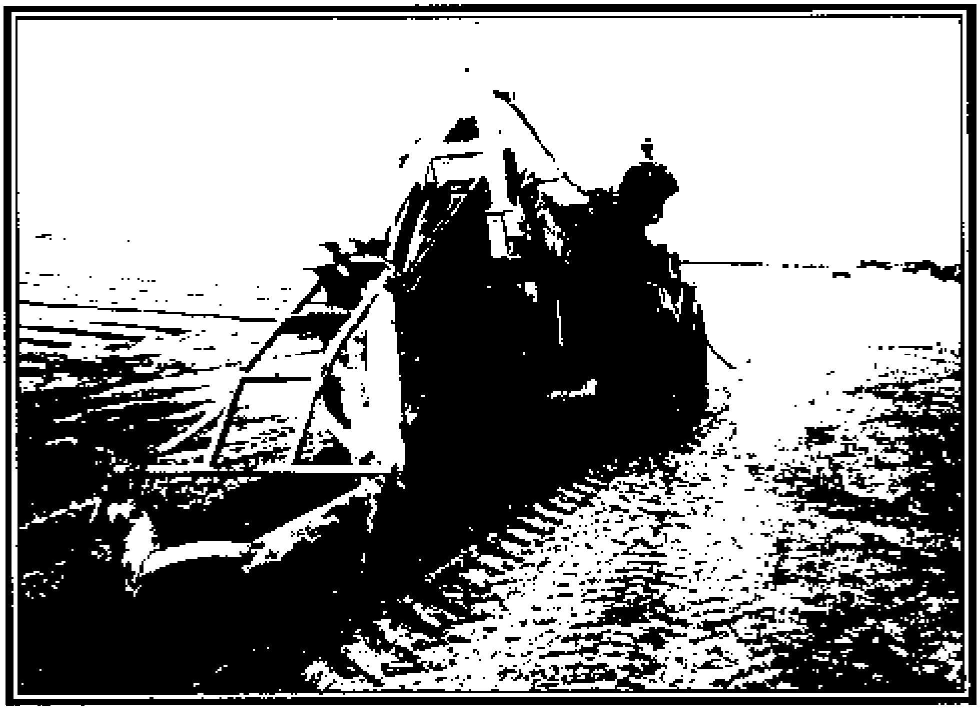

**Figure 4. Drain-tube installation.**

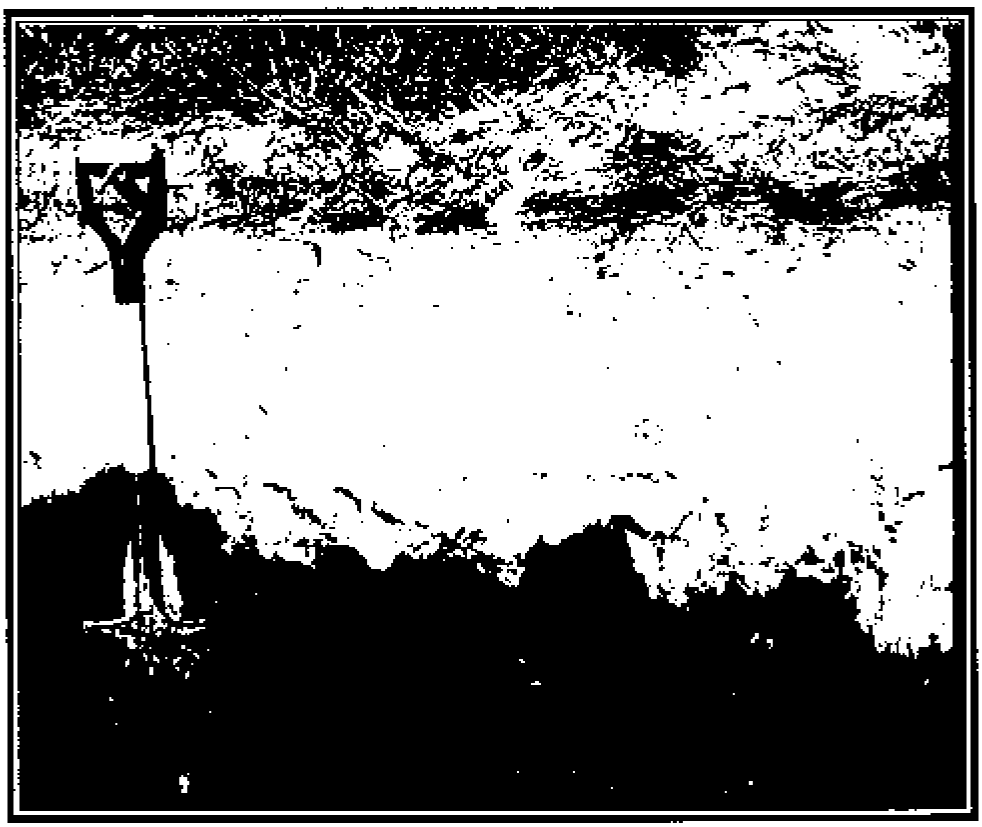

**Figure 5. Myakka soil profile (source NRCS).**

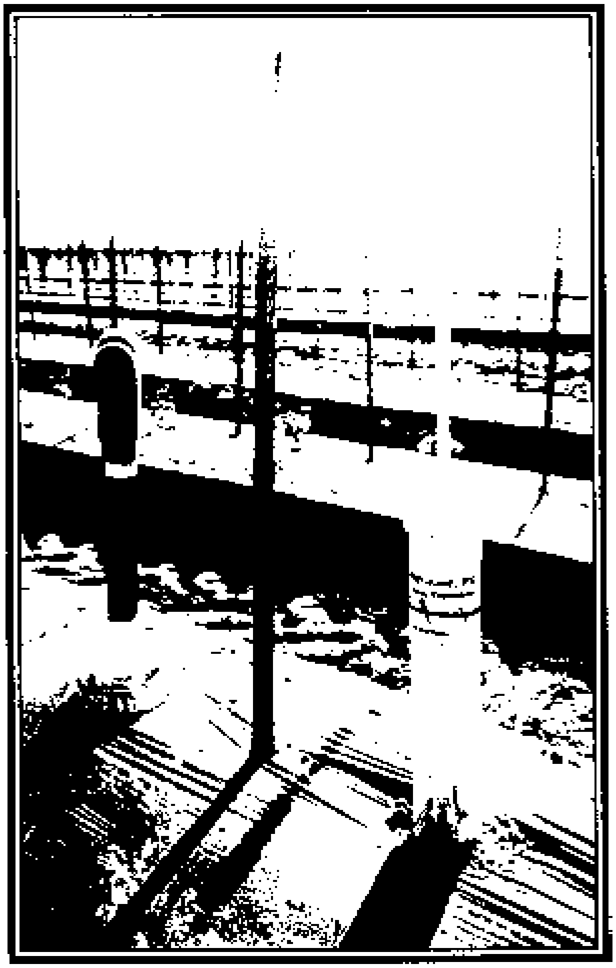

**Figure 6. Automatic water level recorder (left) and water table observation well (right).**

## KKSULTS

Tomatoes wcrc produced during the spring, 1999 growing season. The tomatoes were planted on February 22 and harvested 95 to 102 days later, ending on Junc 4. Production practices typical of the industry were used.

The TRSIS started automatic operation on March 19. Water was pumped into an adjacent pond during the initial calibration phase. Low engine speed pumping rates were measured at approximately 50 gallons per minute (gpm). High engine speed pumping rates were measured at approximately 100 gpm. The TRSIS operated for 77 days, ending on June 4. The total volume pumped was nearly 2,500,000 gallons. The average daily pumpage was more than 32,000 gallons.

All data recorded by the automatic water level recorders are shown in Figure **7. A** one-week period corresponding to a time of peak crop irrigation demand is shown in Figure **8.** Irrigation and rainfall events are labeled on the day they occurred.

TRSJS cost estimates are listed in the following table. Total costs are calculated for 1,000 gallons of water pumped during the 1999 spring season, and extrapolated for **a** 150 day water reuse duration.

| Annual Fixed Costs | ($) |
|---|---|
| New Construction[1] | 20,450.00 |
| Salvage Value | 0.00 |
| Average Cost | 10,225.00 |
| Years of Life[2] | 10 |
| Depreciation[3] | 2,045.00 |
| Interest[4] | 818.00 |
| Insurance[5] | 102.00 |
| Taxes[6] | 102.00 |
| Repairs[7] | 205.00 |
| Total | 3,272.00 |
| Annual Variable Costs | |
| Fuel | 100.00 |
| Total Annual Fixed **and** Variable Costs | **3,372.00** |
| Total Cost per 1,000 gallons Pumped (**77** Day Duration) | 1.35 |
| Total Cost per 1,000 gallons Pumped (150 Day Duration) | 0.70 |

1. Installation of drain-tube, pipeline, pump, and power unit, including engineering and surveying.
2. Based on a 10 year loan.
3. New cost divided by years of life.
4. Average costs multiplied by 8 percent.
5. New cost multiplied by 0.5 percent.
6. **Average costs** multiplied **by** 1.0 percent.
7. New cost multiplied by 1.0 percent.

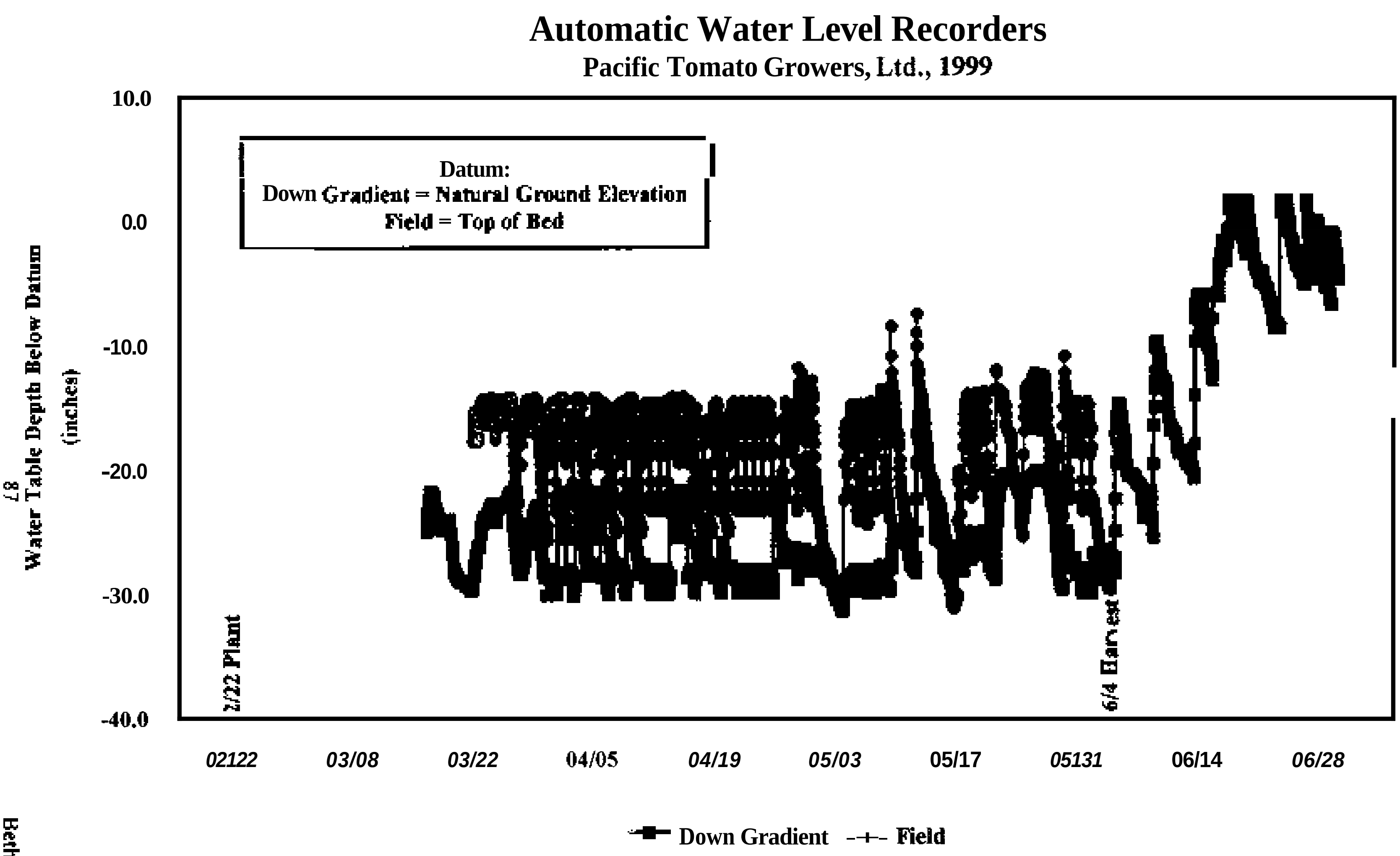


Figure 7. Hourly water table levels for period of record.

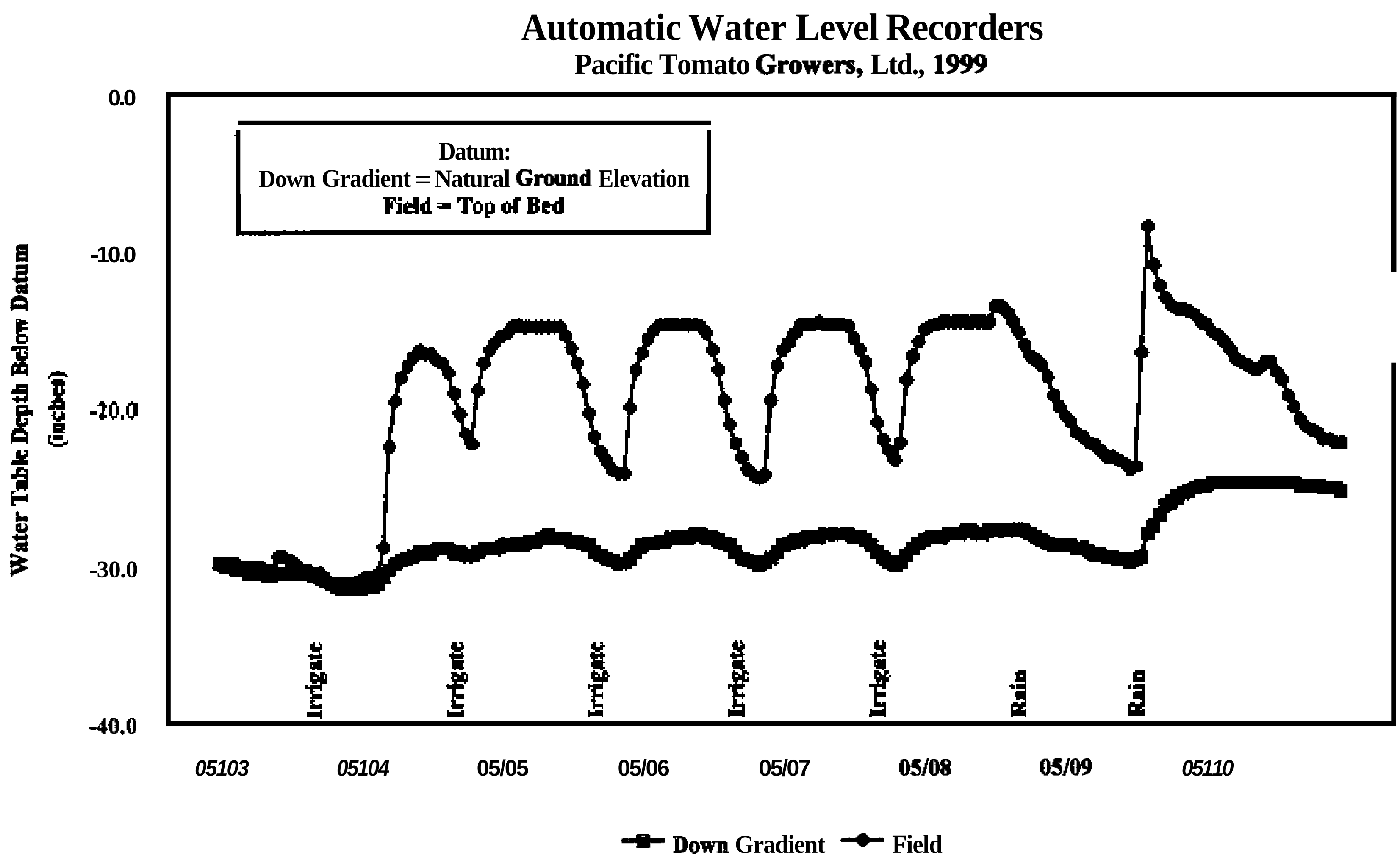


Figure 8. Hourly water table levels during peak crop irrigation demand.

## CONCLUSIONS

The TRSIS performed very well during the period it operated in the 1999 spring tomato season. Water table levels near the end of the farm fields did not appear to adversely effect crop quality or yield. In fact, disease incidences may decrease due to a reduction in saturation at or near the surface. Water table levels down gradient from the farm fields were intercepted by the **TRSIS** and maintained at lower, more natural levels.

The TRSIS offers the same environmental benefits that conventional tailwater recovery systems provide, chiefly, to control offsite runoff. Advantages include a reduction of offsite runoff of tailwater, a reduction of offsite seepage of groundwater, conservation through irrigation reuse, and minimizing land removed from farming operations. Additional advantages include improved water quality achieved through reduced nutrient losses and reduced flood potential by decreasing runoff.

Disadvantages include the cost to construct and operate the TRSIS which are significantly greater than typical groundwater pumping costs. Other possible disadvantages including crop response to reuse water and drain-tube clogging were not determined.

In summary, the Tailwater Recovery and Seepage-Water Interception System offers the grower another Best Management Practice option. However, costs appear to be 15 to 30 times greater than current groundwater pumping costs. This economic disincentive to the grower will be an impediment to wider use.

## ACKNOWLEDGMENTS

Thanks are due to Ron Cohen, P.E., District Irrigation Engineer, for his review of this project.

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# LOW IMPACT PARKING LOT DESIGN REDUCES RUNOFF AND POLLUTANT LOADS

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## ABSTRACT

An innovative parking lot at the Florida Aquarium in Tampa, Florida is being used as a research site and demonstration project to show how small alterations to parking lot designs can dramatically decrease runoff and pollutant loads. Three paving surfaces are compared as well as basins with and without swales to measure pollutant concentrations and infiltration. Preliminary results from sixteen storms indicate that for rainfall less than two centimeters, the basins with swales and permeable paving have 85 to 95 percent less runoff than the basins without swales, and 60 to 80 percent less runoff than the other basins with swales. Larger rain events do not show as much difference in runoff amounts from different paving types but basins with swales have about 40 percent less runoff than the two basins without swales. Rainfall water quality and quantity are also evaluated and rain is found to be a significant input for inorganic nitrogen. Other water quality data show higher phosphorus concentrations in basins with vegetated swales, and higher metal concentrations in basins paved with asphalt rather than cement or permeable paving. Sediment samples exhibit the same trends as water quality samples with higher phosphorus concentrations in basins with swales and higher metal concentrations in basins paved with asphalt. Polycyclic Aromatic Hydrocarbons (PAH) and pesticides were detected in the sediments at almost all sites sampled.

## INTRODUCTION

Impervious surfaces, such as parking lots and roof tops, cause more stormwater runoff and pollutant loads than any other type of land use. As little as ten percent impervious surfaces in the watershed can begin to impact downstream rivers, lakes and estuaries (Shaver *et al*. 1995). These hard surfaces which often replace natural vegetative cover increase the volume and peak rate of runoff and also provide a place for traffic-generated residues and airborne pollutants to accumulate and become available for washoff. Detention of stormwater within recessed landscaped depressions is a technique used in many localities to deal with the problem of increased runoff peaks from relatively minor storms. This practice in itself can also decrease nonpoint source pollution. This study quantifies how much runoff and pollutant loads can be reduced by using swales and landscaped depressions in parking lots. (In this report, swales are defined as vegetated open channels that infiltrate and transport runoff waters and the strands are larger channels collecting runoff after treatment by swales).

To have swales in the parking lot without reducing the number of parking spaces, local rules had to be altered. Changing the rules by making each parking space 61 centimeters shorter provided drainage depressions between parking rows. Now the front end of vehicles hang over a 122 centimeter-wide grassed swale instead of pavement. To determine how these modifications and paving types might change runoff amounts and pollutant concentrations, water quality and quantity were measured in eight small basins in the parking lot.

## METHODS

The parking lot design for the Florida Aquarium uses the entire drainage basin for low-impact stormwater treatment. The study site is a 4.65 hectare parking lot serving 700,000 visitors annually. The research is designed to determine pollutant load reductions measured from three elements in the treatment train: Different treatment types in the parking lot, a planted strand with native wetland trees, and a small pond used for final treatment (Figure 1).

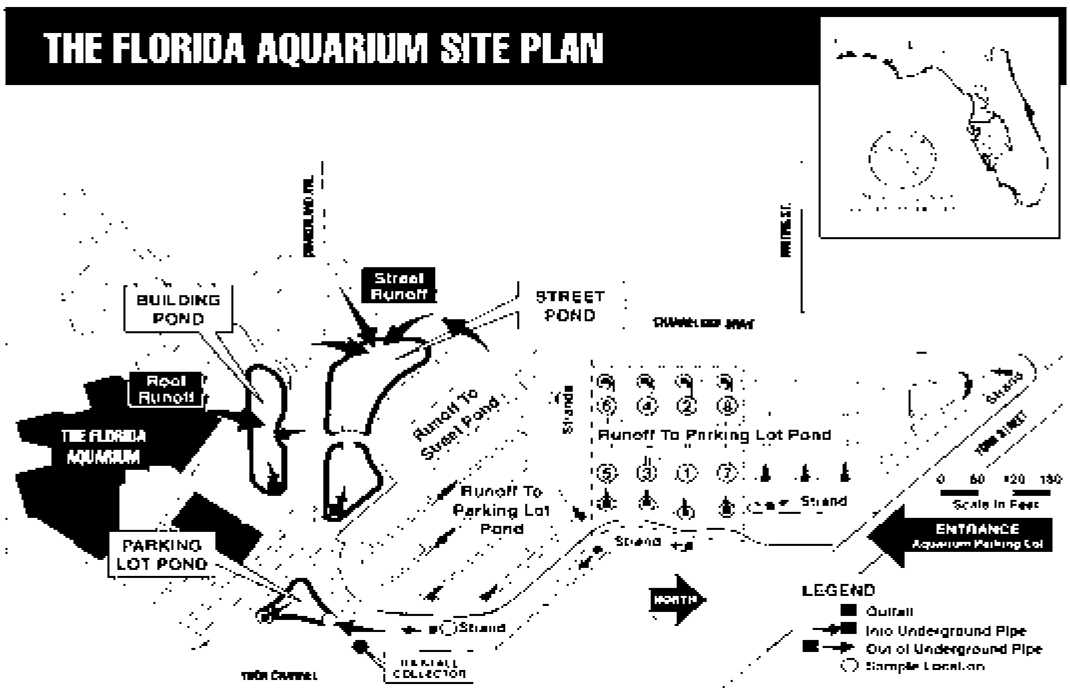



**Figure 1. Site Plan of the Parking Lot Demonstration Project. The eight drainage basins evaluated in this study are outlined by the dotted lines.**

Only the data collected in the parking lot for sixteen storm events are evaluated in this report except for the sediment data which also includes samples from the strand and the pond.

The experimental design in the parking lot allows for the testing of three paving surfaces as well as basins with and without swales creating four treatment types with two replicates of each type. The eight basins have been instrumented to measure discharge amounts and take flow weighted water quality samples during storm events. The four treatment types include: 1) asphalt paving with no swale (typical of most parking lots), 2) asphalt paving with a swale, 3) cement paving with a swale, and 4) permeable paving with a swale. The swales are planted with a shaggy native grass (sand cord grass) that never has to be mowed.

Rainfall amounts were measured for each storm with a tipping bucket rain gauge while rainfall water quality samples were gathered using a collector that is open to the atmosphere only during rain events. Flow out of each of the eight small parking lot drainage basins (0.09 to 0.105 ha) was measured using H-type flumes and shaft encoders (float and pulleys) connected to data loggers. Water quality samples were collected on a flow-weighted basis and stored in iced samplers until picked up, fixed with preservatives and transported to the Southwest Florida Water Management District (SWFWMD) laboratory. Samples were analyzed using standard methods and following SWFWMD's approved quality assurance plan (SWFWMD 1998).

Runoff coefficients (RC) and LOADS were calculated using the following formulae:

*RC = (volume discharged) / ((basin size)*(rainfall amount))*

*LOADS = ((concentrations)*(volume discharged))/((ratio of yearly rain))*(basin size))*

Sediment samples were collected in each of the swales, two locations in the strand and two locations in the pond during the fall of 1998. Samples were extracted intact from the sediments using a two-inch diameter hand driven stainless steel corer. Cores were collected at two depths, the top 2.54 cm of sediments and sediments 10 to 13 cm deep, but only the surface samples are discussed in this report. Four to five cores in the same vicinity were necessary to collect enough sample to analyze for particle size, metals, nutrients, pesticides and polycyclic aromatic hydrocarbons. Cores at each location were mixed using the four corners method and other procedures outlined in the laboratory's approved quality assurance plan (SWFWMD 1998).

## RESULTS AND DISCUSSION

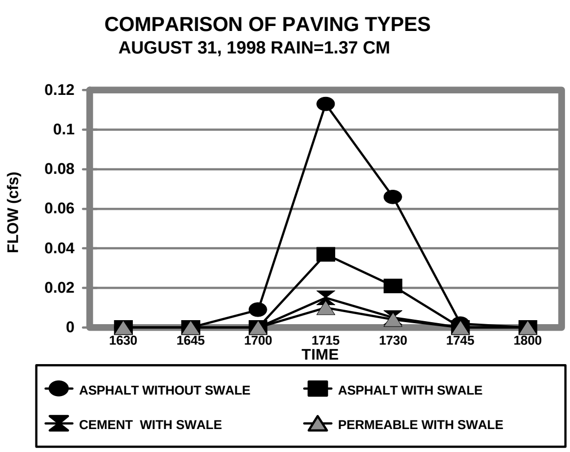
Data for the first sixteen storm events are reported here with emphasis on hydrology, water quality and sediment analyses.

**Hydrology** - Calculations of runoff volumes and coefficients for each basin clearly show the reduction in runoff that results from even these small swales and garden areas (Table 1). Except for basin F1, the odd numbered basins are slightly smaller and have larger recessed garden areas than the even numbered basins. The larger garden areas (about the size of one parking space) account for the 40 to 50 percent lower runoff coefficients calculated for the odd numbered basins. It should be noted that Florida normally has sandy soils, but that this site has been bulldozed and altered many times for redevelopment, perhaps making the soils even more permeable. From soil analyses, this location had higher gravel content (average 8.9%) than any of our other research sites. This may account for the good infiltration rates. It may also have increased the infiltration measured for permeable paving during small rain events.

**Table 1. Total runoff amounts ($M^3$) and average runoff coefficients (RC) for eight basins in the Florida Aquarium Parking Lot calculated for sixteen rain events occurring between August 1, 1998 through March 14, 1999.**

| | Asphalt no swale | | Asphalt with swale | | Cement with Swale | | Permeable w/ swale | |
|---|---|---|---|---|---|---|---|---|
| | F1 | F2 | F7 | F8 | F3 | F4 | F5 | F6 |
| Basin size ($M^2$) ==> | 10522 | 10522 | 971 | 10522 | 931 | 10522 | 931 | 10522 |
| Volume Discharged TOTAL $M^3$ | 337.72 | 294.11 | 101.28 | 194.12 | 156.27 | 219.05 | 94.67 | 144.55 |
| Runoff Coefficient AVERAGE RC | 0.60 | 0.54 | 0.16 | 0.32 | 0.20 | 0.35 | 0.10 | 0.20 |

Graphical examples show how different sized rain events affect runoff amounts (Figure 2). For these comparisons only the even numbered basins were used because they are all the same size and have the same size garden areas. Instead of a grassed swale, the basin without a swale has a recessed asphalt area the same size as the planted swales in the other treatments, but it still has recessed garden areas just like the rest of the basins. The garden areas account for its relatively low runoff coefficient (0.60) for a parking lot. These results demonstrate how even small areas in parking lots can increase infiltration.



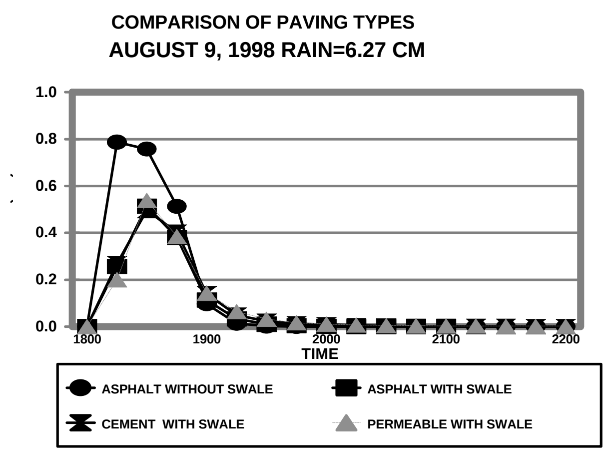




**Figure 2. Comparison of storm runoff amounts with the amount of rainfall show that swales reduced runoff for all events, and that paving type, especially permeable paving, was effective in reducing runoff from storms with less than two cm of rainfall (cement with swale has a few parking spaces with permeable paving). Graphs have different scales.**

**Water Quality Concentrations** - The average concentrations of constituents measured in each of the basins show some differences between paving types and depression storage (Table 2).
The exceptions were nitrogen species where average ammonia concentrations in rain were measured higher than at the outfalls of the basins and nitrate concentrations were about the same. In contrast,

phosphorus concentrations were much lower in rainfall and the highest concentrations of orthophosphorus were measured in basins where runoff had traveled through grassed areas.

Some metals in runoff reflected the type of paving material it traveled over. Iron, manganese, lead, copper and zinc were measured at higher concentrations in the basins paved with asphalt (F1,

Table 2. Average concentrations of constituent measured for eight rain events occurring between August 1 and September 18, 1998
LOD=Laboratory lower limit of detection. For values below the detection limit one half the detection limit was used for calculations of averages.

| Constituents | LOD | unit | Rain | Asphalt no Swale | | Asphalt with Swale | | Cement with Swale | | Permeable with Swale | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | F1 | F2 | F7 | F8 | F3 | F4 | F5 | F6 |
| No. Observations | | | 17 | 18 | 17 | 11 | 11 | 11 | 14 | 7 | 11 |
| Ammonia-Nitrogen | 0.03 | mg/l | 0.081 | 0.065 | 0.059 | 0.037 | 0.080 | 0.016 | 0.039 | 0.079 | 0.014 |
| Nitrate-Nitrogen | 0.01 | mg/l | 0.125 | 0.142 | 0.154 | 0.090 | 0.152 | 0.117 | 0.164 | 0.145 | 0.119 |
| Total Nitrogen | 0.06 | mg/l | 0.304 | 0.360 | 0.337 | 0.409 | 0.521 | 0.275 | 0.451 | 0.449 | 0.363 |
| Ortho-Phosphorus | 0.01 | mg/l | 0.009 | 0.032 | 0.048 | 0.080 | 0.246 | 0.098 | 0.223 | 0.034 | 0.085 |
| Total Phosphorus | 0.01 | mg/l | 0.013 | 0.113 | 0.083 | 0.113 | 0.311 | 0.120 | 0.249 | 0.047 | 0.075 |
| Copper | 1.00 | ug/l | 4.70 | 8.70 | 8.79 | 6.75 | 10.66 | 4.19 | 3.95 | 2.39 | 3.25 |
| Iron | 30.0 | ug/l | 74 | 320 | 336 | 195 | 296 | 76 | 106 | 88 | 66 |
| Manganese | 0.60 | ug/l | 1.90 | 9.20 | 10.10 | 6.00 | 11.20 | 2.00 | 6.50 | 2.30 | 1.60 |
| Lead | 2.00 | ug/l | 1.00 | 3.50 | 3.80 | 3.00 | 3.90 | 1.10 | 1.40 | 1.10 | 1.10 |
| Zinc | 30.0 | ug/l | 39.4 | 38.3 | 41.4 | 28.2 | 36.0 | 17.3 | 20.3 | 15.7 | 17.3 |
| Suspended Solids | 0.05 | mg/l | 0.57 | 7.63 | 10.99 | 4.91 | 13.55 | 1.79 | 6.98 | 4.36 | 2.53 |
| Chloride | 0.05 | mg/l | 1.17 | 1.32 | 1.06 | 1.48 | 1.91 | 1.21 | 1.62 | 1.48 | 1.91 |
| Potassium | 0.07 | mg/l | 0.09 | 0.31 | 0.24 | 0.70 | 1.23 | 2.14 | 2.57 | 1.57 | 2.13 |
| Sodium | 0.06 | mg/l | 0.52 | 0.72 | 0.59 | 0.81 | 0.72 | 1.00 | 1.15 | 1.05 | 0.86 |
| Sulfate | 0.05 | mg/l | 2.10 | 3.10 | 2.79 | 2.94 | 3.48 | 2.74 | 3.76 | 3.22 | 3.52 |
| Hardness | 0.02 | mg/l | 0.91 | 19.48 | 21.35 | 20.21 | 21.91 | 23.03 | 34.30 | 66.47 | 31.49 |

F2, F7, F8) than in the basins paved with cement products (F3, F4, F5, F6). Suspended solids showed no consistent pattern and were generally measured at low levels when compared to other stormwater studies. Copper showed a relationship with rainfall and the concentrations of copper in rainfall were probably the result of several ship dry dock operations in the vicinity and the city incinerator adjacent to the site.

Many of the major ions also reflected the composition of the paving material. Cement which is made with limestone was an example. Potassium, sodium, sulfate and calcium concentrations were much higher in the basins paved with cement products (F3, F4, F5, F6), but these concentrations for major ions were far below levels considered detrimental to the environment.

**Water Quality Loads** - A more realistic measurement for understanding the impact of stormwater on receiving waters is to look at pollutant loads. The most effective method for reducing pollutant loads is to keep runoff on site and allow time for infiltration as well as for chemical, biological and hydrological processes to take place. Yearly loads discharged from each basin type are calculated for a few constituents in Table 3. Since more water was discharged from the basins without swales (F1, F2), they also had much higher loads for all the constituents with the possible exception of phosphorus. When concentrations and loads measured at the Florida Aquarium parking lot are compared to other stormwater studies conducted in Florida, the values are much lower than those measured at the other sites (Harper 1994). This may reflect the fact that we sample all storm events or it may emphasize the differences between years, especially when comparing rainy and drought years, as will be discussed in the next section.

**Table 3. Yearly loads are calculated for each pavement type. Thirty-eight percent of the average yearly amount of rainfall was measured during the eight months of data presented in this report and this ratio was used to calculate estimated yearly loads.**

| Constituents | units | Asphalt no swale F1,F2 | Asphalt with swale F7,F8 | Cement with swale F3,F4 | Permeable w/swale F5,F6 |
|---|---|---|---|---|---|
| Ammonia | kg/ha-yr | 0.61 | 0.16 | 0.15 | 0.20 |
| Nitrate | kg/ha-yr | 1.02 | 0.36 | 0.64 | 0.44 |
| Total Nitrogen | kg/ha-yr | 2.20 | 1.48 | 1.27 | 1.12 |
| Total Phosphorus | kg/ha-yr | 0.40 | 0.29 | 0.55 | 0.22 |
| Total Copper | kg/ha-yr | 0.06 | 0.04 | 0.02 | 0.01 |

**Caveat** - The data presented in this report are preliminary and represent samples taken for only eighteen storm events. Since small storms produced little runoff, especially in the basins with permeable paving, some of the conclusions were based on as few as seven water quality samples. In a previous study at this site, data were collected for rainfall and at station F1 for more than 30 rain events during a ten-month period (April 2, 1997 to February 2, 1998). Samples were collected using the same techniques and analyzed in the same laboratory, but the results were much different.

A comparison of the average values between the previous study (Rushton 1997) and this current study indicates that much higher average concentrations of pollutants may be measured later when data are collected for two years. Average values for the two studies are compared in Table 4. The results may also reflect cycles of wet and dry years, for example, the 1997 data in Table 4 were collected during an "El Nino" year with considerably more rain than normal while the data in this report were collected during a drought year with practically no winter storms. Also the time of year samples are collected may influence results.

**Table 4. A comparison of average constituent concentrations at site F1 and in rainfall for two different years demonstrate the variability that can result between years.**

| Constituents | LOD (Lab Detection Limit ) | Units | RAINFALL | | | | RUNOFF (F1) | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | 1997 | | 1998 | | 1997 | | 1998 | |
| | | | n | avg | n | avg | n | avg | n | avg |
| Ammonia-N | 0.03 | mg/L | 46 | 0.22 | 17 | 0.08 | 47 | 0.17 | 18 | 0.07 |
| Nitrate-N | 0.01 | mg/L | 46 | 0.28 | 17 | 0.13 | 47 | 0.33 | 18 | 0.14 |
| Organic-N | 0.06 | mg/L | 45 | 0.17 | 17 | 0.10 | 47 | 0.43 | 18 | 0.15 |
| Ortho-P | 0.01 | mg/L | 46 | 0.02 | 17 | 0.01 | 47 | 0.05 | 18 | 0.03 |
| Total Phosphate | 0.01 | mg/L | 46 | 0.02 | 17 | 0.01 | 47 | 0.10 | 18 | 0.11 |
| Copper | 1.00 | ug/L | 40 | 4.69 | 17 | 4.79 | 45 | 23.2 | 18 | 8.66 |
| Lead | 2.00 | ug/L | 40 | 1.34 | 17 | 1.00 | 45 | 5.72 | 18 | 2.50 |
| Zinc | 30.0 | ug/L | 39 | 24.9 | 17 | 39.4 | 45 | 80.2 | 18 | 38.3 |
| Iron | 30.0 | ug/L | 40 | 40.2 | 17 | 74 | 45 | 425 | 18 | 320 |

In another example, when the values reported here are compared to an earlier version of this study which evaluated only the first eight rain events that were collected during the end of the summer rainy season, inorganic nitrogen levels are much lower, but almost all the other values are higher in this updated report. These differences emphasize the fact that inorganic nitrogen concentrations are higher in the summer while the longer inter-event dry periods occurring in the fall and winter increased the other constituents and this result was especially true for metals. Our other studies have also measured differences between seasons in rainfall with concentrations of nitrate and ammonia significantly higher during the summer months in Tampa, Florida (Rushton 1994).

**Sediment Samples -** Soil samples were collected in the swales at two depths and also in the drop boxes that the swales discharge to, but only the surface samples are discussed in this paper. In addition, samples were collected in the strand and in the pond to compare with swale samples. For the basins without swales, the sediments that had accumulated in the asphalt depressions were analyzed. For samples collected in swales, metals were detected at all sites (Table 5), and the pattern was similar to the water concentrations measured for storm runoff (see Table 2) where metals were detected at higher concentrations in the swales paved in asphalt instead of grass. The nutrient concentrations measured for organic nitrogen (TKN) and total phosphorus, were usually found at lower concentrations in the basins without grassed swales (F1 and F2). The sediment samples taken at two locations in the strand (that collects runoff from the swales) and the wet-detention pond (that is used for final treatment) are also compared to the swale samples. For metals, the concentrations were much lower than the basins paved with asphalt. Since most of this five-year-old parking lot is paved in asphalt, this indicates that most of the metals are being settled

**Table 5. Constituent concentrations measured in the sediments in each of the swales and also in the strand and the pond. The abbreviation "det" indicates the constituent was detected, but was below the laboratory limit of quantification.**

| METALS & NUTRIENTS | | asphalt no swale | | asphalt with swale | | cement with swale | | permeable with swale | | strand | | pond | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | F1 | F2 | F7 | F8 | F3 | F4 | F5 | F6 | S9 | S10 | P11 | P12 |
| Aluminum | ug/kg | 3590 | 4300 | 4650 | 3410 | 1250 | 1670 | 1450 | 1670 | 9530 | 8190 | 2990 | 2260 |
| Cadmium | ug/kg | 1 | 1 | 2 | det | det | det | det | det | det | det | det | det |
| Chromium | ug/kg | 13 | 13 | 30 | 11 | 7 | 8 | 10 | 8 | 42 | 41 | 13 | 10 |
| Copper | ug/kg | 92 | 38 | 81 | 86 | 22 | 17 | 23 | 12 | 7 | 25 | 103 | 11 |
| Iron_271 | ug/kg | 8940 | 7970 | 10500 | 6730 | 1440 | 1320 | 1400 | 1430 | 6560 | 5750 | 3190 | 2980 |
| Lead | ug/kg | 22 | det | 29 | 22 | 15 | 12 | 16 | 16 | 12 | 9 | 23 | 21 |
| Manganese | ug/kg | 179 | 186 | 290 | 145 | 28 | 22 | 28 | 31 | 17 | 18 | 37 | 28 |
| Nickel | ug/kg | 12 | 9 | 8 | det | det | det | det | det | det | det | det | det |
| Zinc | ug/kg | 248 | 200 | 258 | 171 | 76 | 59 | 115 | 80 | det | 41 | 67 | 49 |
| TKN | mg/kg | 380 | 610 | 1600 | 440 | 2000 | 1300 | 2000 | 2200 | 350 | 590 | 500 | 480 |
| Total - P | mg/kg | 420 | 410 | 550 | 360 | 730 | 540 | 700 | 1700 | 4000 | 4300 | 1600 | 3700 |

out in the swales or deposited in the drop boxes. The higher copper value of 103 ug/kg measured in the pond was probably caused by algicide treatment. The nutrient concentrations reveal a different pattern than measured in the swales, even those swales planted with the native grass that is never mowed. The strand and one area in the pond (P12) both dry out between storms and are planted in grass that is kept mowed. The phosphorus measured in these areas is an order of magnitude greater than any of the other samples, yet TKN concentrations were some of the lowest measured. Either the soils in this area are anthropogenically enriched or dead grass clippings increase P concentrations in the sediments. Supposedly no fertilizer is applied to the native vegetation and grass used to landscape the site. When metals measured in the sediments were compared to chemical toxicity guidelines developed for marine environments by the Environmental Protection Agency (EPA) and the National Oceanic and Atmospheric Administration (NOAA), none of the samples exceeded the level where toxicity to organisms is probable. However, concentrations of copper and zinc were above the level where toxicity is possible. The level below which sediment is unlikely to be toxic is 34 mg/kg for copper and 150 mg/kg for zinc (Long *et al*. 1995).

Polycyclic Aromatic hydrocarbons (PAH) in the environment have increased with the widespread use of technology derived from organic chemicals and has led to widespread hydrocarbon pollution in stormwater runoff. Sediment samples at the site were tested for more than 100 organic pollutants, but only those listed in Table 5 were detected in the surface samples at the site. The PAH's detected were the same ones that were found in a study of sediment toxicity in Tampa Bay (Long *et al*. 1994). In that study the most toxic sites found were in the vicinity of the Florida Aquarium. A major source of PAH's is street dust present as weathered materials of street surfaces, automobile exhaust, lubricating oils, gasoline, diesel fuel, tire particles, and atmospherically deposited materials. The high concentrations found in both the Long study cited above and this study, indicate most of the pollutants may come from atmospheric deposition.

**Table 5. The Polycyclic Aromatic Hydrocarbons measured in the sediments. F1 through F8 represent basins in the parking lot and the other samples were collected in the strand and the pond. Abbreviations include: U=sediment was analyzed for but not detected, det=constituent was detected but was less than the minimum quantification limit.**

| POLYCYCLIC AROMATIC HYDROCARBONS | UNITS | asphalt no swale | | asphalt with swale | | cement with swale | | permeable with swale | | strand | | pond | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | F1 | F2 | F7 | F8 | F3 | F4 | F5 | F6 | S9 | S10 | P11 | P12 |
| **Benzo(a)anthracene** | ug/kg | det | U | **290** | det | det | det | U | U | U | det | det | det |
| **Benzo(a)pyrene** | ug/kg | det | det | **380** | det | det | det | U | U | U | det | det | det |
| **Benzo(b)fluoranthene** | ug/kg | **2100** | det | **940** | det | U | det | det | det | det | **2300** | **3300** | det |
| **Benzo(k)fluoranthene** | ug/kg | **730** | U | **290** | det | U | det | U | U | U | det | det | U |
| **Benzo(g,h,i)perylene** | ug/kg | U | U | det | det | U | U | U | U | U | det | U | U |
| **Chrysene** | ug/kg | **1300** | det | **470** | det | U | det | U | U | U | **1400** | det | det |
| **Di-n-octyl phthalate** | ug/kg | U | U | det | U | U | U | U | U | U | U | U | U |
| **Fluoranthene** | ug/kg | **1900** | det | **640** | **1700** | U | **1700** | det | U | U | **2600** | **2800** | det |
| **Indeno(1,2,3-cd)pyrene** | ug/kg | U | U | det | det | U | U | U | U | U | det | det | U |
| **Phenanthrene** | ug/kg | det | det | **310** | det | det | det | U | det | U | det | det | det |
| **Pyrene** | ug/kg | **1900** | det | **670** | det | det | **1300** | det | det | U | **2400** | **2100** | det |

When these results are compared to the toxic and non-toxic concentrations in amphipod tests conducted for marine sediments, most of the results are below the significantly toxic levels (Long *et al*. 1995). The exception is the concentration for Benzo(b)fluoranthene in the pond, where the concentration was 3300 mg/kg which is above the significantly toxic level of 2958 mg/kg. Other concentrations approached significantly toxic levels and these pollutants need careful study since they are harmful to man and beast.

At most sites pesticides and PBCs were undetected but there were some exceptions (Table 6.).

**Table 6. Pesticide detected at the site. Abbreviations: U=undetected and det=detected but below the minimum quantification limit, and greater than or equal to the minimum detection limit.**

| PESTICIDES | | asphalt no swale | | asphalt with swale | | cement with swale | | permeable with swale | | strand | | pond | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | F1 | F2 | F7 | F8 | F3 | F4 | F5 | F6 | S9 | S10 | P11 | P12 |
| **Diazanon** | ug/kg | U | det | U | U | U | U | U | U | U | U | U | U |
| **Chlordane** | ug/kg | det | det | det | det | U | det | det | det | det | U | U | det |
| **DDD-p,p'** | ug/kg | U | U | U | U | U | U | det | U | U | U | U | U |
| **DDE-p,p'** | ug/kg | det | U | U | det | **7.4** | U | **5.9** | det | **8.8** | U | **7.7** | **4.6** |
| **DDT-p,p'** | ug/kg | U | U | U | U | **6.0** | U | det | det | 5.7 | U | U | U |
| **PCB-1248** | ug/kg | U | U | U | det | U | U | U | U | U | U | U | U |
| **PCB-1260** | ug/kg | U | U | U | U | det | U | U | U | U | U | det | det |

Chlordane was the pesticide most often detected and it was found in all sites but three. DDT and its daughter products were measured at almost all locations and DDE was found in measurable quantities, but the quantities were in the non-toxic to amphipod survival range for marine environments (Long *et al*. 1995).

## ACKNOWLEDGEMENTS

Rebecca Hastings collected samples, kept samplers iced, serviced the electronic equipment, and carefully followed all the stipulations in our quality assurance plan. Ben Bahk and Craig Dye reviewed the manuscript. The project was funded in part by the Environmental Protection Agency (EPA) through the Florida Department of Environmental Protection (FDEP) with a 319(h) grant (WM662). Eric Livingston and David Worley at FDEP provided support and encouragement. The parking lot design was prepared by Art Woodworth, Jr., Engineer, Florida Technical Services and Thomas Levin, Environmental Planner, Ekistics Design Studio, Inc.

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# BAYOU CHICO/MAGGIE'S DITCH WATERSHED RESTORATION

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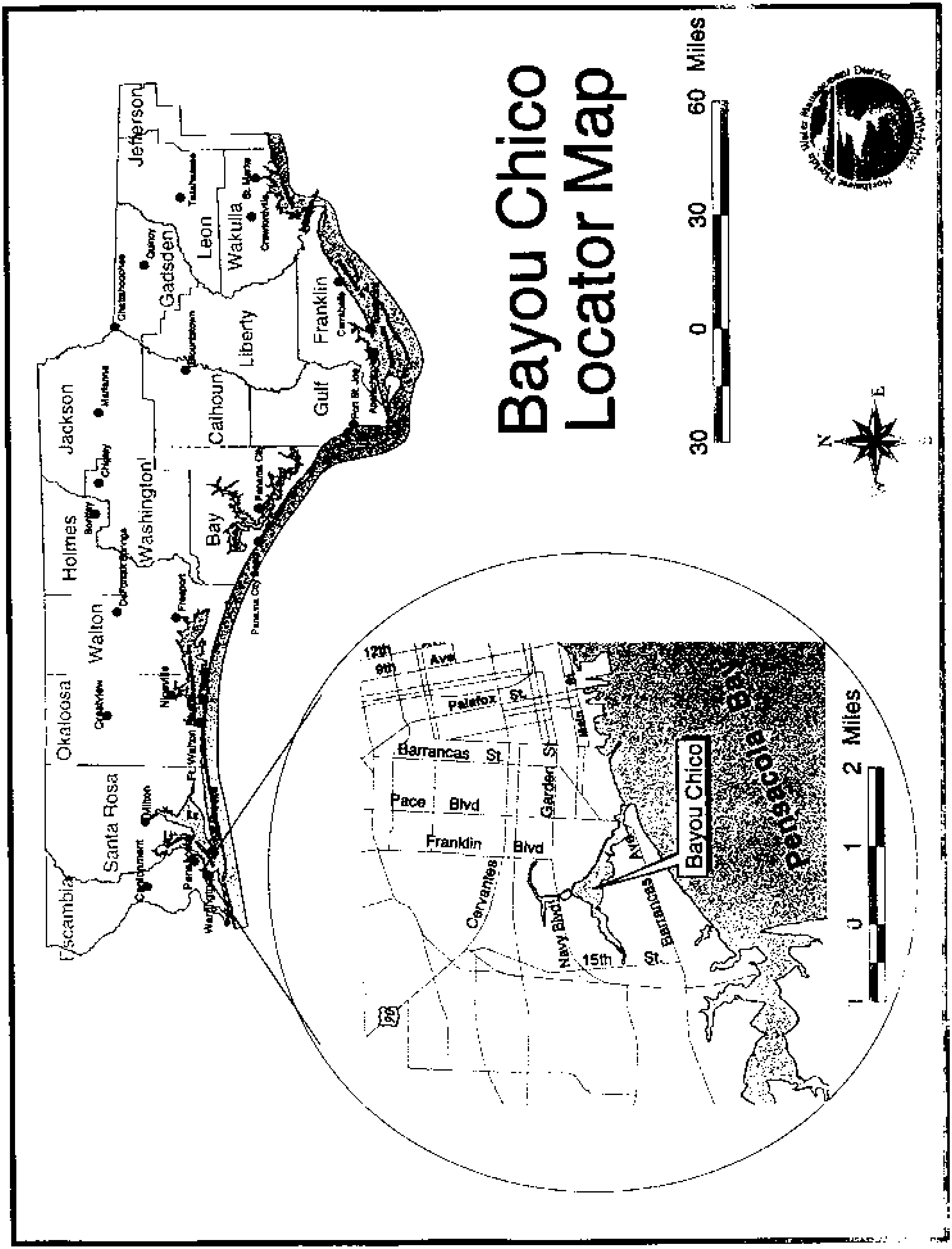
## ABSTRACT

The 94.8-hectare (240 acres) Maggie's Ditch drainage basin in Pensacola, Florida was fully developed long before the advent of stormwater quality rules. For many decades, runoff has been collected and directed into Maggie's Ditch, a channelized and highly altered one-time natural stream, a tributary to the east arm of Bayou Chico, arguably the most polluted water body in the state. As a result of a highly successful collaboration between private industry (Gulf Power), State government (NWFWMD, DEP), local government (Escambia County, Escambia County Utility Authority), and private citizens (Bayou Chico Association), the basin is being retrofitted with stormwater controls. Rather than the typical "square muddy pond" too often seen in the area, the District chose to design a combination wet detention pond/ marsh area, incorporating the best designs for treatment of stormwater, eye appeal, and natural appearance. This allowed the surrounding area to be converted into a City park, for the enjoyment of local residents. The design process was complicated by a city street bisecting the site, the presence of a leaky, antiquated and dilapidated vitrified clay sewer line, and by the presence of the Wildlife Sanctuary of Northwest Florida's buildings and compound, also located on the site. These factors, particularly the presence of the Sanctuary, dictated much of the design process. Many of the Sanctuary's needs, such as protected bird nesting islands and considerably more space for the animals which now call the Sanctuary their home, were met as a result of this project.

## INTRODUCTION

Bayou Chico is located in the northwest quadrant of Pensacola Bay in Escarnbia County, Florida. It is a "T"-shaped estuary, generally less than seven feet deep in most areas, with the exception of areas near the north shore, which have been dredged to provide a shipping channel. The dredged channels have average mid-depths of 8 feet with a maximum depth of 18 feet, according to sounding done by NWFWMD in 1989. Direct tidal exchange and flushing is restricted due to the fact that the mouth of the bayou is less than 164 feet wide. The watershed has an area of about 2,619 hectares (6,630 acres, 10.36 $mi^{-2}$) and lies mostly within Escambia County; however, aportion lies within the Pensacola city limits. The Bayou has had a long history of water quality problems, of which one of the earliest documentation was by de Sylva (1955). This report describes the progression of an extensive fish mortality event between May 20 and May 23. During the investigation, de Sylva reported on the condition of the bayou, "The water is covered with a film of scum. Sludge is present


Bayou Chico
Locator Map
30 0 30 60 Miles
Escambia
Santa Rosa
Okaloosa
Walton
Holmes
Washington
Jackson
Calhoun
Bay
Gulf
Liberty
Franklin
Gadsden
Leon
Wakulla
Jefferson
12th
9th
Ave
Palafox St
Barrancas St
Pace Blvd
Franklin Blvd
Garden St
Main St
Cervantes
Navy Blvd
15th St
Barrancas
Ave
Bayou Chico
Pensacola Bay
0 1 2 Miles
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E

on the bottom, from 2' to 10' thick. The entire area is characterized by an odor of hydrocarbons distinctive of tars and rosins. The entire shoreline of the Bayou is rimmed with layers of accumulated hydrocarbon sludge. **As** a consequence of these conditions, the bayou is almost completely devoid of life, particularly in the lower part." NWFWMD records indicate that prior **to** 1971, there were at least eight industrial and domestic waste sources discharging to Bayou Chico. Since that time, all of them have ceased direct discharges, which has had a remarkable improvement on water quality. Another researcher, Glassen et.al. (1977) reported that "these discharges inhibited biological activity to a great extent. According to local people, one could moor a boat in Bayou Chico and in a week or two the boat's bottom would be free of barnacles. Piles also seemed to last 'forever' as boring organisms were not a problem." There have also been significant nonpoint source inputs to the bayou from ship repair facilities, oil terminals, scrap metal junkyards, and residential areas.

A number of restoration studies and programs have been conducted in Bayou Chico, by the Florida State Board of Health, the US. **Army Corps** of Engineers, the Florida Department of Environmental Protection and others, as well as NWFWMD. Implementation of restoration alternatives for the bayou has been a priority of NWFWMD since the Pensacola Bay System Surface Water Improvement and Management (SWIM) Plan was approved in November 1988. For the purposes of data collection and hydrologic model application, the Bayou Chico watershed was delineated into three principal subwatersheds by Pratt, et.al. (1993), associated with the principal surface water drainage features that discharge into the bayou. These are Jones Creek, draining Jones Swamp on the west side of the waterbody; Jackson Branch on the northwest portion of the upper "T"-shaped portion; and, the subject of this paper, the northeast tributary, Maggie's Branch. This subwatershed is the smallest of the three, having **an** area of about 94.8 hectares (240 acres). Land use within the subwatershed is predominantly commercial and residential. Commercial land use lies along the major traffic corridors: Garden Street, Pace Boulevard, and Cervantes Street. The remainder of the subwatershed is single family residential. Maggie's Ditch was once a natural stream at the toe of an ancient escarpment, now channelized into an open east-west ditch with a perennial baseflow. The ditch flows west, crosses under the Burlington Northern Railroad and "W" Street before discharging into Bayou Chico. Extensive wetlands were once associated with Maggie's Ditch. Encroaching urbanization has eliminated most of these tracts, however, and illegal dumping heavily impacts the remainder.

## MATERIALS AND METHODS

The Maggie's Ditch subwatershed was evaluated by NWFWMD in 1993 as part of a watershed stormwater assessment, which greatly eased the design process. Between May 1989 **and** April 1991, NWFWMD collected stage elevation data and rainfall data on Maggie's Branch, downstream of a culvert under "S" Street, a site which would later prove to be in the center of the project. Additionally, the runoff from four storms was sampled between October 1990 and March 1991, with the results published in 1993. Thus, land use, percent impervious, rainfall **and** runoff characteristics, and other parameters thus were all known, **and** are summarized in Table 1. Water quality within the Maggie's Ditch subbasin was also a previous subject of study, and is presented in Table 2.

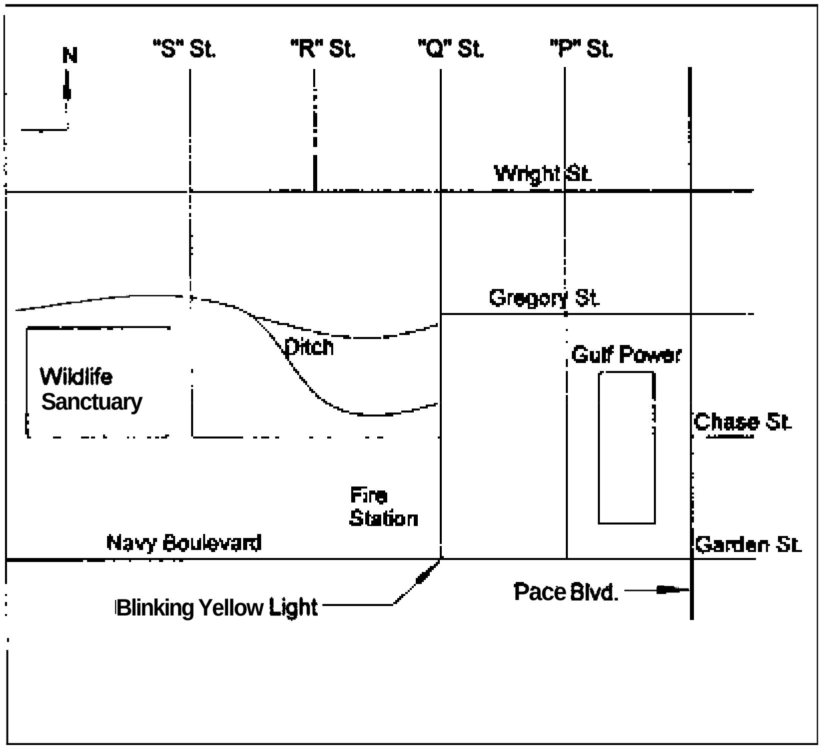
| | |
|---|---|
| Watershed Area | **94.8** Hectares (**240** acres) |
| Percent Impervious | **35%** |
| Average Slope | 0.007 |
| Mean Storm Duration | **8.48** hrs |
| Mean Interval Duration | **96.14** hrs |
| Mean Storm Runoff Volume | **9,916** $m^3$ (**6.94** acre-feet) |
| Mean Storm Peak Flow | 1.0 $m^3/s$ (**35.38** cfs) |
| Mean Base Flow | **0.04 $m^3/s$** (**1.55** cfs) |

Table **2:** Water Qualit Data

| Parameter | Mean Storm Loading Rate | Mean Base Flow Loading Rate | Total Loading | Areal Loading |
|---|---|---|---|---|
| TSS | **93.5** kg/hr (**206** lb/hr) | **kg/hr** (0.0 lb/hr) | **53,118** kg/yr (**117,000** lb/yr) | **87.5** kg/hectare/yr (**488** lb/acre/yr) |
| BOD, | **5.4** kg/hr (**11.9** lb/hr) | **0.194** kg/hr (**0.428** lb/hr) | **4,676** kg/yr (**10,300** lb/yr) | **7.7** kg/ hectare/yr (**42.8** lb/ acre/yr) |
| Total Phosphorous | **0.215** kg/hr (**0.473** lb/hr) | kg/hr (**0.0** lb/hr) | **122** kg/yr (**269** lb/yr) | **0.20** kg/ hectare/yr (**1.12** lb/ acre/yr) |
| Orthophosphate | 0.006 kg/hr (**0.014** lb/hr) | 0.0 kg/hr (0.0 lb/hr) | **3.6** kg/yr (**8** lb/yr) | **0.005** kg/ hectare/yr (0.03 lb/ acre/yr) |
| **TKN** | **1.185** kg/hr (**2.61** lb/hr) | **0.053** kg/hr (**0.116** lb/hr) | **1,108** kg/yr (**2,440** lb/yr) | **1.8** kg/ hectare/yr (**10.2** lb/ acre/yr) |
| **NO, +** NO, | **0.66** kg/hr (**1.46** lb/hr) | **0.269** kg/hr (**0.592** lb/hr) | **2,579** kg/yr (**5,680** lb/yr) | **4.3** kg/ hectare/yr (**23.7** lb/ acre/yr) |
| Total Lead | **0.08** kg/hr (**0.17** lb/hr) | **0.0005** kg/hr (**0.001** lb/hr) | **45.9** kg/yr (**101** lb/yr) | **0.08** kg/ hectare/yr (**0.42** lb/ acre/yr) |
| Total Zinc | **0.148** kg/hr (**0.325** lb/hr) | **0.004** kg/hr (0.008 lb/hr) | **112** kg/yr (**246** lb/yr) | **0.18** kg/ hectare/yr (**1.03** lb/ acre/yr) |

Because the drainage basin was completely built out long before the advent of stormwater quality requirements, the pollutant loading from Table 2 had been discharged untreated to the bayou literally for decades. One of the most difficult problems with retrofitting Completely built-out basins is the lack of available land to construct treatment systems. Typically, there is either no vacant land available or the available land is inconveniently located. This project proved to be **an** exception, which was pointed out by Pratt et.al. (1993), who noted that there were two vacant parcels located between "S" and "W" Streets. These parcels were owned by Gulf Power, who had no plans to develop *them.* In cooperation with the Escambia County Neighborhood and Environmental Services Department (NESD), NWFWMD began lobbying Gulf Power to donate this land for a regional stormwater management system. Simultaneously, NWFWMD and Escambia County's Public Works successfully applied for a grant from the Department of Environmental Protection's Florida Pollution Recovery Program to provide construction capital in the amount of $243,000. Escambia County provided $4000 as **an** in-kind match, and NWFWMD provided $40,000 from the Pensacola Bay **SWIM** funds. NESD also negotiated the purchase and trade of several private parcels to complete the site.

The project site finally obtained contained almost the entirety of the open ditch. The site contained 4.8 hectares (12.1 acres) of heavily impacted wetlands on the western portion of the site, which core soil samples indicated had long been used as an illegal dump, mostly for construction and demolition debris. This consisted of concrete rubble, lumber or other woody debris and asphalt shingle material, all of which would eventually need to be removed. On the east side of the tract, land outside the ditch was relatively dry, and a park was planned to take advantage of this. "S" Street, running north and south, bisected the site. Maggie's Ditch itself entered the site via two culverts, where Wright Street and Gregory Street cross "Q" Street. The ditch trends west until it crosses under "S" Street, via a 107 cm by 183 cm (42-inch by 72-inch) box culvert, and continues west until it exits the project at the western edge of the site. Downstream controls for the project are the two 100-centimeter (39-inch) pipes that allowed flow to cross under the Burlington Northern Railroad embankment. In addition to the storm flows the ditch received from portions of Pace Boulevard, Garden Street and Cervantes Street, the ditch received drainage along its open area from "Q" through "V" Streets through the northern edge of the site.




After exiting the site and crossing the railroad embankment, the ditch meanders through a relatively well preserved wetland until it crosses under "W" Street, where it discharges into a 0.13-hectare (0.32 acre) stormwater detention facility. This detention basin was built in 1990 with joint Department of Environmental Protection and Escambia County funding, and provides sedimentation prior to stormwater discharge to the bayou. Portions of "W" Street drain to this facility as well, and with the reduced pollutant load from Maggie's Ditch, the facility will be able to do a better job at pollutant removal from the "W" Street drainage.

Another complicating factor to the design of the facility was the presence of an antiquated 20-centimeter (8-inch) gravity sewer line on the site. This pipe also ran east-west through what was anticipated to become open water. The vitrified clay pipe was at least 20 years old, and was reported to leak and overflow periodically. The pipe had been in service so long, for example, that when the site was staked out, one of the known manholes could not be found even after land clearing. This presented a major obstacle to the project.

The final complication to the design process was the presence of the Wildlife Sanctuary of Northwest Florida. The Sanctuary is a not-for-profit organization dedicated to the preservation of injured animals, with the goal of eventually returning them to the wild. Their facility is a house and two acres leased from Gulf Power, on "S" Street. Their presence made an inroad on what would have been a uniform parcel. Part of the agreements with Escambia County and Gulf Power were that the Sanctuary would continue to occupy this facility. Although the Sanctuary's intent is to return injured animals to their habitat following recovery, inevitably they receive animals that recover, but are too seriously injured to ever be released. For these animals, the Sanctuary becomes their home for the rest of their natural lives. This leads to an ever-increasing population of pelicans, seagulls, deer, and other animals, including two bald eagles. The partners in this project were particularly concerned with the eagles safety during construction, because they do not well tolerate the noise produced by heavy machinery in action. One of the difficulties faced by the Sanctuary was that it was rapidly running out of room.

Improvement of water quality was a primary concern for this project. However, since this was essentially to be an "end-of-pipe" treatment system, water quantity issues also had to be addressed. With the relatively high impervious surface and the channelized nature of Maggie's Ditch, the basin historically had a rapid, large response to storm events. Flooding of the area was an occasional concern. The treatment concept chosen for this project was a wet detention facility, with both wet and dry marsh areas, This design provides a very high level of water quality treatment, with literature-provided pollutant removal rates of 85 percent for suspended solids, 60 percent for total phosphorus, and 45 percent for total nitrogen. An open pond system with associated marshlands for water storage was expected to attenuate and reduce the velocity of flood flows from the drainage basin. And finally, judicious selection of attractive, flowering wetland vegetation, necessary for nutrient uptake and to buffer flows, would make the facility an attractive parkland area for the community.

Because this project was to be a basin retrofit, with no new impervious surface to be added, it was assumed that the Florida Department of Environmental Protection requirement to treat 2.54 centimeters (one inch) of runoff did not apply. Due to space limitations, the maximum amount of runoff that could be successfully treated was 1.27 centimeters (0.5 inch), which necessitated detaining and storing 14,288 cubic meters (435,600 cubic feet) of water. Assuming a 0.3-meter (one-foot) depth, dictated by the invert elevation of the pipes running beneath the railroad berm, this translated to a 3.95-hectare (10.2-acre) pond system, allotting roughly 70% to open pool and 30% to marsh.

Prior to beginning the design process, NWFWMD met with the Wildlife Sanctuary's Director. With the number of animals present in the Sanctuary, particularly the high strung eagles, there was some concern of disturbing them with heavy equipment. Additionally, the District desired to extend every available opportunity to meet the Sanctuary's needs, and the meetings helped to determine them. Over several discussions, the Director provided the NWFWMD with a "wish list" of items they needed. Many were, of course, beyond the scope of this project, but a number were easily incorporated in the design or construction phase. These included:

1. Additional space for the animals. By dedicating the entire west portion of the pond, west of the dividing "S" Street, their area was increased from two acres to seven.
2. Additional fencing. The animals that make the Sanctuary their home are defenseless to dog attacks, and funding was allotted to surround the perimeter of the new area.
3. Additional open water for the aquatic birds, turtles and other animals. The Sanctuary was making do with an artificial circular pond, about 100 feet in diameter. The pond was fed by groundwater, with no surface inlet or outlets, and given the number of animals using it, was usually foul, particularly in the summer months. By extending the western portion of the pond southward, this pond was incorporated into the proposed larger pond. The base flow of Maggie's Ditch provides circulation and flushing, and the increased volume of water and wetland vegetation allows treatment of the animal wasteload.
4. Nesting islands, Although injured, most of the aquatic fowl retain their drive to reproduce. According to the Sanctuary, many of them require isolated, undisturbed areas in which to do so. NWFWMD also desired a serpentine flow path through the treatment facility to maximize retention times, whereas the available land and slopes dictated a path straight across the pond, following the existing east-west ditch line. The solution to both problems was islands within the west pond area, to interrupt and direct the flow, while allowing isolation for the birds.

5. Rehabilitation habitats for gopher tortoises, beavers, otters, small seabirds, and songbirds. The increased land area, coupled with areas of open water, low marsh, and high marsh, provide a wide range of rehabilitation options.

Special care was also taken during the land clearing and construction phases to protect and shelter the animals. The eastern portion of the pond was constructed first. The animals were then gathered into temporary shelters across "S" Street, while construction was done to tie the existing pond into the new open water area. The equipment operators doing the actual construction were very aware of their responsibility to disturb the animals as little as possible, an attitude which was reinforced as they took their lunch breaks at the Sanctuary. Perhaps prophetically, the eagles in the Sanctuary hatched a chick very shortly prior to beginning construction. This event attracted media attention and served to focus attention on the Sanctuary, and coincidentally, on this project.

## RESULTS

As mentioned, Maggie's Ditch had been extensively channelized, and in spots, lined with broken concrete rubble. The rubble was removed, and on the eastern portion of the pond the channel was widened to allow sedimentation, and a meandering shoreline was reestablished. Terraces were sculpted into the banks for stormwater storage and for the marsh areas. This procedure contrasts sharply with what happened on the western portion. There, the channel was left intact, but the grade was reversed. This section of the ditch received storm flows from "T", "U" and "V" Streets at one-block intervals. Had the pond been expanded from the ditch, these flows would have had a tendency to short-circuit the pond, passing through with little or no treatment. By leaving the existing channel intact, it serves as an interceptor for these flows. Reversing the grade directs them to the head of the western portion of the pond for treatment. The high and low marsh terraces are along the edges of the pond, and clustered thickly at the outfall to provide filtering and polishing actions. Water levels in the system are controlled by a slotted inlet, with an orifice sized to reduce half the treatment volume in greater than 60 hours, but discharge the entire treatment volume within 120 hours (FDEP requirements for wet detention). The pond meets all wet detention requirements, and can store and treat the flows resulting from a 25-year storm. While this has not yet been field tested, computer modeling (XP-SWIM®) indicates that the system will pass a 100-year flood event with a reduction in areal land flooding.

During the construction phase, the Escambia County Utility Authority (ECUA) re-routed the dilapidated sewer line. They had readily agreed to do so in support of the project when they learned of it. The relocation proved to be more difficult and expensive than anticipated, since the sewer line they had planned to tie the re-routed pipe into proved to be already at its maximum flow rates. Extra line had to be laid to extend to another suitable trunk line, and a lift station designed and built to move the waste to its new line. ECUA is to be commended for their responsiveness and willingness to absorb the extra costs.

## CONCLUSIONS

Retrofitting an old urban drainage basin for stormwater controls can be a massive undertaking, usually beyond the means of any one municipality or organization. The project presented here utilized an unusual partnering between private industry, local and state governments, and private citizens to achieve together what none of them could do alone. The basin has been fitted with state-of-the-art stormwater controls that are becoming a showplace for NWFWMD. Escambia County has acquired a new county park and greenspace area. The Wildlife Sanctuary has benefitted by the new areas to rehabilitate and house injured animals. Everybody associated with this project has come out of it a winner.

## ACKNOWLEDGMENTS

The author would like to thank Lori Cunniff of the Escambia County Neighborhood & Environmental Services Department for her work in coordinating land acquisition and administering grant requirements. Linda Bauer of the Escambia County Utility Authority was particularly helpful in relocating the sewer line, and Jim Hasty of the Escambia County Public Works provided on-site project construction management. Gulf Power's generous donation of property was essential to this project, as was the Pollution Recovery Trust Fund Grant provided by the Florida Department of Environmental Protection. Dorothy Kaufmann of the Wildlife Sanctuary was essential in providing guidance for protection of the animals. The Bayou Chico Association, and particularly its past president LaVerne Matheson, provided invaluable support and historical references. And last, but

certainly not least, Ron Bartel of NWFWMD was instrumental in interagency coordination and project overviews, and Judy Duval performed the calculations and permitting.

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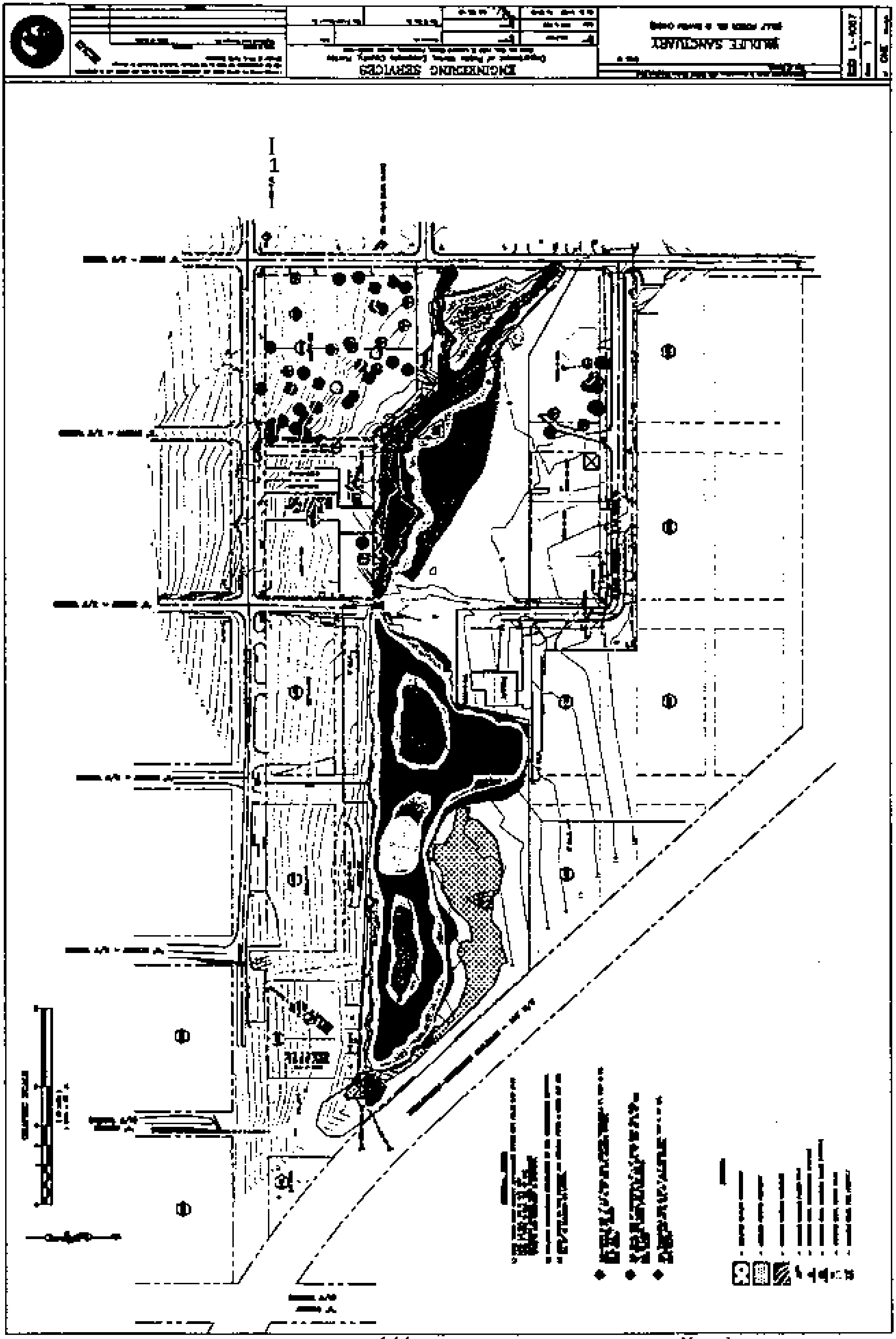
ENGINEERING SERVICES
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# DEVELOPING NONPOINT SOURCE WATER QUALITY LEVELS OF SERVICE FOR HILLSBOROUGH COUNTY, FLORIDA

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## ABSTRACT

**A** pollutant loading and removal model was developed by Hillsborough County Public Works/Stormwater Management Environmental Team to facilitate water quality assessments in Hillsborough County. The model was prepared in response to the county's desire to establish water quality levels-of-service. Water quality levels-of-service may in turn be used to guide development and management actions.

The pollutant loading and removal model has three main components: calculation of gross pollutant loads, estimation of net loads considering the effects of best management practices (BMPs), and evaluation of water quality levels-of-service. The model uses GIS coverages of soils and land use *to* calculate pollutant loads. Pollutant removals are estimated by evaluating BMP coverage and removal efficiencies. **A** list of removal efficiencies for standard BMPs is supplied with the model, and user-specified BMPs are allowed. Finally, water quality levels of service are calculated and the GIS subbasin coverage is updated to include an attribute for water quality level-of-service.

## INTRODUCTION

Nestled on the eastern shores of Tampa Bay, Hillsborough County covers about 900 square miles of land and 25 square miles of inland water area. Organized in **1934** as Florida's 19th County, Hillsborough has enjoyed growth and a population increase synonymous with that experienced by the rest of Florida. Despite the rapid growth rate, about 60% of the County remains either undeveloped or in agricultural use. The remaining 40% consists of urbanized land, With this increased growth and development, there have been many challenges for the County to provide and maintain the services required to meet the needs of its expanding citizenry.

During the El Niño rains of 1997 - 1998, large areas in Hillsborough County, especially in the Northwest, experienced street and structural flooding for an extended period of time. With over 50 inches of rain falling in a 4-month period (following a wet rainy season), El Niño proved to be the ultimate test for the effectiveness of the County's stormwater infrastructure.

Flood investigations during El Niño have generated more than three hundred (300) stormwater neighborhood projects. The need to accelerate the stormwater master planning of the entire county was also recognized. Such master plans would be managed as a group standardizing methodologies countywide. They would also allow for solutions to be implemented for the regional problems, while more effectively solving the localized problems.

Through discussions with staff in various County departments and with personnel from state agencies, it was quickly recognized that these plans could, and should, do more than solve flood problems. With $96 million approved by the Board of County Commissioners for an accelerated 5-year stormwater program, the objectives of these new plans came into fruition. One of the objectives is to establish a Water Quality Level of Service in the basins throughout the county. The Pollutant Loading and Removal Model was developed to serve this purpose.

## Model Input Data

The Pollutant Loading and Removal Model has three main components: calculation of gross pollutant loads, estimation of net loads based on existing treatment, and evaluation of water quality levels-of-service. GIS coverages of land use and soils are used together with drainage basin delineations to determine runoff characteristics. Gross pollutant loads for each subbasin are calculated as the product of the runoff volume and the stormwater event mean concentrations (EMC) for each chemical of interest. The EMCs are based on measurements taken during stormwater characterization studies performed by Hillsborough County, and later submitted as part of the County's National Pollutant Discharge Elimination System (NPDES) permit. Net pollutant loads are estimated by evaluating removal from existing stormwater treatment within each sub-basin. A water quality level-of-service is then determined based on a comparison of existing net loads to a benchmark condition represented by the pollutant load produced by a typical land use in the area, in this case low/medium density residential.

## Land Use

Land use composition within a sub-basin determines the extent of impervious areas, which in turn determines the volume of runoff expected from these basins and subbasins within the watershed. The 1995 land use coverages prepared by the SWFWMD were used herein to evaluate land use in each watershed. These coverages are based on the Florida Land Use and Cover Classification System (FLUCCS).

For pollutant loading estimates, land use categories were aggregated to correspond with the Hillsborough County NPDES permit. Major non-natural land use categories evaluated for pollutant loading included:

- low/medium density residential
- high density residential
- light industrial
- agricultural
- commercial
- institutional
- highway/utility
- recreational
- open land, and
- extractive (mining)/disturbed.

## Soil Characteristics

For hydrologic analyses, a standard method of soils classification is the hydrologic soils group. Soils are grouped into four hydrologic soil groups **A** through D, which are commonly used to estimate infiltration rates and soil moisture capacities. Runoff volume calculations are based on the application of runoff coefficients by soil and land use type. The values assigned to the runoff coefficients were based on those obtained from NPDES permit studies conducted in Hillsborough County. Most of the coefficients, listed by land use, can be found in the FDOT drainage manual.

## Basin Delineations

For purposes of comparing hydrologic, hydraulic, and runoff water quality characteristics of the different areas, each watershed is divided into sub-basins. Sub-basins may be aggregated to an intermediate, or basin, level to evaluate particular areas or reaches of interest within a watershed.

## Pollutant Concentrations

The chemicals of interest for pollution load analysis were those required for NPDES permitting for stormwater discharges, as listed in Table 1. The annual amount of constituent mass that is washed-off from each basin during rainfall events was calculated as the product of the annual runoff volume times the corresponding event mean concentration (EMC). The EMC is the mean concentration of a chemical parameter expected in the stormwater runoff discharged from a particular land use category during a typical storm event. The calculated constituent mass represents the pollution load.

For watershed analyses in Hillsborough County, the EMC values reported in the County's NPDES permit applications for stormwater discharges and supporting documents were used if available. For land use categories or parameters not reported by Hillsborough County, EMC data from other studies in Florida were evaluated and used if appropriate. EMC values were available for many land uses for numerous pollutants including five-day biological oxygen demand ($BOD_5$), total suspended solids **(TSS),** total kjeldahl nitrogen (TKN), nitrite plus nitrate ($NO_2$+$NO_3$), total nitrogen (TN), total and dissolved phosphorous (TP **and** TDP), oil **and** grease (O&G), cadmium (Cd), copper (Cu), lead (Pb), and zinc (Zn). EMC values used to estimate pollutant loads are summarized in Table 1. **A** comparison of these values to other Florida and national studies is provided in the following paragraphs.

BOD, data found in Hillsborough County samples tend to be lower, or similar, than those found in other areas in Florida, except for agriculture. The agriculture EMC for BOD, is

Table 1 - Event Mean Concentration (EMC) Values by Land Use

| Land Use | $BOD_5$ | TSS | TKN | $NO_2+NO_3$ | TN | TP | TDP | Oil and Grease | Cd | Cu | Pb | Zn |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Low/Medium Density Residential | 1 e | 19 | 1.082 | 0.281 | 1.363 g | 0.401 | 0.282 | 1.08 | 0.001 e | 0,013 | 0.008 | 0.022 |
| High Density Residential | 2.6 | 29 | 1.368 | 0.679 | 2.047 g | 1.337 | 0.552 | 1.073 | 0.001 e | 0.047 | 0.006 | 0.058 |
| Light Industrial | 2.87 | 18.2 | 2.088 | 0.187 | 2.275 g | 0.332 | 0.187 | 3.663 | 0.001 e | 0.024 | 0.0055 | 0.096 |
| Agricultural | 18.3 | 12.7 | 2.167 | 0.803 | 2.97 g | 2.349 | 1.223 | 0.5 e | 0.013 | 0.041 | 0.0025 e | 0.017 |
| Commercial Office | 2.62 | **36.5** | 2.207 | 0.171 | 2.378 g | 0.305 | 0.182 | 0.793 | 0.001 e | 0.014 | 0.0025 e | 0.036 |
| Commercial Retail | 2.717 | 9.33 | 1.083 | 0.603 | 1.686 g | 0.253 | 0.132 | 0.5 e | 0.001 e | 0.021 | 0.005 | 0.015 |
| Commercial, combined | 2.6685 | 22.915 | 1.645 | 0.387 | 2.032 g | 0.279 | 0.157 | 0.6465 | 0.001 | 0.0175 | 0,00375 | 0.0255 |
| Institutional | 2.6685 f | 22.915 f | 1.645 f | 0.387 f | 2.032 g | 0.279 f | 0.157 f | 0.6465 f | 0,001 f | 0.0175 f | 0.00375 f | 0.0255 f |
| Highway/Utility | 24 a | 261 a | 2.99 a | 1.14 a | 4.13 g | 0.12 a | 0.3 d | 0.4 d | 0.04 a | 0.103 a | 0.96 a | 0.41 a |
| Recreational | 3.8 b | 11.1 b | 2.09 b | 0.508 b | 2.598 g | 0.05 b | 0.134 c | 0.9 d | 0.007 b | 0.041 b | 0.0056 b | 0.004 b |
| Open Land | 3.8 f | 11.1 f | 2.09 f | 0.03 c | 2.598 g | 0.194 | 0.134 f | 0.9 f | 0.0003 c | 0.001 c | 0.001 c | 0.006 |
| Extractive (Mining)/Disturbed | 28.94 c | 13.2 c | 3.5 c | 0.03 c | 3.53 g | 0.194 c | 0.134 c | 0.9 d | 0.0003 c | 0.001 c | 0.001 c | 0.006 c |
| Upland Forest | 0 h | 0 h | 0 h | 0 h | 0 h | 0 h | 0 h | 0h | 0h | 0h | 0h | 0h |
| Wetland Forest | 0 h | 0 h | 0 h | 0 h | 0 h | 0 h | 0 h | 0h | 0h | 0h | 0h | 0 h |
| Wetland Non-Forested | 0 h | 0 h | 0 h | 0 h | 0 h | 0 h | 0 h | 0h | 0h | 0h | 0h | 0h |

Note:

NPDES parameters: BOD5, COD, TSS, TDS, TKN, NO3+NO2, TP, DP, O&G; cadmium, copper, lead, zinc.
All EMC values without footnotes were obtained from samples collected for the Hills. Co. NPDES Permit Application (1993).
For parameters not detected in all samples, EMCs were calculated using in-half the reporting limit for nondetects.
"BDL" - indicates below detection limits for all Hills. Co. samples collected for a particular land use.
For pollutants not reported by Hills. Co. (1993), additional sources were used as noted:

a. Average values used by Hillsborough Co. (1994) (from Smith and Lord (1990), provided in Wanielista and Yousef (1993).
b. Literature value reported as EMC in Hillsborough Co. 1994.
c. Calculated value from Sarasota County stormwater samples.
d. Orange County, 1993.
e. Surrogate based on 1/2 DL for values reported as BDL.
f. EMCs for open land use were assumed to be less than or equal EMCs for recreational land use.
g. Total nitrogen (TN) estimated as the sum of $NH_3$ + organic-N (TKN) and oxidized-N ($NO_2$+$NO_3$).
h. EMCs for upland forest, wetland forest, and non-forested wetland were assumed to zero for benchmark comparisons.

EMCs reported as representative of agricultural land use were used for all subcategories of agricultural land use (e.g., pastures, crops, and groves).

approximately five times larger than other values reported in Florida. In general, Hillsborough County agricultural land use EMCs for a number of parameters, tend to be much higher than those reported elsewhere in Florida. For most parameters, these elevated EMCs increase estimated load calculations significantly where agricultural land use is found.

Nitrogen from residential land uses tends to be higher in Florida and Hillsborough County than nationally due to the increased application of lawn fertilizer by homeowners **and** golf course managers, Slightly higher TKN and TP values for multi-family sites may reflect more intensive landscape maintenance for these land uses. Commercial land uses also have nitrogen values that are higher than national averages. This may reflect primarily atmospheric deposition, as studies in Florida have shown that commercial sites produce elevated nitrogen loads even if little green area is present. Phosphorous runoff tends to be lower in Florida than the U.S. average, although data from Hillsborough County studies differs somewhat. Phosphorous runoff from residential and commercial land uses are higher than Florida average, while runoff from industrial land uses **are** similar to Florida and national averages. **As** with nitrogen, elevated loads from multi-family land uses could reflect more intensive landscape maintenance.

The Hillsborough County data indicate that total nitrogen and total phosphorus EMCs for the agricultural land use are **74** and 586 percent higher, respectively, than that for low/medium family residential uses. The total nitrogen EMC is similar to that found for other locations in Florida. However, the EMC for total phosphorus is six times as high as the average EMC found for various agricultural sites in Florida. This situation makes agriculture one of the main contributors of nutrient loadings.

TSS data for Hillsborough County are comparable to other Florida locations and lower than U.S. averages. **TSS** results from soil erosion, with construction sites a major contributor and agricultural practices. Additional primary sources of **TSS** include vehicle emissions and atmospheric deposition. BOD data for Hillsborough County is somewhat low relative to other locations in Florida and across the U.S. Low levels of organic matter may reflect the low organic matter typically present in Florida soils.

Lead data for Hillsborough County are lower than other locations in Florida and across the U.S. Relatively low lead concentrations may reflect concentrations may reflect fate and transport characteristics of the particular systems sampled and/or decreased emissions due to the use of unleaded gasoline. Copper data for Hillsborough County are higher than other locations in Florida, but similar to the nationwide average, Relatively high values were observed for residential land uses. Transportation-related activities, particularly releases from brake linings, have been identified as primary sources for copper. Copper is also a common element in algaecides and fungicides, and many fertilizers contain copper. Zinc data are much lower far Hillsborough County and Florida in general than the rest of the **U.S.** Sources of zinc include industrial processes, transportation-related activities, atmospheric deposition and fertilizers. Relatively low zinc concentrations may reflect fate and transport characteristics of the particular systems sampled and/or the presence of fewer industrial-processing facilities in Hillsborough County than other parts of the U.S.

## Existing Stormwater Treatment

The type and coverage of best management practices (BMPs) providing pollutant removal needs to be determined to estimate net loads from each basin, BMP type and coverage data is developed for each aggregate land use for each sub-basin based on interpretation of aerial photos and entered into the model input data set.

**Loading and Level-of-service Calculations**

The model uses the EPA Simple Method (U.S. EPA 1992) to calculate pollutant loads. The runoff characteristics discussed above were used with EMC values for particular land uses to calculate gross pollutant loads. All EMCs, runoff coefficients, and other lookup values required were incorporated into lookup tables provided with the Hillsborough County model. In the model, the average annual runoff expected from each specific land use/soils polygon, as determined from the GIS, is calculated as the product of the rainfall amount times the corresponding runoff coefficient. **A** correction factor of 0.9 is used to account for the numerous small rainfall events (possibly less than 0.1 inches) that do not result in any runoff. The total volume of runoff for a sub-basin is then determined by aggregating the calculated runoff for each polygon within that basin. Each sub-basin's contribution in terms of stormwater runoff discharges were calculated as average annual runoff flow. The average annual rainfall in the Tampa Bay area, which amounts to 51.4 inches, was used to calculate expected annual pollution loads.

According to the Simple Method, non-point source pollutant loads are calculated using the following formula:

$$L_I = (0.227)(P)(CF)(Rv_I)(C_I)(A_I)$$

where

$L_I$ = annual pollutant load per basin (lb/yr)
$P$ = annual average precipitation (in/yr)
$Rv_I$ = weighted average runoff coefficient based on impervious area
$C_I$ = event mean concentration of pollutant (mg/L)
$A_I$ = catchment area contributing to outfall (acres)
CF = correction factor for storms that do not produce runoff
(assumed CF=0.9, 10 percent of storms do not produce runoff)

**Levels-of-Service**

In order to effectively manage stormwater pollution in Hillsborough County, water quality levels-of-service criteria were established as part of this study to allow comparison of existing or proposed conditions to pollutant loading goals. For comparison purposes, pollutant loads based on runoff from single family (low to medium density) residential land use were selected as the standard for comparison. In this manner, the calculation of pollutant loads is consistent with the concept of standard residential unit (SRU) used for stormwater utility assessments.

The procedure applied to each sub-basin consisted of the following steps:

1. Calculate the net pollutant load for each chemical of interest based on actual land use composition and treatment levels
2. Calculate the gross pollutant load for that same chemical, assuming that 100 percent of area is developed for low/medium residential land uses
3. Calculate the ratio net load/gross load
4. Apply the criteria described below to determine the LOS for the sub-basin
   LOS criteria A through F were defined based on the following ranges:

**LOS A, net load equivalent to 20% or less of untreated single family residential.** A **LOS** equal to **A** for a sub-basin would indicate the presence of undisturbed natural systems, or areas supplied with treatment systems capable of removing pollution levels to those representing natural systems. Areas where typical land uses (residential) exhibit stormwater treatment levels above the minimum required per 62-40.432(5) F.A.C. (Water Policy) would also receive LOS **A.**

**LOS B, net load equivalent to between 20 and 40% of untreated single family residential areas. A** LOS equal to B would indicate the presence of treatment systems showing removal efficiencies consistent with those representing adequately designed and maintained conditions.

**LOS C, net load equivalent to between 40 and 70% of untreated single family residential areas.** A **LOS** equal to C would indicate the presence of treatment systems showing removal efficiencies consistent with those representing average to poorly maintained conditions.

**LOS D, net load equivalent to between 70 and 100% of untreated single family residential areas. A LOS** equal to D would indicate minimal treatment of sub-basin discharges.

**LOS F, net load equal to or greater than 100% of untreated single family residential areas. A LOS** equal to F would indicate no treatment for sub-basin discharges, or the presence of extensive areas of land uses producing larger pollution loads per unit area than typical residential land uses.

## SAMPLE RESULTS

Water quality levels-of-services have been developed for several Hillsborough County watersheds. Summary results for one of these watersheds, Double Branch Creek Watershed, is presented herein. The Double Branch Creek Watershed comprises a total of approximately 13,588 acres. Developed land use is primarily agricultural (3,266 acres or 24%), low density residential (120 acres or 9%), and high density residential (780 acres or 6%). Approximately 40 percent of the watershed consists of natural communities including upland forest (1**882** acres or 14%), wetlands **(3,123** acres or 23%) and open water **(791** acres or ***6%).*** The remainder includes open land (1,256 acres or 9%), recreational **(657** or 5%), and highway/utility (405 acres or 3%) land uses, with other developed land uses (extractive, light industrial, and institutional) contributing less than one percent.

**A** *summary* of pollutant loads from sub-basins within the Double Branch Creek Watershed **and** LOS based on a comparison of these loads to the residential benchmark are provided in Tables 2 and 3, respectively.

**Table 2**
**Summary of Net Pollutant Loads - Double Branch Creek**

| | Area (acres) | Volume (acre-feet) | $BOD_5$ (lbs/yr) | TSS (lbs/yr) | TKN (lbs/yr) | $NO_3+NO_2$ (lbs/yr) | TN (lbs/yr) | TP (lbs/yr) | TDP (lbs/yr) | O&G (lbs/yr) | Cd (lbs/yr) | Cu (lbs/yr) | Pb (lbs/yr) | Zn (lbs/yr) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Total | 13588 | 13940 | 259665 | 1087866 | 47750 | 16684 | 64435 | 29772 | 16156 | 21020 | 243.29 | 993.03 | 2660.2 | 1701.8 |
| Number | 161 | 161 | 161 | 161 | 161 | 161 | 161 | 161 | 161 | 161 | 161 | 161 | 161 | 161 |
| Min | 0.02 | 0.01 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.0000 | 0.0000 | 0.0000 | 0.0000 |
| Max | 464.47 | 314.79 | 10073 | 52583 | 1260 | 448.60 | 1665 | 1260 | 660.76 | 2026 | 10.50 | 28.69 | 183.66 | 81.95 |
| Mean | 84.40 | 86.59 | 1613 | 6756.9 | 296.6 | 103.63 | 400.22 | 184.92 | 100.35 | 130.56 | 1.51 | 6.17 | 16.52 | 10.57 |

**Table 3**
**Summary of Sub-Basin Levels-of-Service - Double Branch Creek**

| LOS (count) | BOD, | TSS | TKN | NO3 +NO2 | TN | TP | TDP | Oil and Grease | Cd | Cu | Pb | Zn |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| A | 3 | 21 | 7 | 7 | 7 | 38 | 23 | 28 | 4 | 12 | 43 | 31 |
| B | 3 | 38 | 11 | 10 | 10 | 7 | 21 | 70 | 11 | 7 | 31 | 21 |
| C | 4 | 33 | 29 | 19 | 26 | 13 | 16 | 36 | 4 | 13 | 15 | 37 |
| D | 10 | 10 | 46 | 37 | 46 | 12 | 16 | 10 | 6 | 17 | 10 | 6 |
| F | 141 | 59 | 68 | 88 | 72 | 91 | 85 | 17 | 136 | 112 | 62 | 66 |
| No. Sub-Basins | 161 | 161 | 161 | 161 | 161 | 161 | 161 | 161 | 161 | 161 | 161 | 161 |
| Overall Watershed | F | F | D | F | D | F | F | B | F | F | F | F |

| LOS (percent) | BOD, | TSS | TKN | NO3 +NO2 | TN | TP | TDP | Oil and Grease | Cd | Cu | Pb | Zn |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| A | 2% | 13% | 4% | 4% | 4% | 24% | 14% | 17% | 2% | 7% | 27% | 19% |
| B | 2% | 24% | 7% | 6% | 6% | 4% | 13% | 43% | 7% | 4% | 19% | 13% |
| C | 2% | 20% | 18% | 12% | 16% | 8% | 10% | 22% | 2% | 8% | 9% | 23% |
| D | 6% | 6% | 29% | 23% | 29% | 7% | 10% | 6% | 4% | 11% | 6% | 4% |
| F | 88% | 37% | 42% | 55% | 45% | 57% | 53% | 11% | 84% | 70% | 39% | 41% |

Overall LOS for many parameters were "D" or "F" indicating discharge over untreated residential land use and/or minimal treatment within much of the watershed. These results also suggest that much of the pollutant load may be assimilated by natural wetlands and waterbodies. Several important parameters, however, including TSS, oil and grease, lead, **and** zinc, attained LOS "C" or higher in the majority of basins, indicating low discharge and/or adequate treatment for these parameters. **TSS,** oil and grease, lead, and zinc, attained LOS "C" or above in more than **80%** of basins. **As** shown in Table **3,** LOS were "D" or "F" in greater than 80-90% of sub-basins for three parameters $BOD_5$, copper, and cadmium. Low overall LOS values in Double Branch Creek may result from the lack of adequate treatment in areas with agricultural runoff.

## CONCLUSIONS

The pollutant loading and removal model is a valuable aid to assist in water quality assessments in Hillsborough County. The model provides the county with the ability to establish water quality levels-of-service. Water quality levels-of-service may in turn be used to guide development and management actions. Reducing nonpoint source quality data to a more simplified grading scheme to represent water quality status will facilitate policy and regulatory decision-making.

# DEVELOPMENT OF A LOCAL WATER QUALITY INDEX FOR FRESHWATER LAKES

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## ABSTRACT

To assess water quality in freshwater lakes various methodologies have been developed with different degrees of success. In Florida, the Trophic State Index (TSI) is one of the most popular methods presently used and based on a classification scheme developed in **1974.** However, it fails to meet some of the characteristics necessary for a qualitative index. Characteristics cited for development of a water quality index embody the following: 1) easy to calculate, 2) simple, 3) narrow in scope, 4) absolute rather than relative, and *5)* scientifically valid and consist of linear relationships. The water quality index (WQI) proposed here, has been developed with these fundamental characteristics in mind. It is a linear equation using six ***(6)*** water quality parameters that are numerically weighted based on current literature and authority. In the design of this index, the formula or equation compares the ambient water quality data to known standards and regulatory classifications. Therefore, the index is absolute and not as dependent on "experts" interpretation. The index developed here is also useful in comparing the overall trophic conditions between lakes. Lakes in the study area are all Classification 111, freshwater lakes. It is simple enough to be useful to the nontechnical public, as well as, the scientific managers. Comparison of the index to the FTSI indicates the two are comparable in assessing long term water quality variations, but the WQI is more sensitive to changes in the variables.
(KEY TERMS: trophic state, TSI, WQI, Classification 111, ambient, linear equation, absolute, temporal)

## INTRODUCTION

Lake classification is becoming increasingly important in management and planning activities. Limnologists have been interested in classifying lakes since the early days of science, and nearly all types of characteristics have been used to define lake classes. With the great diversity of lakes in terms of geology, morphology, hydrology, and chemistry, this prevents development of any simple classification scheme. With complicated properties including nutritional status and productivity, i.e., trophic state, a simple universal model is unrealistic, The term trophic state is also broad and loosely defined. Trophic status is a qualitative assignment of a lake into a class. The classification is defined by the productivity into three broad categories: oligotrophic, mesotrophic, and eutrophic. One of the first Trophic State Index models (TSI) used in Florida to classify lakes was developed by R.E.

Carlson (Carlson, **1974)** who used three parameters (analytes), Secchi depth, chlorophyll *a*, and total phosphorus concentrations. Various other models have been developed since then based on the rational of Carlson and others. The current Florida Trophic State Index (FTSI) used by the Florida Department of Environmental Protection (FDEP) and other agencies are also based on the Carlson scheme but eliminates the Secchi depth **and** adds total nitrogen concentration as a third analyte (FDEP, 1996). The TSI and FTSI using the three parameters effectively measures algal biomass production which is then correlated to lake productivity, or trophic status. This classification is based on the productivity as lakes evolve or age. Productivity increases from the early oligotrophic stage through the mesotrophic **and** later eutrophic. The **TSI** classification of lakes, alibiet quantified by individual analytes, is ultimately a relative interpretation when definedby as a trophic state. No single parameter defines trophic state; the concept is multidimensional. Even simple qualitative assignment is difficult because even among limnologists there are disagreements to the exact definition and meaning of the trophic states and therefore the exact lake condition. This paper attempts to develop a simple water quality index based on analytes that has either a regulatory definition or water quality based numerical limits.

## DISCUSSION

Many different approaches have been use to characterize water quality in lakes and streams. Some models require complex calculating. Still, others are based on qualitative and/or interpretations of trophic status. These attempts are meaningful only if it is implied that data representing a multiplicity of factors has been reduced to a single continuum. This process also results in debates between the experts. The approach should be linear and the index should have several characteristics as outlined by Brezonik and Shannon (Brezonik and Shannon, 1971).

1. Validity. The index should measure what it is supposed to measure. Validation is critical and not an easy process.

2. Accuracy. The index should measure the water quality or change in quality with accuracy.

3. Occam's Razor, The index should not contain more variables than is necessary, (The simplest model is usually the best model)

4. Applicability, The index should have wide applications.

5. Construction and cost. The index should involve variables in a meaningful way and should only use variables that are easily and cheaply measured.

6. Simple statistics. The index should not involve the means of the samples on which the index is based.

**7.** Robustness. The index should not vary greatly with small changes in the variables and should not reflex seasonal variation unless it is coupled with a change in water quality or trophic state.

The purpose of an index is two fold. First, it can be used to monitor a lake over time to determine if there is a change in water quality. And second, to characterize new lakes or water bodies. It is not critical that one index satisfy both functions. It however, is important that all water quality indices are specific to local conditions.

A water quality index developed in 1993 for south florida surface waters (King, 1993) was used as the working model for this current attempt to characterize water quality for lakes in Polk County, Florida. One of the first considerations is the parameter or analyte that is to be included in an index. Most often, the list includes current data, and data that have been archived. More **is** not always better, and the parameters that are truly meaningful to accomplish the purpose of the index, need to be examined guardedly,

In 1992, approximately 50 letters were sent out to various aquatic scientists, lake's managers, and regulatory individuals to recruit their help in identifying what parameters were important to include in an index. They were also asked to rank the parameters they selected in order of importance. About 30 individuals returned the survey with their assessments and this information was tabulated. The responses were surprisingly congruent.

Nine of the top twelve parameters were selected for use in the original 1993 index. Primarily because they were represented in years of historical data, and the Broward County Department of Natural Resource Protection (DNRP) was currently using these nine as **part** of the ongoing ambient monitoring program. The nine parameters are: dissolved oxygen, biochemical oxygen demand, total nitrogen, ammonium nitrogen, total phosphorus, turbidity, fecal coliform, fecal streptococci, and total organic carbon. Based on the importance of each parameter, as tabulated by the "experts" responses, each analyte was given a weighting factor. Biological parameters, e.g., benthic macroinvertebrates, were also listed as high importance, but the lack of representative data and current sampling regime prevented them from inclusion at that time.

To construct the formula, each individual parameter fractional subset was added together in a simple linear formula and no conversion factors were calculated, e.g., log transformations. The formula substituted current DNRP regulatory standards as the denominator (numerator for DO) for each parameter and the ambient laboratory chemistry analysis for the inverse variable. **A** weighting factor, as derived above, was multiplied against its subset. A baseline for water quality evaluation was calculated inserting the regulatory standard in place of the measured analyte. Then, the measured index number could be compared to the baseline. Measured values above the baseline were assumed to not meet water quality standards and were considered degradated waters. Those values below the baseline were considered as meeting current water quality designations. The index was developed and calibrated over a six-month period using the TSI **and** local expert input.

The general formula is given as:

$$WQI = \{w_1(p_i/s_i)] + ......+ [w_n(p_i /s_n)]$$

Where, w = the relative weighting factor
p = the specific measured parameter
s = the regulatory standard or expert water quality quantitative limit consensus

In **1998,** the water quality index developed for **Polk** County lakes depended heavily on the current water chemistry monitoring regime and parameters being weighted for local conditions. Current literature reviews were conducted to update the parameter list and the weighting that were assigned to each.

The ambient measured parameters along with the relative (weight) assigned to each are:

A = Dissolved oxygen in mg/L, (1S0)
B = Turbidity, in NTU, (0.50)
*C* = Ammonium nitrogen in mg/L, (0.75)
D = Total nitrogen in mg/L, (1.00)
E = Total phosphorus in mg/l, (1.00)
F = Chlorophyll a, in mg/$M^3$, (2.00)

The Polk County Index, substituting the regulatory standard (or consensus value) for x, is then given **as**:

$$WQI = [1.50(4.00/A + 0.01)] + [0.50(B/10)] + [0.75(C/0.10)] + [1(D/1.50)] + [1(E/0.05)] + [2(F/40.00)]$$

Substituting the standard for each measured analytes gives a baseline of 6.50. The formula was calibrated, "tweaked", against the TSI, using historical water quality data, and local lakes observations. **A** comparison of the WQX to the TSI is illustrated in Figures 1, and **2.** Visually the two indices track each other very well.

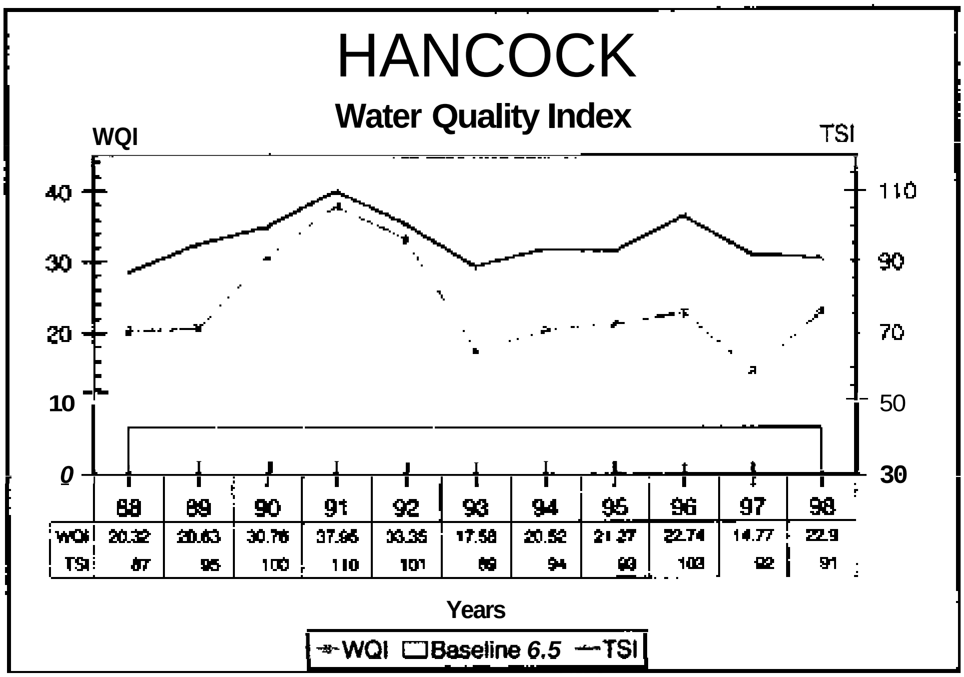


**Figure 1**

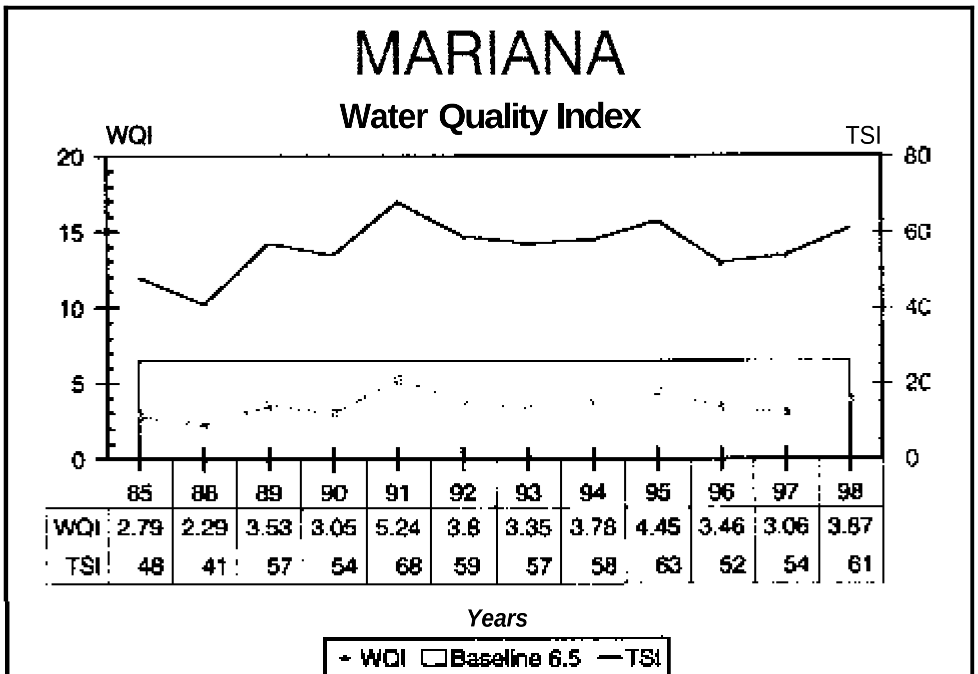


Figure 2

## CONCLUSION

The robustness of any index is always a concern. Other analytes, e.g., total organic carbon, will be added to the index during the next year as our laboratory completes the analyses. But it must also be kept in mind that Occum's Razor is an axiom best to adhere. The **Polk** County Index is currently used to help our managers and policy staff make informed decisions concerning activities that affect lake water quality. **As** additional data are made available, it is expected the WQI will become more refined and, as a management tool, will prove beneficial.

In recent years, many limnologists and other scientists have recognized that the biology component of some lake system needs to be included in any characterization scheme. Although there is increasing data available to include the biology in the index, additional data are needed to include bioassessement in **the** formula. Various habitat assessment **and** bioreconnaissance criteria are currently being evaluated for inclusion, as additional parameters, to improve the accuracy and reliability of the model. And finally, caution should always be applied when using any index model in lake management. Water quality indices are deducted from a small sample of complex, multidimensional factors and represent only a snapshot of the true lake condition and functionality.

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# FLOOD MANAGEMENT COORDINATION WITH LOCAL GOVERNMENTS

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## ABSTRACT

After El Niño, citizens began to focus on flood protection within the District. The need for a clear comprehensive document that identified the District's role in the area of flood protection with local governments and a long term plan identifying how each would coordinate their respective activities. The thought is to enter into a memorandum of understanding with local governments for the purpose of developing a "Flood Protection Management Plan" that would provide guidance into what services and information is available from the District and local governments. Some of the benefits to this type of **an** agreement and plan, is that local governments, and ultimately their citizens, will have a clear understanding of who is responsible for what elements of flood protection. In addition, and more important to the long term planning of both the District (primarily through the basin boards) and local governments would be the identification of preventive and restoration flood protection projects to **be** cooperatively funded. Having such a plan would benefit the District and local governments in developing their long term budgets.

# PERFORMANCE EVALUATION OF DRY DETENTION STORMWATER MANAGEMENT SYSTEMS

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## ABSTRACT

Field and laboratory investigations were conducted from August 1997 to March 1998 at a project site in DeBary, Florida to evaluate the hydraulic and water quality characteristics of a dry detention pond system constructed with a perforated pipe vertical filter system as an outlet control structure and anti-clogging device. The dry detention pond was constructed in 1996 to provide stormwater treatment for a 9.66 ha (23.86 ac) single-family residential watershed. Field instrumentation was installed at the dry detention pond site to conduct a complete hydrologic budget for the pond, including water level recorder, rainfall recorder, Class **A** pan evaporimeter, and groundwater piezometers. Automatic sequential samplers with integral flow meters were installed to provide continuous records of inflow and outflow from the pond and to collect stormwater and outflow samples on a flow-weighted basis.

On a mass basis, the dry detention pond was extremely effective in retaining mass inputs for all measured parameters. Overall mass removal for total nitrogen within the system was approximately 86%, with **84%** removal of total phosphorus, 99% removal of TSS, **82%** removal of BOD, and 88-**96%** removal for heavy metals. However, the magnitude of the mass removal efficiencies are due to the fact that more than 70% of the inputs into the pond were lost as a result of groundwater seepage through the pond bottom. On a concentration basis, the water column of the dry detention pond was capable of providing removal efficiencies of 30-90% for all input parameters with the exception of dissolved organic nitrogen, particulate nitrogen, total nitrogen, and BOD. Migration through the filter system provided little additional removal for most parameters,

The filter underdrain system was observed to exhibit highly variable hydraulic characteristics and was prone to clogging after only a few weeks of operation. Routine backwashing was necessary to maintain the filter system in an operational manner. In the absence of the substantial losses observed as a result of groundwater seepage from the pond, it appears that the filter underdrain system would be incapable of maintaining the pond in a near-dry condition.

## INTRODUCTION

One of the most common stormwater treatment methodologies used in Central and South Florida today for pollution abatement is dry detention. Stormwater inputs into a *dry* detention system are typically evacuated within 24-72 hours through an outlet structure, leaving the system in a "dry" condition between storm events. Dry detention systems are commonly used in high groundwater table areas where the normal groundwater level will not allow the use of a retention-type facility. Removal of particulates and associated pollutants by sedimentation within the pond is the primary physical removal process occurring in dry detention systems.

**A** common problem associated with the use of dry detention systems has been clogging of the outfall structure orifice used to regulate the discharge of water from the storage basin. In response to this persistent problem, the St. Johns River Water Management District (SJRWMD) published new criteria in 1994, as outlined in Chapter 4OC-42 F.A.C., which requires that outlet structures for dry detention basins contain a device to prevent the discharge of accumulated sediment, minimize exit velocities, and reduce clogging. Examples of such devices, provided by SJRWMD, include a perforated riser enclosed in a gravel jacket, along with perforated pipes enclosed in either sand or gravel. However, the performance efficiency of these new systems has not been evaluated.

### Study Site

Field and laboratory investigations were conducted from August 1997 to March 1998 at a dry detention pond site in DeBary, Florida. The dry detention pond was constructed in 1996 to provide stormwater treatment for a 9.66 ha (23.86 ac) single-family residential watershed with approximately 37% impervious coverage. Soils within the drainage basin are classified in Hydrologic Soil Group **A.**

The detention pond is constructed with a small vertical bottom filter system adjacent to the outfall structure according to criteria outlined by the SJRWMD in Chapter 40C-42 FAC. The filter system consists of a 10 cm (4 in) perforated PVC pipe covered with a filter fabric sock. The perforated pipe **is** approximately 3.3 m (10 ft) in length, with a 30 cm (12 in) layer of 20-30 silica sand on all sides of the perforated pipe covered by a 10 cm (4 in) top layer of FDOT coarse aggregate. Filter media used in the filter system met all applicable criteria for filter systems outlined in Chapter 62-25 **FAC.** Based on a total filter length of approximately 3.3 m (10 ft) and a width of 0.6 m (2 ft), the filter system provides a vertical filter area of approximately 2.0 $m^2$ (20 $ft^2$). The filter is the only drawdown mechanism provided for the detention pond other than **an** overflow weir designed at a 100-year flood elevation. The detention pond is designed to be maintained in a dry condition except during the drawdown period immediately following rain events. At the mean pond bottom elevation of 50 ft (MSL), the pond surface area is approximately 1515 $m^2$ (16,299 ft'). Discharges from the underdrain system flow to **an** adjacent final retention pond which is constructed in a depressional area with no direct off-site discharge. **A** schematic of the outfall structure with bottom filter system is given in Figure 1.

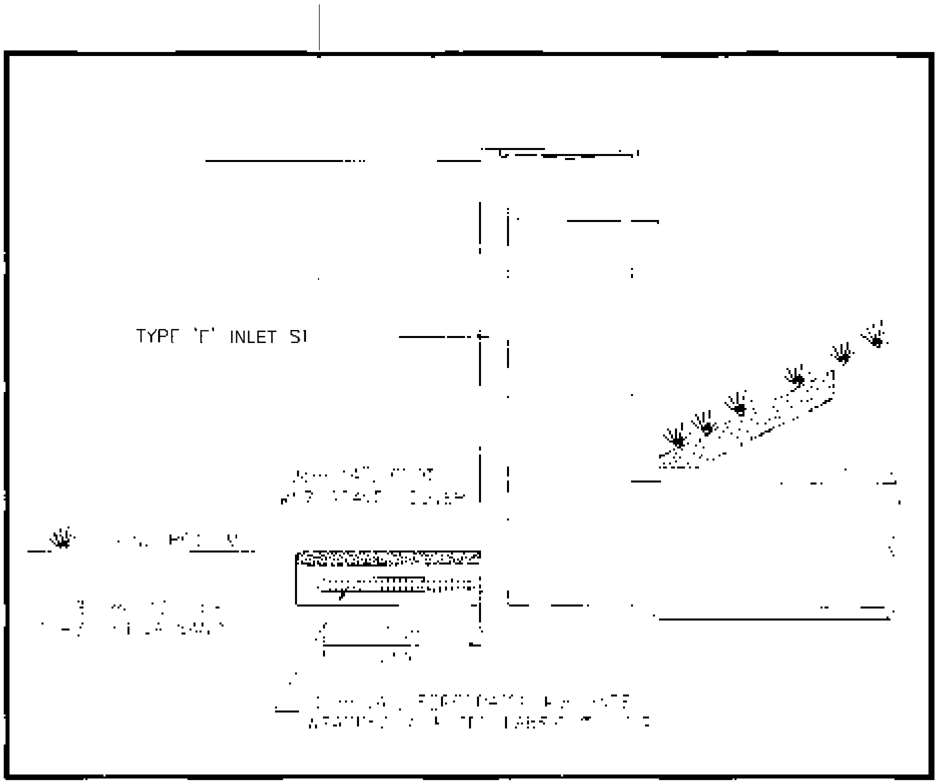


Figure 1. Schematic of dry detention pond outfall structure with anti-clogging device.

## MATERIALS AND METHODS

Field instrumentation was installed at the dry detention pond site to conduct a complete hydrologic budget for the pond, including a water level recorder, rainfall recorder, Class **A** pan evaporimeter, and groundwater piezometers. Automatic sequential samplers with integral flow meters were installed to provide continuous records of inflow and outflow from the pond and to collect stormwater and outflow samples on a flow-weighted basis, **A** total of 21 groundwater piezometers were installed along seven transects around the perimeter of the detention pond to

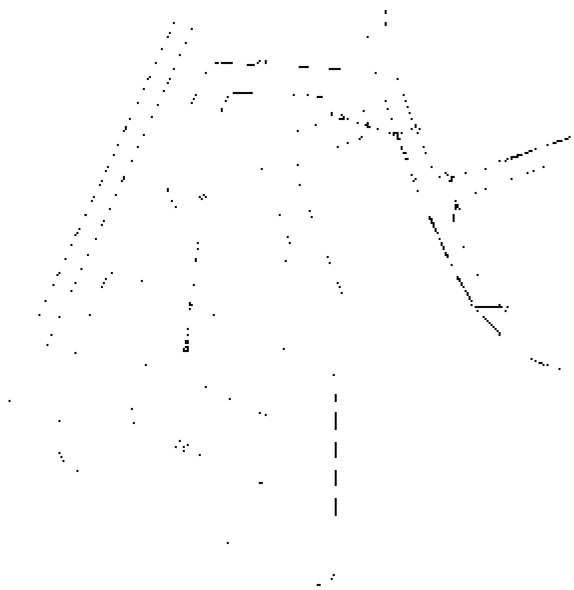

Figure 2. Field instrumentation.

provide information on horizontal groundwater gradients in the vicinity of the pond site. A schematic of field instrumentation used at the dry detention pond site is given in Figure 2.

Laboratory analyses were conducted on collected samples of stormwater runoff, dry weather baseflow, outflow, surface water, and bulk precipitation. Analyses performed included nutrients, general inorganic parameters, demand parameters, chlorophyll-a, oil and grease, TRPH, fecal coliform, and dissolved and total heavy metals. In excess of 80,000 separate field and laboratory measurements were generated during the course of this project.

## RESULTS

A continuous record of rainfall characteristics was collected at the DeBary site from August **10,** 1997 to March **1,1998** using a tipping bucket rainfall collector with a digital data logging recorder. Individual rainfall events ranged from 0.03-4.70 cm (0.01-1.85 in), with a mean of 0.9 cm (0.36 in) per rain event. **A** total of 64.4 cm *(25.35* in) of rainfall was measured at the site from August 1997 through February **1998.** Total daily evaporative losses at the site ranged from a high of 0.42 cm/day in September to **a** low of 0.18 cm/day during December.

Pond water surface elevations had **a** maximum fluctuation of approximately 0.8 m **(2.75 ft)** during the project period, Typical water depths in the pond throughout the project period ranged from approximately 15-30 cm (6-12 inches). The maximum measured water level in the pond of 51.85 ft **(MSL)** resulted in a maximum water depth of **0.72** m **(2.35 ft)**. Although the pond is designed as a dry detention pond, areas of standing water were present within the pond at all times throughout the 6-month assessment period. Based on piezometric elevations measured at the project site from September **1997** to February **1998,** no significant evidence of migration of groundwater into the pond was observed during the project period.

Continuous inflow hydrographs were recorded for inputs of stormwater and baseflow into the detention pond at 10-minute intervals from August 16, 1997 to March 1, 1998. Calculated runoff coefficients at the pond site ranged from a low of 0.102 in October to a high of 0.167 in February, with a weighted average runoff coefficient of 0.128.

During the 6-month sampling period, stormwater runoff contributed approximately 85% of the hydrologic inputs into the system, with approximately 12% contributed by direct rainfall and 3% contributed by dry weather baseflow. Based upon the results of piezometric measurements, no direct groundwater inflow into the pond is assumed. The dominant loss from the pond appears to be groundwater seepage which accounted for approximately 71% of the total pond losses. Underdrain outflow appears to account for approximately 20% of the pond losses, with evaporation comprising the remaining 9%. **A** comparison of overall hydrologic inputs and losses at the dry detention pond site is given in Figure **3**. Average detention time within the pond ranged from a low of 4.3 days in February to a maximum of 60 days in November.

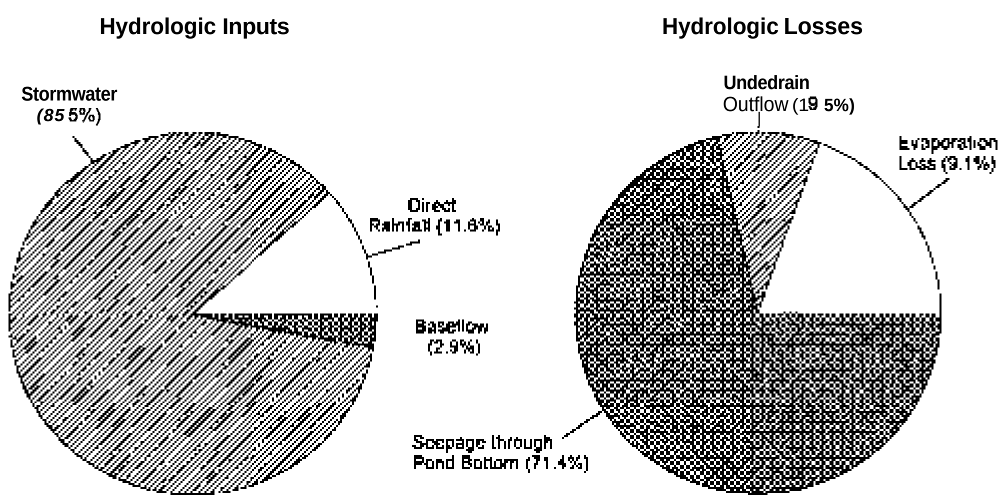


Figure **3**. Comparison of hydrologic inputs and losses.

## Characteristics of Stormwater and Dry Weather Baseflow

A total of 35 separate storm event composite samples were collected and analyzed over the 6-month sampling period, representing more than 64% of the total storm events which generated measurable runoff into the detention pond. Six composite baseflow samples were also collected at the inflow to the detention pond. A summary of chemical characteristics of runoff and baseflow measured at the site is given in Table 1.

## TABLE 1

## MEAN CHARACTERISTICS OF STORMWATER RUNOFF AND DRY WEATHER BASEFLOW MEASURED AT THE DRY DETENTION POND SITE

| PARAMETER | UNITS | STORMWATER RUNOFF MEAN[1] | BASEFLOW MEAN[2] | PARAMETER | UNITS | STORMWATER RUNOFF MEAN[1] | BASEFLOW MEAN[2] |
|---|---|---|---|---|---|---|---|
| pH | s.u. | 3.73 | 7.75 | Oil and Grease | mg/l | < 0.5 | < 0.5 |
| Spec. Conductivity | µmho/cm | 122 | 222 | TRPH | mg/l | < 0.5 | < 0.5 |
| Alkalinity | mg/l | 70.5 | 100 | Fecal Coliform | #/100 ml | 3412 | 372 |
| $NH_3$ | µg/l | 1374 | 615 | Cd - Diss. | µg/l | < 0.5 | 0.6 |
| $NO_x$ | µg/l | 283 | 601 | Cd - Total | µg/l | 0.6 | 2.3 |
| Diss. Organic N | µg/l | 1042 | 564 | Cr - Diss. | µg/l | 3 | 3 |
| Particulate N | µg/l | 1204 | 460 | Cr - Total | µg/l | 12 | 7 |
| Total N | µg/l | 3905 | 2240 | Cu - Diss. | µg/l | 2 | 3 |
| Ortho-P | µg/l | 153 | 316 | Cu - Total | µg/l | 4 | 5 |
| Particulate P | µg/l | 212 | 131 | Pb - Diss. | µg/l | < 2 | < 2 |
| Total P | µg/l | 383 | 467 | Pb - Total | µg/l | 6 | < 2 |
| Turbidity | NTU | 412 | 8.7 | Fe - Diss. | µg/l | 212 | 69 |
| Chloride | mg/l | 9.5 | 21 | Fe - Total | µg/l | 2985 | 686 |
| TSS | mg/l | 299 | 9.7 | Zn - Diss. | µg/l | 6 | 13 |
| BOD | mg/l | 5.8 | 2.4 | Zn - Total | µg/l | 35 | 28 |

1. n = 35 samples
2. n = 6 samples

The mean concentration of total nitrogen in stormwater runoff at the DeBary site is higher than concentrations of total nitrogen typically found in residential runoff in Central Florida, although this value was influenced substantially by elevated levels of total nitrogen measured during one or two individual storm events. In contrast, the mean total phosphorus concentration in stormwater runoff measured at the site is more typical of total phosphorus values normally measured in urban runoff, although substantially elevated total phosphorus concentrations were observed during several individual events. Stormwater runoff at the site was found to have elevated levels of turbidity and TSS. Measured concentrations of oil and grease and TRPH were found at or below minimum detection limits in all monitored stormwater samples, In general, measured concentrations of all heavy metals in stormwater runoff, with the exception of iron, were found to be extremely low in value. Each of the measured heavy metals was found to exist primarily in a particulate form.

Dry weather baseflow was found to have lower concentrations of both total nitrogen and total phosphorus than observed in stormwater runoff. Measured concentrations of heavy metals in baseflow samples collected at the site were found to be extremely low in value for virtually all measured metals. The majority of heavy metals in baseflow inputs were present as particulate metal species.

## Characteristics of Bulk Precipitation

A summary of measured characteristics of bulk precipitation is given in Table 2. In general, bulk precipitation was found to be acidic, low in ionic strength, and poorly buffered. Elevated levels of ammonia, organic nitrogen, nitrate, and total phosphorus were measured on several occasions, presumably impacted by clearing and burning activities on property adjacent to the site. Periods of elevated phosphorus concentrations appear to correspond with elevated levels of nitrogen, turbidity, and TSS, suggesting a significant influence from the burning and clearing activities. Relatively low levels of heavy metals were measured in bulk precipitation, although evidence of elevated concentrations during specific events are apparent for both total iron and total zinc. These elevated levels also appear to be related to the clearing activities observed at the adjacent site.

### TABLE 2

### MEAN CHARACTERISTICS OF BULK PRECIPITATION MEASURED AT THE DEBARY DRY DETENTION POND SITE FROM AUGUST 1997-FEBRUARY 1998[1]

| PARAMETER | UNITS | MEAN | PARAMETER | UNITS | MEAN |
|---|---|---|---|---|---|
| pH | s.u. | 6.72 | Turbidity | NTU | 6.7 |
| Spec. Conductivity | µmho/cm | 20 | Chloride | mg/l | 2 |
| Alkalinity | mg/l | 19.2 | TSS | mg/l | 13.2 |
| $NH_3$ | µg/l | 450 | Total Cadmium | µg/l | 0.5 |
| $NO_x$ | µg/l | 184 | Total Chromium | µg/l | 3 |
| Diss. Organic N | µg/l | 876 | Total Copper | µg/l | 1 |
| Total N | µg/l | 1510 | Total Lead | µg/l | < 2 |
| Ortho-P | µg/l | 113 | Total Iron | µg/l | 94 |
| Total P | µg/l | 191 | Total Zinc | µg/l | 12 |

1. n = 15 samples

## Characteristics of Pond Surface Water

A summary of mean chemical characteristics of pond surface water is given in Table 3. Visually, the detention pond was characterized by a green water column, presumably resulting from excess algal growth, and a relatively turbid appearance. The pond water column was well oxygenated on all monitoring dates, with field measured values of ORP indicating oxidized conditions within the pond for all monitoring events.

### TABLE 3

### MEAN CHARACTERISTICS OF DETENTION POND SURFACE WATER MEASURED AT THE DEBARY DRY DETENTION POND SITE FROM AUGUST 1997-FEBRUARY 1998[1]

| PARAMETER | UNITS | MEAN | PARAMETER | UNITS | MEAN |
|---|---|---|---|---|---|
| pH[2] | s.u. | 7.14 | Chlorophyll-a | mg/m$^3$ | 16.3 |
| Spec. Conductivity[2] | μmho/cm | 121 | BOD | mg/l | 12.2 |
| Diss. Oxygen[2] | mg/l | 9.3 | Oil and Grease | mg/l | < 0.5 |
| Oxygen Saturation | % | 107 | TRPH | mg/l | < 0.5 |
| ORP[2] | mV | 560 | Fecal Coliform | #/100 ml | 115 |
| Alkalinity | mg/l | 52.5 | Cd - Diss. | μg/l | < 0.5 |
| $NH_3$ | μg/l | 97 | Cd - Total | μg/l | 0.5 |
| $NO_x$ | μg/l | 52 | Cr - Diss. | μg/l | 3 |
| Diss. Organic N | μg/l | 921 | Cr - Total | μg/l | 4 |
| Particulate N | μg/l | 1859 | Cu - Diss. | μg/l | 1.2 |
| Total N | μg/l | 2929 | Cu - Total | μg/l | 1.2 |
| Ortho-P | μg/l | 15 | Pb - Diss. | μg/l | 3 |
| Particulate P | μg/l | 220 | Pb - Total | μg/l | 3 |
| Total P | μg/l | 257 | Fe - Diss. | μg/l | 225 |
| Turbidity | NTU | 56.7 | Fe - Total | μg/l | 1407 |
| Chloride | mg/l | 4 | Zn - Diss. | μg/l | 3 |
| TSS | mg/l | 42.6 | Zn - Total | μg/l | 6 |

1. n = 27 samples
2. Field measured value

Measured concentrations of total nitrogen and total phosphorus in the pond water were elevated on many monitoring dates due to a high percentage of particulate species. Surface water within the pond typically exhibited low concentrations of fecal coliform bacteria. Measured concentrations of chlorophyll-a within the pond were highly variable throughout the monitoring period, ranging from 0.2-70.2 $mg/m^3$. In general, measured concentrations of heavy metals in pond surface water were found to be extremely low in value and substantially lower than input concentrations measured in stormwater runoff.

## Characteristics of Underdrain Outflow

Mean chemical characteristics of underdrain outflow are summarized in Table 4. In general, chemical characteristics of underdrain outflow appear to be similar to those found in pond surface water. Migration through the underdrain appears to result in a slight reduction in measured concentrations of total nitrogen. Measured concentrations of dissolved organic nitrogen and particulate nitrogen appear to decrease during migration through the filter media, while substantial increases are observed in measured concentrations of ammonia and nitrate. **A** slight reduction in particulate phosphorus is apparent during migration through the filter media, with a corresponding increase in soluble orthophosphorus in the underdrain outflow compared with the pond surface water. Measured concentrations of total phosphorus remain unchanged during migration through the filter,

## TABLE 4

## MEAN CHARACTERISTICS OF UNDERDRAIN OUTFLOW MEASURED AT THE DEBARY DRY DETENTION POND SITE FROM AUGUST 1997-FEBRUARY 1998[1]

| PARAMETER | UNITS | MEAN | PARAMETER | UNITS | MEAN |
|---|---|---|---|---|---|
| pH | s.u. | 7.28 | TRPH | mg/l | < 0.5 |
| Spec, Conductivity | µmho/cm | 114 | Fecal Coliform | #/100 ml | 724 |
| Alkalinity | mg/l | 53.4 | Chlorophyll-a | mg/m³ | 11.9 |
| $NH_3$ | µg/l | 149 | Cd - Diss. | µg/l | < 0.5 |
| $NO_x$ | µg/l | 174 | Cd - Total | µg/l | < 0.5 |
| Diss. Organic N | µg/l | 832 | Cr - Diss. | µg/l | 3 |
| Particulate N | µg/l | 1566 | Cr - Total | µg/l | 3 |
| Total N | µg/l | 2721 | Cu - Diss. | µg/l | 1.1 |
| Ortho-P | µg/l | 31 | Cu - Total | µg/l | 1.1 |
| Particulate P | µg/l | 208 | Pb - Diss. | µg/l | 3 |
| Total P | µg/l | 260 | Pb - Total | µg/l | 3 |
| Turbidity | NTU | 41.3 | Fe - Diss. | µg/l | 149 |
| Chloride | mg/l | 3 | Fe - Total | µg/l | 1113 |
| TSS | mg/l | 31.0 | Zn - Diss. | µg/l | 4 |
| BOD | mg/l | 9.8 | Zn - Total | µg/l | 7 |
| Oil and Grease | mg/l | < 0.5 | | | |

1. n = 48 samples

Underdrain discharges from the dry detention pond were found to exhibit chronic violations of Class III surface water quality criteria for turbidity, fecal coliform bacteria and total iron. Approximately one out of three measured outflow samples exceeded applicable Class III criteria for turbidity and total iron, with one out of five outflow samples exceeding applicable Class III criteria for fecal coliform bacteria.

## Estimated Removal Efficiencies for System Components

### System Mass Removal Efficiencies

Overall system removal efficiencies for the dry detention pond over the 6-month period are summarized in Table 5. On an overall mass basis, removal efficiencies for measured parameters ranged from approximately 82-99%, with an overall mass removal of 86% for total nitrogen, 84% removal of total phosphorus, 99% removal of TSS, 82% removal of BOD, and 88-96% removal for heavy metals.

**TABLE 5**

**ESTIMATED MASS REMOVAL EFFICIENCIES FOR THE DEBARY DRY DETENTION POND FROM SEPTEMBER 1997-FEBRUARY 1998**

| PARAMETER | UNITS | TOTAL INPUTS[1] | OUTFALL LOSSES[2] | SYSTEM REMOVAL (%) | PARAMETER | UNITS | TOTAL INPUTS[1] | OUTFALL LOSSES[2] | SYSTEM REMOVAL (%) |
|---|---|---|---|---|---|---|---|---|---|
| $NH_3$-N | g | 3,236 | 407 | 87 | TSS | kg | 3,368 | 41 | 99 |
| $NO_x$-N | g | 2,016 | 184 | 91 | BOD | kg | 37 | 6.8 | 82 |
| Diss. Org. N | g | 4,875 | 842 | 83 | Cadmium | g | 6.1 | 0.74 | 88 |
| Particulate N | g | 10,063 | 1,311 | 87 | Chromium | g | 119 | 5.7 | 95 |
| Total N | g | 20,123 | 2,749 | 86 | Copper | g | 37 | 1.5 | 96 |
| Ortho-P | g | 929 | 127 | 86 | Iron | g | 26,962 | 1,744 | 94 |
| Particulate P | g | 2,007 | 336 | 83 | Lead | g | 52 | 4.1 | 92 |
| Total P | g | 3,167 | 498 | 84 | Zinc | g | 207 | 10 | 95 |
| Chloride | kg | 37 | 3.9 | 89 | | | | | |

1. Sum of inputs from stormwater, baseflow, and bulk precipitation
2. Measured mass losses through the outfall structure

The extremely high mass removal efficiencies observed within the system are primarily due to the fact that only a small portion of the hydraulic inputs into the pond system left the pond through the underdrain outflow. More than 70% of the inputs into the pond were lost as a result of groundwater seepage through the pond bottom which carried a corresponding mass of pollutants as the water migrated through the bottom sediments and into adjacent groundwater. Although the dry detention pond appears to exhibit excellent mass removal efficiencies for all measured constituents, this assessment does not indicate that similar removal efficiencies can be achieved in a dry detention pond which does not have a significant loss component due to groundwater seepage.

## Concentration-Based Removal Efficiencies

### Pond Water Column

Concentration-based removal efficiencies were calculated to estimate pollutant attenuation which occurred only within the water column of the detention pond. These efficiencies were calculated by comparing the estimated weighted inflow concentrations for all measured inputs from stormwater runoff, dry weather baseflow, and bulk precipitation, with the calculated mean pond concentrations. A summary of concentration-based removal efficiencies for the pond water column is given in Table 6.

**TABLE 6**

**CONCENTRATION-BASED REMOVAL EFFICIENCIES FOR SYSTEM COMPONENTS**

| PARAMETER | UNITS | POND REMOVAL (%) | FILTER REMOVAL (%) | OVERALL SYSTEM REMOVAL (%) | PARAMETER | UNITS | POND REMOVAL (%) | FILTER REMOVAL (%) | OVERALL SYSTEM REMOVAL (%) |
|---|---|---|---|---|---|---|---|---|---|
| $NH_3$-N | µg/l | 76 | -182 | 31 | TSS | mg/l | 90 | 37 | 93 |
| $NO_x$-N | µg/l | 79 | -138 | 50 | BOD | mg/l | -171 | 62 | -2 |
| Dis. Org. N | µg/l | -54 | 38 | 5 | F. Coliform | #/100 ml | 97 | 27 | 98 |
| Part. N | µg/l | -51 | 53 | 28 | Cadmium | µg/l | 33 | 0 | 33 |
| Total N | µg/l | -19 | 37 | 25 | Chromium | µg/l | 73 | 14 | 75 |
| Ortho-P | µg/l | 87 | -473 | 25 | Copper | µg/l | 73 | 17 | 78 |
| Part. P | µg/l | 31 | -3 | 8 | Iron | µg/l | 57 | 17 | 64 |
| Total P | µg/l | 34 | -31 | 13 | Lead | µg/l | 52 | -7 | 56 |
| Turbidity | NTU | 90 | -8 | 89 | Zinc | µg/l | 76 | -17 | 72 |
| Chloride | mg/l | 0 | 25 | 42 | | | | | |

The pond water column appears to provide good removal efficiencies for inorganic nitrogen species, although increases in measured concentrations were observed for dissolved organic nitrogen, particulate nitrogen, and total nitrogen. Unlike the trend observed for species of nitrogen, measured phosphorus species exhibited consistent removals within the water column of the pond. On an overall basis, the pond water column appeared to be capable of removing approximately one-third of the total phosphorus input. The water column of the pond was also found to be capable of providing significant removal efficiencies for both turbidity and TSS, with an estimated removal efficiency of 90% for each parameter. Increases in measured BOD concentrations within the pond may be related to the use of the pond by waterfowl on a periodic basis. The pond provided excellent removal efficiencies for fecal coliform. The detention pond water column provided good removal efficiencies for all heavy metals, largely due to settling of particulate metal forms.

## Filter Svstem

Removal efficiencies were also calculated for changes in concentration during migration through the underdrain outflow system, Migration through the underdrain system appeared to reduce measured concentrations of dissolved organic nitrogen and particulate nitrogen, while increasing measured concentrations of ammonia and nitrate. On **an** overall basis, total nitrogen concentrations were reduced by approximately **37%** within the filter underdrain system. Although the filter system appears capable of removing total nitrogen, the long-term fate of these pollutants in the filter system is uncertain. However, migration through the filter media did not result in a measured reduction in phosphorus species. Measured concentrations of orthophosphorus, particulate phosphorus, and total phosphorus were found to increase in concentration during migration through the underdrain filter system.

Measured concentrations of turbidity were relatively unchanged during migration through the filter media. Turbidity within the water column of the pond appears to be a result of colloidal particles which are capable of migrating through the sand filter media. In contrast to the trends observed for turbidity, however, measured concentrations of **TSS** decreased by **37%** during migration through the filter. The filter underdrain system is also approximately 62% effective in reducing concentrations of BOD. Migration through the filter underdrain system also reduced measured concentrations of fecal coliform bacteria and chlorophyll-a. However, migration through the filter underdrain system was relatively ineffective in reducing measured concentrations of either dissolved or total heavy metals.

## Overall Svstem Removal Efficiencies

Concentration-based removal efficiencies were calculated for the overall detention pond system as the change between the weighted input concentration and the weighted output concentration discharging through the underdrain system. On an overall basis, the *dry* detention system was found to exhibit positive removal efficiencies for all measured parameters with the exception of BOD. An overall removal efficiency of **25%** was observed for total nitrogen, with an overall removal of 13% for total phosphorus. The dry detention pond system appears to be highly effective in reducing concentrations of certain particulate species, as evidenced by an **89%** removal efficiency for turbidity, **93%** removal efficiency for suspended solids, **and 98%** removal efficiency for fecal coliform. Measured removal efficiencies ranged from 56-78% for heavy metals.

## Performance Characteristics of the Filter Underdrain System

### Field Maintenance Activities

When field monitoring activities at the dry detention pond site first began in August 1997, it was discovered that the original filter underdrain system was inoperable. **As** a result, the original filter underdrain system was removed and reconstructed according to the original pond design details. After reconstructing the filter underdrain system, the hydraulic performance of the filter media was restored for a period of approximately two weeks. After this time, the hydraulic conductivity of the filter media decreased rapidly, becoming virtually totally clogged after a period of four weeks.

During September 1997, a backwash of the filter system was performed at a rate of approximately 15 gpm/ft$^2$ of filter area to fluidize the silica sand media, allowing trapped particles to escape. After completion of the backwash procedure, the original hydraulic performance of the filter media was restored for approximately **2-3** weeks, followed by a rapid decrease in conductivity of the filter media. Backwash attempts were again performed during October and November 1997 to maintain the hydraulic performance of the filter system. During the backwash procedures in November, it appears that the media became channelized, allowing water to migrate directly to the perforated pipe system, bypassing the filter media entirely.

### Hydraulic Characteristics of the Filter System

In general, recovery within the pond following rain events appeared to be relatively slow due to the poor hydraulic performance of the filter underdrain system. If the large groundwater loss component had not been present, the hydraulic function of the underdrain system would have been insufficient for maintaining the pond in a dry condition, and the pond would have rapidly filled to the 100-year weir overflow elevation.

Over the 6-month monitoring period, the filter system was found to exhibit a high degree of variability in calculated permeability (K) values. Normal operation of the underdrain filter system using clean filter media resulted in measured permeability values ranging from approximately **3-4** ft/day. Field measured permeability values began to approach zero as the filter became clogged between backwash events. When channelization of the filter system occurred, field measured permeability increased substantially to values ranging from approximately **5**-12 ft/day.

## CONCLUSIONS

Based upon the results obtained during this project, the following specific conclusions were reached:

1. On a mass basis, the dry detention pond was extremely effective in retaining mass inputs for all measured parameters. Overall mass removal for total nitrogen within the system was approximately 86%, with **84%** removal of total phosphorus, 99% removal of **TSS, 82%** removal

of BOD, and **88-96%** removal for heavy metals. However, the magnitude of the mass removal efficiencies obtained in this assessment are largely a function of the fact that more than 70% of the inputs into the pond were lost as a result of groundwater seepage through the pond bottom which carried a corresponding mass of pollutants as the water migrated through the sediments and into the adjacent groundwater. Similar removal efficiencies could not be achieved in a dry detention pond which did not have a significant loss component due to groundwater seepage.

2. The water column of the dry detention pond was capable of reducing input concentrations of all input parameters with the exception of dissolved organic nitrogen, particulate nitrogen, total nitrogen, and BOD, Measured increases in concentrations of these parameters within the pond may be related to the presence of waterfowl which were observed to utilize the pond on a periodic basis. Although the pond water column provided no net removal for total nitrogen, the pond was capable of reducing concentrations of total phosphorus by 34%, turbidity by 90%, **TSS** by 90%, fecal coliform by 97%, cadmium by 33%, chromium and copper by 73%, iron by 57%, lead by 52%, and zinc by 76%.

3. Migration through the filter media was capable of reducing measured concentrations of dissolved organic nitrogen, particulate nitrogen, total nitrogen, **TSS,** BOD, fecal coliform, and chlorophyll-a. The filter media appeared to exhibit poor removal efficiencies for phosphorus species and heavy metals. Conversion of trapped particulate matter in the filter media into dissolved forms was observed for several parameters, such as ammonia, nitrate, and orthophosphorus.

4, The filter underdrain system was observed to exhibit highly variable hydraulic characteristics and was prone to clogging after only a few weeks of operation, Routine backwashing was necessary to maintain the filter system in an operational manner. In the absence of the substantial losses observed as a result of groundwater seepage from the pond, it appears that the filter underdrain system would have been incapable of maintaining the pond in **a** near-dry condition.

## RECOMMENDATIONS

Based on the results obtained during this project, and the specific conclusions presented previously, the following recommendations are made for improving the performance of dry detention systems:

1. Further use of orifice anti-clogging devices which are similar in design to the anti-clogging device investigated during this project, should be discontinued. Evidence gathered during this project indicates that these vertical filer systems exhibit extremely variable hydraulic characteristics and are subject to clogging after relatively short run times. Continual maintenance will be required for these systems to maintain the filter media in an operational

mode. In addition, these anti-clogging devices provide little additional pollutant attenuation for the overall stormwater system.

2. Due to the rapid potential for clogging, dry detention systems constructed according to current **SJRWMD** criteria should be inspected and maintained at a frequency not to exceed once each month. Field maintenance activities may include filter backwashing, replacement of filter media, or other options necessary to maintain the hydraulic performance of the system.

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# COMMUNITY RESPONSES TO RUNOFF POLLUTION: FINDING FROM CASE STUDIES ON STORMWATER POLLUTION CONTROL

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## ABSTRACT

Urban stormwater runoff poses a serious threat to the nation's water resources. Concerns about urban runoff and interest in proposed new federal stormwater regulations prompted documentation of existing effective stormwater strategies. The purpose of this documentation is to encourage municipal action and help empower communities to address this critical issue. To achieve this goal, more than 150 examples of effective strategies from across the nation were evaluated and compiled. The case studies highlight effective pollution prevention, administrative, and financing measures for addressing stormwater runoff. They show on a practical level that stormwater management can be environmentally effective, economically advantageous, and politically feasible. In Addition, they offer an outline for further successful stormwater management strategies. Elements critical to the effectiveness of these programs include: a pollution prevention emphasis with structural treatment measures when needed; a focus on preserving natural features and processes; a framework that creates and maintains accountability; a dedicated and equitable funding source to ensure long-term viability; strong leadership; and effective administration. These broad themes translate into a set of nine local actions for addressing the technical, social, and political issues associated with stormwater runoff. Following these actions will help communities form a sound stormwater policy.

**KEY TERMS:** urban stormwater runoff; impervious surfaces; pollution prevention; best management practices; diffuse pollution; accountability.

## INTRODUCTION

Pollution from all diffuse sources, including urban stormwater pollution, is considered to be the most important source of contamination in the nation's waters (U.S. Environmental Protection Agency, 1997a). Specifically, urban and suburban runoff is the second most prevalent source of water quality impairment in the nation's estuaries after industrial discharges, and the fourth most prevalent source of impairment in lakes after agriculture, unspecified nonpoint sources, and atmospheric deposition of pollutants (U.S. Environmental Protection Agency, 1998b). Uncontrolled urban runoff also contributes to hydrologic and habitat modification, two important sources of river impairment identified by the U.S. Environmental Protection Agency (EPA).

The polluted stormwater runoff problem has two main components: the increased volume and rate of runoff from impervious surfaces and the concentration of pollutants in the runoff. Both components are closely related to development in urban and urbanizing areas (Booth and Reinelt, 1993; Schueler, 1994; U.S. Environmental Protection Agency, 1997b). When impervious cover (roads, highways, parking lots, and roof tops) reaches between 10 and 20 percent of the area of a watershed, ecological stress becomes clearly apparent (Klein, 1979; Booth and Reinelt, 1993; Schueler, 1994). Everyday activities can deposit on these surfaces a coating of various harmful materials. When it rains or when snows melts, many of these pollutants are washed into receiving waters, often without any treatment.

The deposition of pollutants and the increased velocity and volume of runoff together, cause dramatic changes in hydrology and water quality (Klein, 1979; Jones and Clark, 1987; Booth, 1990; Galli, 1990; US Environmental Protection Agency, 1997b). These changes affect ecosystem functions, biological diversity, public health, recreation, economic activity, and general community well-being (Bannerman *et al.*, 1993; Novotny and Olem, 1994; Haile *et al.*, 1996; Carpenter *et al.*, 1998). Urban stormwater is not alone in polluting the nation's waters. Industrial and agricultural runoff are often equal or greater contributors. But the environmental, aesthetic, and public health impacts of diffuse pollution will not be eliminated until urban stormwater pollution is controlled.

Currently, there is substantial concern about the impacts of urban and suburban runoff. Stormwater runoff pollution is an important issue since most of the population of the United States lives in urban and coastal areas. Water resources in urban and coastal areas are highly vulnerable to and are often severely degraded by stormwater runoff. Economic impacts are another important aspect of this concern. Even a partial accounting shows that hundreds of millions of dollars are lost each year through added government expenditures, illness, or loss in economic output due to urban runoff pollution and damages (U.S. Environmental Protection Agency, 1998a). The ecological damage is also severe and is at least as significant.

While urban and suburban runoff continues to be a critical issue, there is substantial evidence that the problems are not intractable. Increasingly, communities are recognizing the causes and consequences of uncontrolled urban runoff and taking action to control and prevent runoff pollution, often without any mandate, These innovative communities are realizing the environmental, economic, and social benefits of preventing stormwater pollution. However, neither the extent of these efforts nor the specific actions being taken have been well documented.

There is also a growing interest in proposed new federal stormwater regulations. Comprehensive stormwater regulation is required under Section 402(p) of the Clean Water Act. Since 1992, cities with populations over 100,000, certain industries, and construction sites over *5* acres have been required to develop and implement stormwater plans under Phase I of the National Pollutant Discharge Elimination System (NPDES) stormwater regulations (U.S. Environmental Protection Agency, 1990). In October 1999, **EPA** is expected to promulgate a new rule requiring municipalities with populations fewer than 100,000 people located in "urbanized areas" (where population density is greater than 1,000 persons per square mile) to develop stormwater plans. Under what is known as the "Phase II" rule, the **EPA** and states will develop "tool boxes" from which the smaller local governments can choose particular stormwater strategies to develop their stormwater plans (U.S. Environmental Protection Agency, 1998a).

To address these issues and concerns, we developed a study to examine, document, and disseminate information on successful stormwater pollution prevention efforts. The primary goal of this study was to document environmentally effective and economically advantageous stormwater pollution prevention strategies. The study resulted in a report, ***Stormwater Strategies:*** *Community* ***Responses*** *to Runoff* ***Pollution,*** that highlights some of the most effective existing stormwater strategies from around the country (Lehner ***et al.,*** 1999). The report provides substantial evidence that such programs exist and highlights a variety of innovative strategies actually being used. The report also aims to provides guidance to communities addressing stormwater issues, encourage municipal action, and help empower communities to be involved in this critical issue, This paper summarizes the study and presents its primary findings and recommendations.

## STUDY DESIGN AND OBJECTIVES

The study was exploratory in nature, with the intent of presenting information on existing effective stormwater management programs. To achieve this goal, we collected cases of environmentally beneficial and cost-effective stormwater programs from across the country. We compiled this information into the case-study-based report described above. This information and report is now the basis for a comprehensive outreach effort.

The first step was to gather information on programs and projects by examining existing programs (several now under Phase I requirements as well **as** many that started earlier), reviewing literature, contacting regional and local stormwater management experts and researchers, and interviewing representatives from stormwater management or other local government agencies. We gathered information on over *250* programs. The information was then examined in detail and narrowed down to a set of case studies that demonstrated some element of success. Three fundamental criteria for selection were used: environmental gains, economic advantages, and community benefits. Environmental gains included biological, hydrological, or chemical improvements resulting from stormwater management. Economic advantages included cost savings to the municipality or developer, or increases in property values related to the pollution prevention measure. Community benefits included aesthetic or recreational enhancement, administrative or institutional successes, or community relations improvements.

Seventy-seven programs and projects were selected as case studies for the final report. Another 88 programs were annotated to provide additional references that were not fully evaluated for the report. The case studies represent communities of all sizes, types, and regions throughout the United States. To help ensure accuracy, local experts or people familiar with the program, called "groundtruthers," were contacted to review the case studies and add information from their own knowledge and experience.

The case studies were first organized geographically by dividing the United States into six regions based in part on general rainfall patterns. Within each of the regions, case studies were then further subdivided into the following five categories of stormwater management measures: addressing stormwater in new development and redevelopment; promoting public education and participation; controlling construction site runoff; detecting and eliminating improper or illegal connections and discharges; and implementing pollution prevention for municipal operations. These categories roughly parallel those measures that large municipalities currently address under Federal regulations (40 CFR parts **122.26** and **123.25)** and small municipalities will address under pending Federal regulations (U.S. Environmental Protection Agency, 1998a).

## Findings

Through over 150 examples of actual programs, this report provides substantial evidence that stormwater pollution can be prevented with proper planning and implementation in growing or redeveloping areas. The examples presented in the report also demonstrate that if the communities highlighted can measurably and cost-effectively reduce stormwater pollution, so can other communities and states.

## Stormwater Management Measures

Individually, the case studies provide detailed examples of substantial water quality improvement, effective or innovative stormwater control strategies to protect the natural environment, significant cost-savings, and important ancillary benefits to the community. The programs and strategies highlighted come from communities of all sizes, types, and regions, They include efforts by municipal agencies, developers, and community groups. In many cases, several of these groups worked together to create win-win outcomes, The case studies highlight a variety of strategies for addressing runoff in new development and redevelopment, promoting public education and participation, controlling construction site runoff, detecting and eliminating improper or illegal connections and discharges, and implementing pollution prevention for municipal operations.

***Addressing Stormwater** in **New Development and Redevelopment.*** By far the most important category of stormwater strategies focuses on land use and development. It encompasses a wide range of measures including regional or watershed planning, buffers and open space preservation, infill development, conservation design, and the use of site-specific structural and nonstructural treatment measures. One of the best strategies a municipality or developer can employ is to minimize the aggregate amount of new impervious surfaces, since where impervious surface does increase, treatment or control of runoff is needed. The case studies demonstrate that minimizing impervious surfaces, within desired growth targets, can be a highly effective and beneficial strategy. For example, the Magdalene Reserve development in Hillsborough County, Florida was able to reduce impervious cover and prevent runoff pollution while saving money, The developer did so by eliminating 25 percent of roadway, preserving existing trees and ground cover, reducing lot grading, installing swales and retention ponds to control runoff, using alternative landscaping techniques to reduce runoff pollution, and preserving 45 percent of the site as common open space. In addition, the more attractive houses sold better than conventional subdivisions (see Lehner *et al.*, 1999, p. 120).

***Promoting Public Education and Participation.*** Individuals play a key role in reducing stormwater impacts both in their own day-to-day activities and in showing support for municipal programs and ordinances. Effective public education, outreach, and participation programs are essential for involving citizens in pollution prevention activities, volunteer monitoring and inspection efforts, and the political and planning processes. The most successful programs highlighted accomplished three goals: they educated the public about the nature of the problem, they informed the people about what they can do to solve the problem, and they involved citizens in hands-on activities to achieve pollutant reduction or restoration targets. One example of this success is the University of Florida Cooperative Extension Service's programs that target landscape management. By teaching homeowners and professionals about the consequences of landscaping decisions and how to minimize environmental impacts by using sound practices, the extension service efforts have dramatically effected chemical use in 47 Florida counties (see Lehner ***et al.***, 1999, p. 129).

Controlling Construction Site Runoff. The case studies demonstrate that effective construction site pollution prevention is politically and economically feasible and can dramatically reduce pollution. In addition, these measures can have benefits for the developer as well: control measures such as phasing, mulching, and revegetation not only reduce erosion, but also have proven repeatedly to increase the value of the property (Herzog ***et al.*, 1998).** While existing programs rely on **a** fairly wide variety of erosion and sediment control practices, virtually all successful strategies require proper planning and phasing of construction activities to avoid disturbing more land than necessary during construction. The case studies demonstrate that the most effective programs rest on four cornerstones laid in pairs: enforcement **and** education; erosion prevention **and** sediment control. However, the first and over-aching necessity is a clear set of requirements. Chattanooga, Tennessee achieved greater compliance by taking this approach. The city developed **a** program with well defined erosion control requirements, a contractor education and certification program, and an aggressive inspection effort with stiff fines for noncompliance. Chattanooga's Erosion Control School has certified over 185 developers to date (see Lehner ***et al.***, 1999, p. 135).

***Detecting and Eliminating Improper or Illegal Connections and Discharges.*** Local governments have found that identifying and eliminating illicit connections and discharges is a remarkably simple and cost-effective way to eliminate some of the worst pollution from stormwater and to improve water quality. The case studies demonstrate that two factors are critical to success of this element of stormwater programs: tracking or finding illicit connections and discharges and enforcement. To find illegal discharges and illicit connections, the most successful programs use a range of techniques. Enforcement, however, is often the key to success. In Cohasset, Massachusetts, for example, enforcement orders mandating that private owners fix their septic systems resulted in the reopening of over 400 acres of shellfish beds. Citizens can also play an important role, In Alabama, the Alabama Water Watch Association and the Birmingham Stormwater Management Agency forged a partnership to train volunteers to help identify and detect illicit discharges by monitoring the city's 158 critical screening sites and outfalls.

***Implementing Pollution Prevention for Municipal Operations. A*** wide range of municipal operations can affect stormwater quantity and quality. The case studies reveal that some local governments have been able to manage their municipal operations to make a significant positive contribution to reducing stormwater pollution. The municipalities highlighted have done so in a variety of ways including reducing the use of harmful chemicals in the maintenance of municipal properties and vehicles, improving the maintenance and cleaning of roads and stormwater infrastructure, and training staff in pollution prevention practices. Several municipalities have taken these steps at their golf courses. For example, the Legacy golf course in Springfield, Tennessee is preventing runoff pollution by taking the following actions: maintaining an uncultivated natural buffer and 25-foot no-spray zone around all waterbodies; designing water hazards as stormwater retention ponds; and practicing integrated pest management (IPM). The course currently uses 75 percent organic or slow-release fertilizers and has significantly reduced the use of chemical pesticides. In addition to protecting the environment, the turf management approaches used at the Legacy have saved the course money (see Lehner ***et al.*, 1999, p. 142).**

## The Foundation of Success

Collectively, the case studies present a clear model for success. Evaluation of the case studies revealed several common elements among the highlighted programs. We distilled these elements into the broad themes listed below to help guide communities as they develop or improve stormwater programs. Since they are based on actual programs, these themes form a solid foundation for successful programs.

***Preventing pollution is highly effective and saves money.*** There are a range of measures know as "pollution prevention" that dramatically and cost-effectively reduce the quantity and concentration of pollutants winding up in stormwater. Common pollution prevention measures include reducing or eliminating the use of products with harmful chemicals, preventing erosion at construction sites, reducing the amount of pavement in new developments, and changing maintenance practices at home and in businesses or municipal operations. In highly urbanized areas, however, such measures may not be possible. In such cases, several communities have found treatment of runoff with structural measures or retrofitting existing structures to be effective alternatives.

***Preserving and utilizing natural features and processes have many benefits.*** Many communities and developers have found strategies that rely on natural processes to be highly effective and efficient. Undeveloped landscapes absorb large quantities of rainfall and snowmelt; vegetation helps to filter out pollutants from stormwater. These communities have benefitted from implementing environmentally friendly alternative site design or "greenfrastructure" by saving money and optimizing open space. Buffer zones, conservation-designed development, sensitive area protection, or encouragement of infill development all try to enhance natural processes and are among the most effective stormwater programs highlighted.

***Educuting and informing the general public and municipal staff improves program effectiveness.*** Providing information and training to the general public and local businesses is a key component to many of the highlighted programs. Public participation and education form a link between local governments and their citizens. Education programs encouraging citizens to change their habits and to contribute to cooperative efforts often form an early element of stormwater programs. Since many sources of stormwater pollution are derived from individual activities such as driving and maintaining homes, educating the public goes a long way to reducing stormwater pollution. Several communities involve the public in civic activities such as monitoring water quality or stenciling storm drains, which not only provide educational opportunities but also save the municipality money.

***Strong incentives, routine monitoring, and consistent enforcement establish accountability.*** Enforcement, or more broadly accountability, is a key element to improving water quality. All actors need a clear statement of performance goals, and they need to be held accountable by all the others for accomplishing these goals. We found that programs with high accountability were the most effective, often achieving pollutant reductions of 50 percent or greater.

***Establishing a dedicated source of funding builds strong support.*** Effective stormwater programs are financially viable and affordable. A stable funding source is critical to program success and community support. Stormwater fees have proven effective and popular for paying for necessary measures without political or community resistance. Nearly 200 Communities across the nation are already realizing the benefits of implementing stormwater utilities as dedicated and equitable funding sources.

***Strong leadership is often a catalyst for success.*** Success, at least at first, often requires an individual to champion the project and make it happen.

***Effective administration is critical.*** Regardless of which strategies a community chooses, those programs with clear goals and objectives were the most successful. Such clarity enhances accountability, responsibility, and trust, Furthermore, an established and understood institutional framework often improves administration by fostering collaboration among different parts and levels of government, neighboring communities, and local citizens. Effective administration allows implementation of broad-based, multi-faceted programs, which are often the most effective at controlling the diffuse problem of stormwater pollution.

## DISCUSSION: RECOMMENDATIONS FOR LOCAL ACTION

To further guide communities addressing stormwater runoff issues, we translated the broad themes presented above into an action plan based on nine key recommendations. These actions roughly parallel the broad themes presented above. The case studies demonstrated that following the nine local actions outlined below will help build a strong framework for effective, efficient, and successful stormwater management over the long term.

- ***Plan in advance and set clear goals.*** Carefully plan programs as opposed to simply reacting to provided opportunities, crises, or transient pressures. Planning allows development of more effective and cost-effective actions. An essential outcome of planning is to **address** the issues and concerns of all stakeholders involved. Planning does not require large staffs or extensive technology.

- ***Encourage and facilitate broad participation.*** Planning and program development processes should involve multiple levels of government, key members of the community, and professionals from a variety of related disciplines. Include and encourage planning, education, public participation, regulation, monitoring, and enforcement in stormwater programs. Key to this outcome is the public's understanding of the issue, how it relates to them, and what they can do about it, Look for public-public and private-public collaboration opportunities.

- ***Promote public education opportunities.*** Implement broad-based programs that reach a range of audiences and solicit different levels of public involvement. Remain committed to the education program and take advantage of existing community organizations to enhance participation. Educating and informing the public not only helps to reduce pollution, it also builds support for municipal stormwater programs.
- ***Work to prevent pollution first; rely on structural treatment only when necessary.*** Focus on prevention-based approaches, through regional and watershed planning, local zoning ordinances, preservation of natural areas, stormwater-sensitive site design, widespread compliance with dumping and connection prohibitions, erosion prevention, and broad-based education as these are significantly more effective **than** treatment of polluted runoff.

- ***Establish and maintain accountability.*** Essential components of this process are setting clear standards, creating strong incentives and disincentives, conducting routine monitoring and inspections, keeping the public informed, promoting public availability of stormwater plans and permits, and consistently enforcing laws and regulations. Consider and encourage innovative strategies and approaches, Strong enforcement is often key to significant water quality improvements.

- ***Create a dedicated funding source.*** Dedicated funding sources, such as stormwater utilities or dedicated environmental fees, help ensure that stormwater programs are stable over time **and** help gain public support. **Also** consider budget-saving measures such **as** creative staffing, public-public and public-private collaboration, and building off existing programs.

- ***Tailor strategies to the region and setting.*** Recognizing that every case will be different, consider strategies that are particularly tailored to the region, the specific audience, and the problem.

- ***Evaluate and allowfor evolution ofprograms.*** Set clear goals and priorities, and allow programs to develop over time. Establish clear ways to check and see that goals and objectives are being met. This opens opportunities for improvement and helps ensure long-term success.

- ***Recognize the importance of associated community benefits.*** Stormwater pollution measures usually offer ancillary quality-of-life benefits in addition to targeted improvements. For example, preserved areas offer parks, ponds offer beauty and habitat, clean streets are more attractive, education helps empower people, and sediment control improves fisheries and prevents flooding.

## CONCLUSION

Many fine handbooks provide theoretical and technical guidance concerning the design and implementation of effective stormwater pollution prevention and control measures. This study took a different approach. Focusing on existing effective programs in a variety of settings accomplished two key goals. First, the study demonstrates that stormwater management does not have to be overwhelming. The case studies show on a practical level that stormwater management can be environmentally effective, economically advantageous, and politically feasible. Second, the case studies enable communities developing or improving stormwater programs to learn from their peers. In doing so, the case studies offer an outline for future successful stormwater management strategies.

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# STORMWATER MASTER PLAN IMPLEMENTATION IN CENTRAL FLORIDA

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## ABSTRACT

Municipalities spend considerable funds and effort developing stormwater management master plans for basins within their jurisdiction. Once completed, recommended stormwater quantity and quality improvements identified as a high priority need to be implemented in a timely fashion to show a return to the community on their investment. This paper presents a case study of master plan implementation by the City of Rockledge to correct existing flooding and water quality problems through stormwater system retrofit.

## INTRODUCTION

This paper describes the actions taken by the City of Rockledge in the implementation of stormwater master plans, There are several issues that any city must consider during the implementation process, such as:

- Program goals (e.g. flood control, water quality protection, aquifer recharge, and wetlands management)
- Prioritization
- Funding
- Permitability, and
- Public acceptance

How the City of Rockledge managed these issues in the implementation of stormwater master plans is described below.

The City of Rockledge began its stormwater management program in **1989.** During the course of the Stormwater Management Needs Assessment **(SMNA),** a major storm produced over 5 centimeters (13 inches) of rainfall within an 8-hour period in October 1989. This storm resulted in severe flooding in several areas of the City and provided the impetus for the City to implement solutions on an accelerated time schedule. Between **1989** and **1997,** Rockledge enlarged an existing stormwater pond near one of the worst flooded areas from the 1989 storm from **2.8** to 4.0 hectares (7 to 10 acres), negotiated with borrow pit operators to construct a 16-hectare (40-acre) regional detention pond adjacent to the City's main canal, joined with the City of Cocoa and Brevard County in the design and construction of a regional stormwater facility, installed a baffle box to treat runoff from a small outfall to the Indian River, and replaced several undersized culverts with CONSPAN bridge structures. The flooding problem areas from the **1989** storm and the improvements that have been started since then are presented in **Figure 1.**

However, the City realized that a comprehensive stormwater master plan would be essential to meet all of Rockledge's flood protection and water quality goals. The City wanted all of the flooding problem areas identified, proposed solutions preliminarily designed, and prioritized.

In **1998,** Rockledge (in conjunction with the St. Johns River Water Management District (SJRWMD)) contracted with Camp Dresser & McKee Inc. (CDM) to prepare a Stormwater Master Plan (SWMP). The SWMP laid out a foundation to meet the following program goals identified by the City:

- Flood control
- Water quality control
- Aquifer recharge (where possible)
- Wetlands management
- Effective operation and maintenance
- Public recreation and aesthetics

To meet these program goals, 15 structural projects were identified, including two already in the planning or design phase. Nine of the projects will have both water quantity and quality benefits. The remaining six projects were driven by maintenance concerns. Included in the proposed projects were six regional detention facilities to provide much needed flood protection and improve water quality. The locations of these proposed projects is presented in **Figure 2.** When all of the projects are implemented, approximately **84** percent of the City will be served by retrofit stormwater facilities. If the City were to fund these projects through conventional means, the cost would be approximately $36,000,000. It has been estimated that public-private cooperation with developers, borrow pit operators, FDOT, City of Cocoa, and Brevard County could reduce this to 10 to 15 million dollars making the plan more affordable.

After the projects were identified, they were prioritized based on flood protection benefits, water quality benefits, potential for being a multi-use facility, and permitability. The biggest problem the City faced was how to pay for the desired improvements without sacrificing their stormwater program goals. The City identified the following potential funding sources/payment options:

- Donated land for regional facilities
- Grants
- Public-private partnerships
- Partial funding from other local governments where a joint benefit is identified, and
- Stormwater utility fee

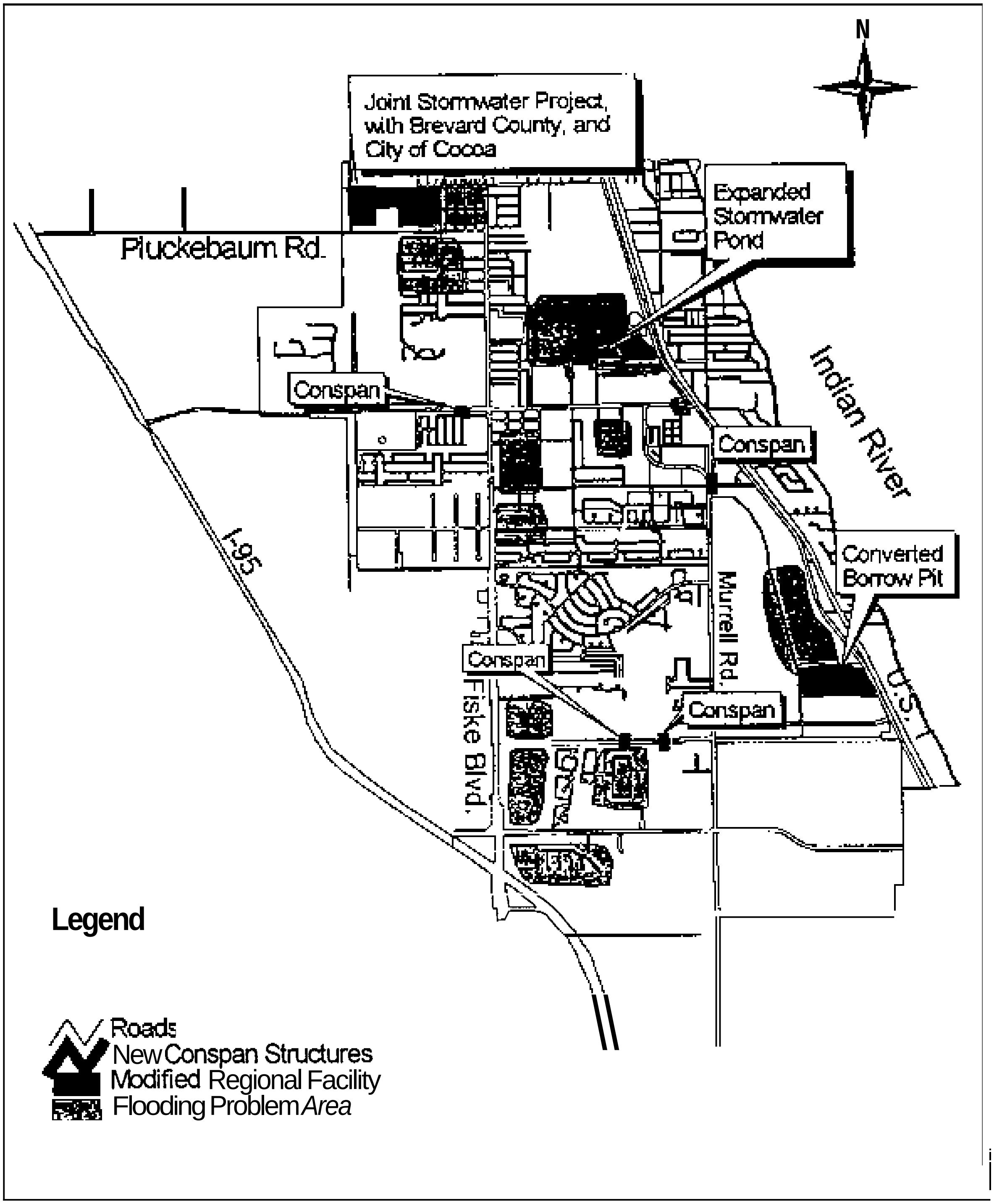


Figure 1

Flooding problem areas, stormwater improvements started prior to 1998

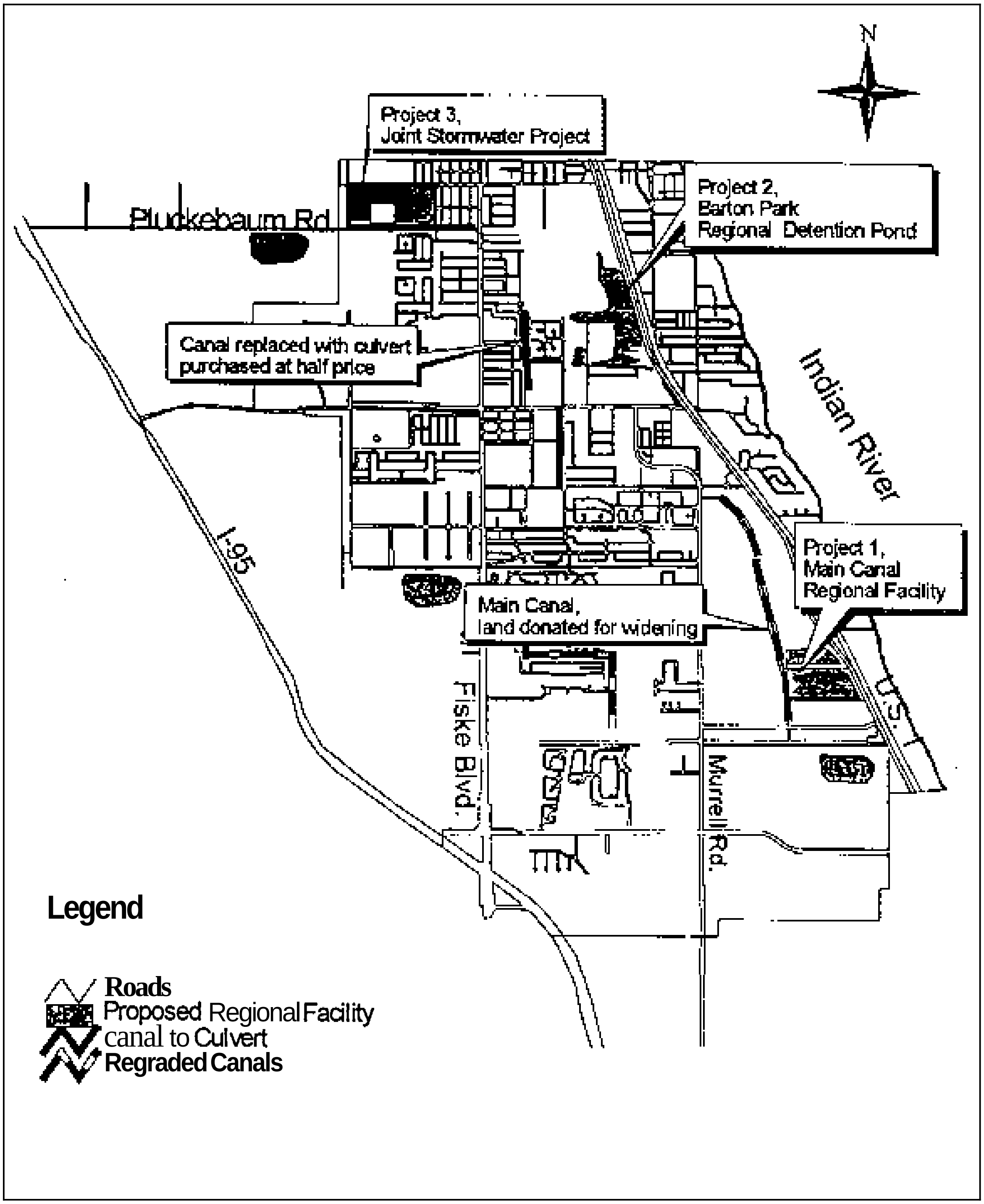


Figure 2

Proposed stormwater improvements

The first four funding sources/options are already being used by the City for projects startedjust prior to the SWMP, or since its completion. How these funding options were used for several individual projects is discussed in detail below.

The first project identified under the SWMP is the Main Canal Regional Facility (Project 1). One of the main drainage canals serving Rockledge flows from the north to the south between Murrel Road and U.S. 1. Near the south end of the canal, a borrow pit operator has two borrow pits currently in use. The City negotiated with the borrow pit operator (Watson Paving) to have the borrow pits donated to the City after the usable fill has been removed. The City will then convert the borrow pits to wet detention facilities and connect them to the canal. When completed, the former borrow pits will be two stormwater facilities comprising about **21.5** hectares **(53** acres) in area that will provide flood protection and water quality benefits to a large upstream area. In addition, the City is designing a community park around the borrow pits. The City received Florida Recreation Development Assistance Program (FRDAP) grants totaling $200,000 from the Florida Department of Environmental Protection (FDEP). This public-private partnership along with the state grants has allowed the City to begin this major project much sooner than otherwise would be possible by substantially decreasing the cost to the City since land purchase and excavation are the major costs associated with wet detention ponds.

The next project identified under the SWMP is the Barton **Park** Regional Detention Pond which is located at the northern end of main North-South Canal. During the **1989** storm, the Barton Park area experienced severe flooding. The first step the City took **was** to expand an existing stormwater pond from **2.8** to 4.0 hectares **(7** to 10) acres to provide some immediate relief, but a much larger storage facility is needed to prevent the flooding that occurred in 1989. The cost to design and construct the **24** hectares (60 acre) regional facility to provide the desired degree of flood protection is estimated to be over $10,000,000. The City undertook several initiatives to bring the estimated costs down. The first step was to obtain grant money from the Federal Emergency Management Agency (FEMA) to purchase the land for the proposed regional stormwater facility. **A** total of $792,000 dollars was obtained from FEMA for this purpose, which was slightly less than the total cost to buy the land **A** total of about 38.5 hectares (95 acres) of land was purchased, which includes some wetlands areas that will be preserved. Preliminary soil sampling has been done at the site which indicates that soil removed during construction will be suitable fill material. The City is currently negotiating with several borrow pit operators to have perform the excavation over a 5-year period, and expects to begin the design **of** the facility in the fall of 1999. By obtaining grant money to purchase the land and having borrow pit operators perform the excavation, the estimated cost to the City is expected to be a third of the original cost. In addition, the regional facility already has an associated park which will be expanded,

**A** third project involves another regional wet detention pond in the northwest area of the City which experiences chronic flooding. Since the area borders the City of Cocoa and unincorporated Brevard County, the City entered into a partnership with them to design **and** construct the facility. The design is being done by Brevard County, with all three entities providing funding for the project. In this way, a project is being constructed that will benefit property owners in three jurisdictions in a cost-effective manner.

Besides these major regional projects, Rockledge is making other improvements to the stormwater system. Much of the City is served by canals, some of which are very difficult to maintain due to accessability and steep side slopes. The City is planning to regrade some of the canals with steep side slopes, and pipe some of the canals that are inaccessible. The City was able to purchase used 122 centimeter (48-inch) RCP storm sewer at about half the cost of new culvert to replace about 244 meters (800 feet) of inaccessible canal with **pipes.**

Another example of the public-private partnerships that the City has undertaken involves the Main Canal. The City has negotiated with adjacent land owners and they have agreed to donate land by the canal so that a **2,285** meter-long (7,500-foot-long) section may be widened from an average of 7.6 to 9.1 meters (25 to 30 feet) to about 24.3 meters (80 feet). This will provide flood protection benefits to the adjacent landowners and to upstream areas.

By forming partnerships with other local governments, borrow pit operators, and landowners in conjunction with obtaining grants, Rockledge has been able to begin an aggressive implementation of the SWMP.

## CONCLUSIONS

Communities that develop long-term, phased SWMP capital improvements can speed implementation and develop public-private partnerships to significantly reduce costs and improve benefits. The City of Rockledge is an example of a community that has significantly implemented facilities using this approach.

## ACKNOWLEDGMENTS

We wish to thank the City of Rockledge for providing data for this paper. Special thanks go to James Gilliard, Public Works Director.

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# THE LAKE SEMINOLE WATERSHED MANAGEMENT PLAN: EVALUATION OF QUANTITY AND QUALITY OBJECTIVES

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## ABSTRACT

Once estuarine tidal flats, Lake Seminole was created in the 1940's by impoundment of an upper portion of Long Bayou. Lake Seminole has been used extensively throughout its existence for recreational purposes including skiing, boating, and fishing, as well as passive recreation. Recreational use, however, has declined in recent years as fishing, water clarity, and water appearance have all declined.

As a part of an overall management plan being developed for the lake and watershed by Pinellas County and the Southwest Florida Water Management District, a digital watershed and water body management model was developed using EPA's SWMM and WASP software. **SWMM** was used to address both water quantity and quality issues within the Lake Seminole Watershed. Water quality within Lake Seminole was then modeled using the water body model WASP. Due to the intrinsic differences between water quantity and water quality model simulations, and the different objectives of each model, two separate SWMM models were developed. A water quantity SWMM model addressed flooding and a water quality SWMM model provided pollutant loads to WASP.

Calibration was performed on the SWMM and WASP models using rainfall, runoff, and quality data collected in the field. The water quantity calibration was completed first in order to quantify the hydrologic and hydraulic characteristics of the watershed, which was followed by the water quality analyses. Although separate, the two models are similar in that the water quality SWMM model essentially provides the hydrological loads for each water quality simulation. This interrelationship is discussed, and how it related to the development of the Lake Seminole Watershed Master Plan.

## INTRODUCTION

The Lake Seminole Watershed encompasses approximately 1416 hectares (3,500 ac) of land within unincorporated Pinellas County and the incorporated cites of Largo and Seminole, and is almost entirely developed with urban land uses. The watershed was historically much larger than its current extent, however the limits of the watershed were altered in the early 1970s following construction of the Lake Seminole Bypass Canal for flood relief. The canal diverts the runoff from

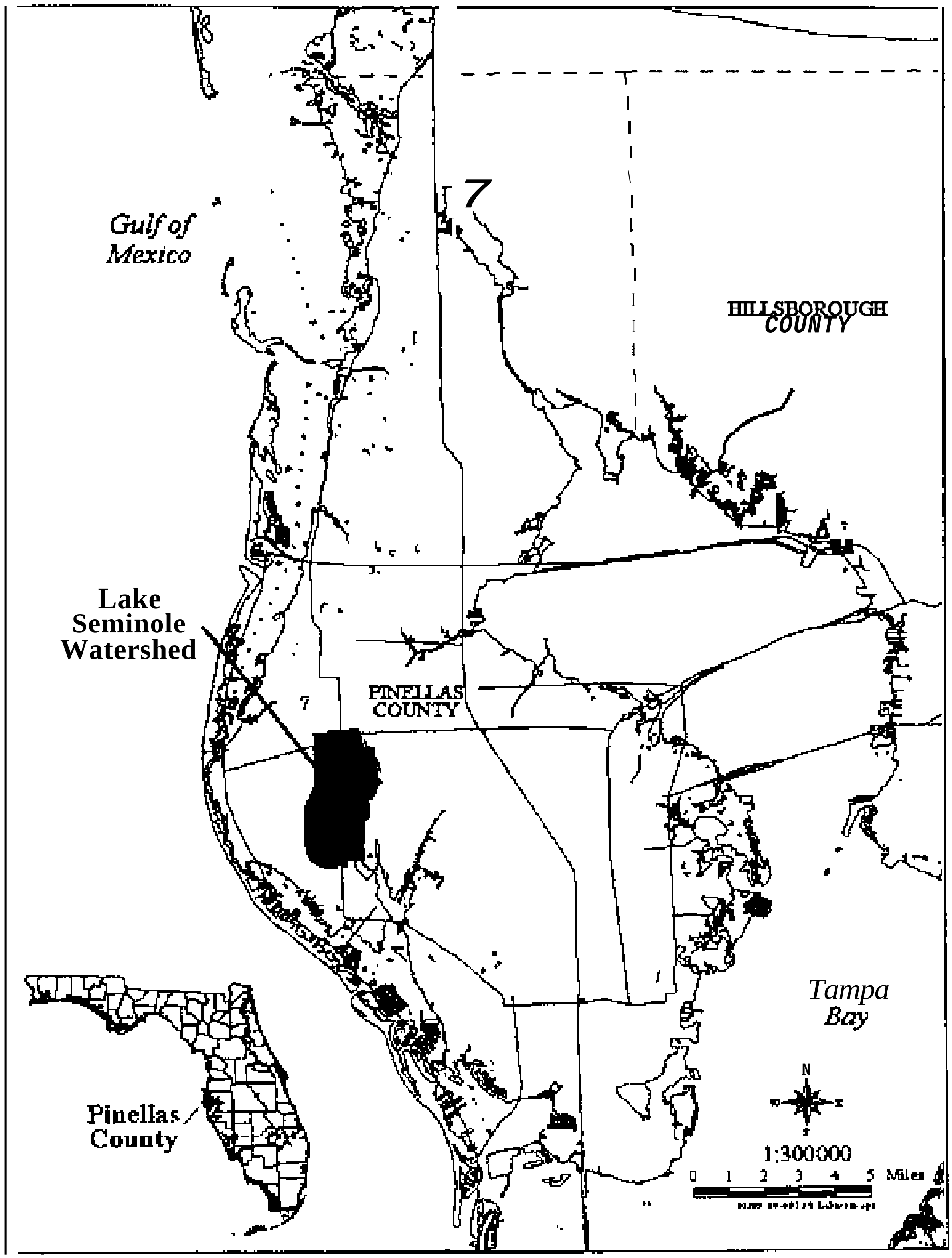



Figure 1 Lake Seminole watershed location map.

a large portion of the historical drainage area north and east of the lake, and discharges directly into Long Bayou through a separate structure along Park Boulevard east of Lake Seminole County Park.

Lake Seminole has a relatively small watershed area in relationship to total lake volume, This suggests that less storm water runoff is delivered to the lake per unit volume than for some other local lakes, Less lake water is replaced by runoff during a storm event, therefore, and the residence time for water within the lake is longer than if a larger drainage area contributed runoff. Long residence times may increase the potential for algae blooms **and** other symptoms of eutrophication (nutrient over enrichment). However, a smaller contributing drainage area also means that pollutant loading from nonpoint sources is likely lower than for large watersheds. Lower loadings can reduce the potential for eutrophication, and prove beneficial to the lake.

Unfortunately, the Lake Seminole Watershed appears to possess conditions that may foster eutrophication. The small drainage area allows a relatively small amount of runoff to enter the lake, thus increasing residence time. However, the highly urbanized drainage area produces runoff with relatively high concentrations of nutrients, metals, and other pollutants, thus enhancing the potential for water quality problems.

Pinellas County authorized PBS&J in late 1996 to assist in the preparation of the Lake Seminole Watershed Management Plan (LSWMP). The LSWMP is to be a comprehensive guide to managing the lake, and will include provisions for habitat protection and enhancement, water quality and flood protection and improvement, recreational opportunities, and aesthetic enhancement. **As** a part of this plan, the Lake Seminole Watershed Management Model (LSMM) was developed. This model was comprised of two separate but related components outlined below.

- Water quantity simulations were made using EPA's SWMM version **4.3** software. Hydrologic and hydraulic simulations for both the watershed and the lake itself were performed using the RUNOFF and EXTRAN computational blocks of SWMM.
- Water quality simulations provided non-point source pollutant loading estimates to the lake using the RUNOFF computational block of SWMM. These non-point source loads were then routed to Lake Seminole via the TRANSPORT computational block of SWMM. Water quality within Lake Seminole was then simulated using the DYNHAD and EUTERO subroutines of WASP version 4.0 software, The TRANSPORT block of SWMM and **WASP** water body model are currently under development, and are not discussed in this submittal.

Both the water quantity and water quality SWMM RUNOFF blocks were calibrated with respect to measured rainfall, stage, flow, and pollutant concentration data obtained from five sampling locations within the Lake Seminole Watershed during late 1997. Rainfall, stage, and flow data were used to develop stage-discharge relationships at the sampling sites, and to construct runoff hydrographs for each recorded storm event. Modifications were made to the input parameters of the water quantity portions of the LSMM to bring predicted stage and flow values at each sample location into closer agreement with recorded values for three calibration storm events recorded in late 1997. These calibrated hydrologic input data were then also used as input for water quality simulations, Laboratory analysis of flow-weighted water quality samples were used to develop Event Mean Concentrations (EMC's) for the following parameters: Total Nitrogen, Total Phosphorous, Total Suspended Solids, and BOD. Modifications were made to the input parameters of the water quality portions of the LSMM to bring predicted pollutant loading values at each sample location into closer agreement with laboratory results of water quality samples collected during three calibration storm events recorded in late **1997.**

Results of surface water quantity simulations of the LSMM were used to predict flow rates and stages within the watershed during design storm simulations. The goal of the water quantity simulations conducted using the LSMM was to identify existing and potential future flood prone areas within the Lake Seminole Watershed. Water quantity simulations for the 100-year 24-hour, 25-year 24-hour, and 25-year 6-hour storm events under existing and future land use conditions were developed to predict flooding potential in the Lake Seminole Watershed under existing and projected ultimate build-out land use scenario.

Results of surface water quality simulations of the LSMM were used to predict non-point source loadings to Lake Seminole during three separate year-long continuous simulations. Water quantity simulations for an "average", "wet", and "dry" rainfall year under existing and future land use conditions were developed to predict pollutant loadings to Lake Seminole from the surrounding watershed under existing and projected ultimate build-out land use scenario.

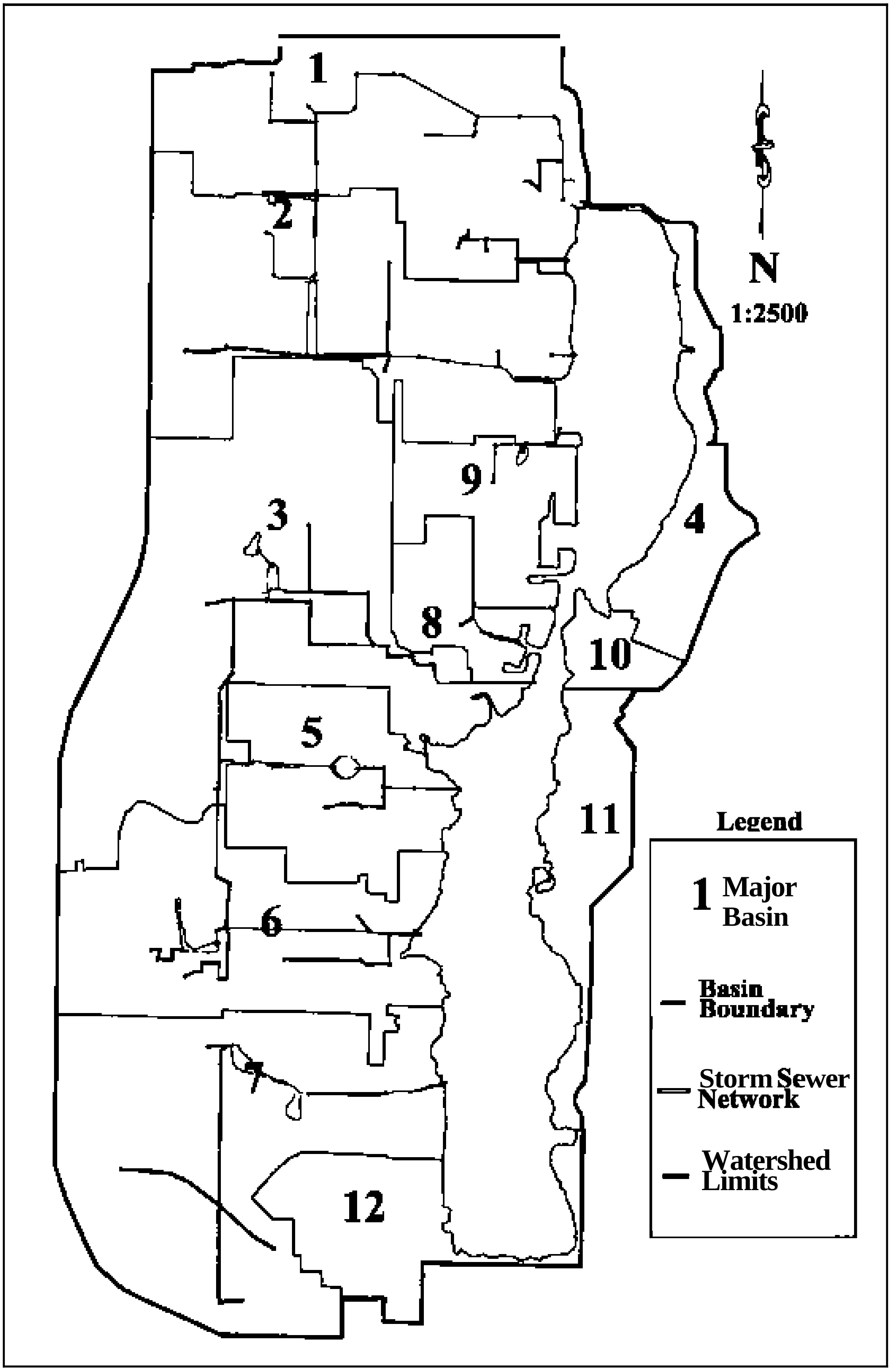
Based on these modeling results, alternatives can be developed to manage existing water pollution problems, and potential flood problems. Ultimately the LSMM will be used to predict the effects of various lake management actions on water quality, living resources, **and** flood control.

## MATERIALS AND METHODS

Many models are readily available for simulating surface water flow and flood water elevations. Pinellas County, however, requested that SWMM be used to conduct the floodplain analysis and watershed water quality simulations. Surface water runoff flows and pollutant loads were generated and routed through several subroutines of the SWMM model. Rainfall and watershed characteristic data were input into the RUNOFF block of SWMM, which computed runoff hydrographs for each subbasin. These hydrographs were written to an interface file, which allowed data transfer to other SWMM computational blocks, including EXTRAN and TRANSPORT. EXTRAN was used in water quantity simulations to estimate design storm flows and levels, and TRANSPORT was used in the water quality simulations to deliver pollutant loads to the WASP water body model. TRANSPORT and WASP model components are still under development, and not included in the discussion below.

### RUNOFF Block Input Parameters

Watershed boundaries, basins, and subbasins were delineated using SWFWMD topographic aerials, aerials from the Pinellas County Property Appraiser's office, SWFWMD and Pinellas County Geographic Information System (GIS) data files, the Pinellas County Master Drainage Plan (Pinellas County, 1981), field data and reconnaissance, and other data as noted. **A** total of 214 subbasins were identified. Subbasins represent the smallest spatial unit delineated considering land use, drainage infrastructure, and topography. These subbasins were aggregated into twelve (12) basins, which were delineated by encompassing all subbasins contributing flow to the same major drainage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system with a single outfall to the lake. In areas where there was no major drainage system with a common outfall, basins were constructed of adjoining subbasins that discharged directly into Lake Seminole in the same vicinity (Figure 2).

Existing land use within the Lake Seminole Watershed consists primarily of developed urban land. A large percentage of the watershed is residential, with little undeveloped land. However, small areas of agricultural, public, recreational, industrial, and other land uses are scattered throughout the watershed. In addition, several conservation and preservation areas are located around or in close proximity to the lake. Because the watershed is nearly built out, the variety of land uses within the watershed is consistent for both existing and future conditions. Land use coverage within the watershed for existing conditions was obtained through the Pinellas County Geographic Information System (GIS) data base from the Property Appraiser's office. These data were aggregated into one of the eighteen land use categories used to develop model input parameters. In addition, water and road coverage by subbasin were tabulated from the GIS data, although no formal category was assigned to these coverages by Pinellas County. Land use coverage within the watershed for future conditions was obtained through the Pinellas County Geographic Information System (GIS) data base from the Planning Department.

Soil types within the limits of the watershed were determined using the USDA Soil Conservation Service (SCS) Soil Survey of Pinellas County (USDA, 1972). Each of the four soil hydrological groups (HSG) "A", "B", "C", and "D" are represented in the various soil types. HSG "A" typically generates the least runoff per unit rainfall and is often associated with soils having a high sand content and low water table. HSG "D" soils generate the most runoff per unit rainfall and are often associated with soils with higher organic content and a high water table. The limits of each soil hydrologic group as reflected by soil type was determined by subbasin.

RUNOFF can also generate pollutant loads based on watershed characteristics, and this procedure was used during the water quality simulations. The "Rating Curve Method" was used for water quality simulations, which uses a single Event Mean Concentration (EMC) for pollutants. This results in pollutant concentrations which do not vary with flow. Initial EMC values were based on literature values by land use type, Final values were determined from the results of the calibration of the water quality portions of the LSMM.

## EXTRAN Block Input Parameters

The water quantity portion of the LSMM included EXTRAN blocks for eight of the 12 basins, and the lake itself to route runoff. Each separate routine read an interface file generated by the RUNOFF block for each storm event simulation. Each interface file contained the simulated runoff data for each subbasin and design storm by load point. EXTRAN was used to route the runoff through the simulated drainage network. Lake Seminole itself was also modeled using EXTRAN in order to determine floodplain boundaries around the lake. The four basins (4, 10, 11, and 12) that were not modeled using EXTRAN either lacked a well-defined conveyance system, had a direct discharge to Lake Seminole, or were too small to be appropriate for proper EXTRAN routing. Surface water runoff from these basins was simulated within the RUNOFF block and discharged directly to Lake Seminole. Only the major conveyance features within each of the basins selected for routing were modeled using the EXTRAN block. Extreme upper reaches within the basins may contain smaller closed conduit, open channel, and pond systems which were not coded into the EXTRAN block input, **As** stated above, the major drainage systems were followed upstream into the basins until pipe sizes of less **than** 24" in diameter, or equivalent were encountered,

Floodplain geometry, storage element and preliminary drainage network information was taken from SWFWMD contour maps, FDOT design plans, and previous studies performed within the watershed. Additional field survey data of major channel reaches, pond outfall structures, and other drainage structures and invert elevations within the drainage network were collected by PBS&J from August through October of 1997. These data provided the basis for the detailed drainage system characterization of the Lake Seminole Watershed. Survey data were collected from the Lake Seminole outfall of each basin upstream to the cutoff point. The cutoff point was defined by the County as the upstream limit of pipes having a 24-inch or greater diameter and was upstream extent of the EXTRAN Block domain,

## EPA SWMM Model Constraints

EPA distributes the public domain version of SWMM version **4.30,** which has a limit on the maximum number of nodes, reaches, storage junctions, weirs, and several other parameters which may be modeled in a single simulation. This dictated that separate models be compiled for each of the basins, and the lake itself. Since the Lake Seminole model was not dynamically linked to the basin models, an iterative approach was utilized to determine the 100-year & 25-year/24-hour flood stages within the lake. RUNOFF and EXTRAN block output files generated by preliminary runs were reviewed, and flows generated by each of the basin models were determined for each design storm simulation. **A** spreadsheet was then used to combine all twelve (12) separate hydrographs from the output file for each basin into a single combined time series of flows for the 100 & 25-year/24-hour design storm simulations. Discharge into the lake resulting from rainfall excess over all basins was obtained from the EXTRAN output files for those listed above, or RUNOFF block output files for the non-routed basins. These individual hydrographs were summed, and entered in EXTRAN as a user-input hydrograph. This hydrograph was then combined by EXTRAN with the hydrograph contained on the RUNOFF block interface file calculated by each of the design storm simulations for the lake. **A** user-input hydrograph option then allowed this single time series of flows to be coded directly into the EXTRAN data input file (K3 cards) for the 100-year and 25-year 24-hour design storm runs for Lake Seminole. EXTRAN automatically combined these user-input hydrographs with the hydrograph contained on the interface file calculated by each of the design storm RUNOFF block simulations for the lake.

# RESULTS

Calibration for the SWMM surface water quantity and quality models was accomplished using rainfall volumes and distributions measured during three storm events at five separate storm water monitoring stations established within basins 1, 2, 3, **6, and 7** of the Lake Seminole Watershed during 1997. Model results were then compared to the measured runoff responses. RUNOFF block input parameters including basin width, DCIA, impervious "n" and pervious "n", were then adjusted to bring measured and modeled runoff peak stages and flow rates into closer agreement. Once the water quantity portions of the LSMM were calibrated, pollutant loading input parameters were adjusted to more closely align the simulated loadings to match laboratory results from storm water samples.

At each of the 5 monitoring stations, rainfall, runoff stage, and velocity data were collected through manual measurements and/or automated storm water sampling equipment. All monitoring stations were established in locations which would sample a significant portion of each basin, while remaining up-stream of any tailwater influences. In addition, storm water samples were collected at each monitoring station for use in water quality calibration simulations.

Three storm events recorded in late 1997 were selected for model calibration and verification purposes. Selection criteria required the following storm characteristics: 1) **an** isolated event, 2) a typical rainfall distribution, and **3)** a typical hydrograph shape **and** runoff response. Rainfall hyetographs were developed from digital data recorded during each storm. Incremental values of rainfall were used to generate the storm record, and were input into the SWMM runoff blocks for each of the five basins containing the monitoring stations. **A** September 27, 1997 storm totaled approximately **3.8**1 cm (1.5 in) of rainfall over a two-hour period. Although this was a relatively large storm, which was relatively evenly distributed over the watershed and met the above three criteria, An October 17, 1997 storm totaled approximately 1.27 cm (0.5 in) of rainfall over a two-hour period. Although this was not a large storm, it was relatively evenly distributed over the watershed and met the above three criteria. A November 29,1997 storm event totaled approximately .89 cm (0.35 in) of rainfall over a two-hour period. Although this was another relatively small storm, which displayed some variation in total rainfall distribution over the watershed, it met the remaining above criteria,

Simulated stage and flow data at the conduits and junctions corresponding to monitoring station locations in the SWMM model were then computed. Subbasin parameters in the **SWMM** RUNOFF Block were adjusted in **an** iterative process to achieve the best fit between the recorded and simulated stage and flow data. Because the majority of the parameters that were initially used in the RUNOFF Block were based on default or standard values, the parameters used in the modeling do not always reflect the conditions within each subbasin. Adjustments made during calibration were relatively minimal, with the result of the model closely approximating the measured data. Calibration runs utilized both the EXTRAN and RUNOFF Blocks of SWMM.

Modeled estimates of peak stage and flow for the five sampled basins were compared to measured values for the three storms. Deviations from maximum measured depths at the five calibration stations averaged approximately -5%, approximately -8% for peak flow, and approximately **4%** for total flow. Modeled estimates of pollutant loads were then compared to laboratory results of storm water samples collected during the three storms. Following several iterations, the average deviations from measured EMC values for Total Phosphorous, Total Nitrogen, and Total Suspended Solids were also reduced as much as possible. Using these calibrated RUNOFF blocks, design storm event simulations were then performed with the water quantity portion of the LSMM to determine the extent of floodplains within the watershed, and year-long continuous simulations were performed with the water quality portion of the LSMM to determine annual pollutant loadings to Lake Seminole.

## DISCUSSION AND CONCLUSIONS

Following the successful calibration of the revised LSMM, design storm simulations were run for existing and future land use conditions for the 25-year 24-hour, and 100-year 24-hour design storms. Peak flood elevations predicted by EXTRAN were then used to identify areas of existing or potential flooding problems within the Lake Seminole Watershed. Potential flooding problem areas were also identified by running the calibrated model for future conditions. For both existing and future conditions, flooding was primarily restricted to minor street flooding, pond and lake overtopping and junction surcharging. No major flood problems were identified and simulated 25-year and 100-year floodplains were virtually unchanged for the existing and future land use conditions.

Non-point source pollutant loading estimates from the watershed were ranked by constituent and basin to determine the most significant loads to Lake Seminole during the three separate year-long water quality continuous simulations. Land use within the watershed is predominantly urbanized, with a relatively homogeneous mix of land uses. Not surprisingly, therefore, the largest TP, TN, TSS and BOD loads came from the basins with the largest areas, and were ranked in the same order. This may indicate that the best locations for BMP's and/or storm water retrofit projects are at locations near the main drainage network which covey runoff from the largest upstream areas possible. An analysis of such locations will be performed upon completion of the water quality model, which includes the **WASP** water body model.

Summarized below are several of the many lessons encountered so far during the completion of this study using the above described approaches:

- Since SWMM 3.0 was to be used as it is available directly from the **EPA** for this project, maximum model array sizes for model parameters such as number of basins, noted and junctions prevented the entire watershed and lake to be modeled as one. Individual basins were therefore run separately, and only adjacent basins with intermingled flood flows were modeled together. In order to obtain tailwater elevations for each of the modeled drainage networks dumping into Lake Seminole, as well as a floodplain elevation of the lake itself, all flows from the basin watershed models were summed in a spreadsheet, This hydrograph was then entered into an EXTRAN model of the lake as a user-input hydrograph as described above. Since this process required two separate runs with an intermediate spreadsheet summation step during each step, substantial effort was required to generate a complete a water quantity simulation for a given storm event. Limiting model detail or using a modified version of the SWMM code could have prevented this effort required during the water quantity simulations.

- Land use input parameters developed for the RUNOFF block water quantity simulations were based on Pinellas County classifications, and totaled 18. Separate basin input parameters were assigned to each land use category, and ultimately used to calibrate the water quantity simulations by bringing predicted calibration hydrographs into agreement with hydrographs recorded during the three calibration storm events. Although more land uses provided a greater number of possible model input parameter variations, any gains achieved in obtaining in a more accurate and precise calibration ended up costing a price in terms of effort keeping track of 18 land uses during manipulation of the input data set and QC, and this should be considered.

- Three calibration storms were recorded, during which both water quantity and quality data were collected for calibration of both of these aspects of the LSMM. Field **work** involved in collecting data for these calibration events was focused on recording both water quantity and quality data during the same events. Since water quality parameters included grab samples which must be collected during the rising limb of the hydrograph, many events were not monitored where these water quality samples were not collected at the front end. In retrospect, more focus could have been placed on obtaining good stage-discharge relationships at each of the monitoring stations as a first priority, prior to attempting to obtain good water quality samples.

## ACKNOWLEDGMENTS

Policy **3.1.4** of the Conservation Element within the Pinellas County Comprehensive Plan calls for the systematic development of watershed and water body-specific management plans for all major drainage basins in the County. Pursuant to that policy, in June of 1992 a preliminary diagnostic feasibility study was prepared for the County and the Southwest Florida Water Management District (SWFWMD, 1992).

Since **1992,** Pinellas County has sponsored the Lake Seminole Advisory Committee (LSAC), which meets periodically to provide input from local governments, citizens, and businesses owners for management of the lake. As part of the County's on-going work to develop Comprehensive management plans for all significant basins within their jurisdiction, the County authorized Coastal Environmental, Inc. (now PBS&J, Inc.) in late **1996** to assist in the preparation of the Lake Seminole Watershed Management Plan (LSWMP). The LSWMP is to be a comprehensive guide to managing the lake, and will include provisions for habitat protection and enhancement, water quality and flood protection and improvement, recreational opportunities, and aesthetic enhancement.

The authors would like to like to *thank* David Talhouk, and the entire Pinellas County Engineering Staff, as well as the SWFWMD for the opportunity to participate in development of the LSWMP.

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# WATER CONSERVATION WITH SUBSURFACE DRIP IRRIGATION FOR POTATO PRODUCTION

by

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## ABSTRACT

An automatically-controlled subsurface drip (SDI) irrigation system was compared to conventional semi-closed seepage irrigation (subirrigation) for potato (Solanum tuberosum L.) production in a 3-year field research project. Both the SDI irrigation system and the automated irrigation control system performed well and produced crop yields that exceeded the industry average each year. The field water table responded more quickly to irrigation with SDI, and the water table was more accurately controlled at the desired level. Potato yields were not statistically different with the two irrigation systems, although 36% less irrigation water was applied with SDI, despite water requirements for filtration and flushing to prevent SDI emitter plugging.

Continuous injection of a commercially-available irrigation line cleaner prevented the buried irrigation emitters from plugging throughout the crop season. Reductions in emitter flow rates occurred whenever irrigation was interrupted for extended periods of time, however, flow rates recovered within days of the resumption of regular irrigations and chemical water treatment. Energy required for irrigation pumping was about 70% higher with SDI despite smaller water applications because the operating pressure was much higher with SDI. We estimated the cost to convert an existing seepage system to SDI as $990 per ha.

## INTRODUCTION

Potatoes are an important Florida agricultural crop. The 1990-94 cropped area and yield statistics were part of the justification for this research. The 1990-94 average cropped area was 20,200 ha (50,000 ac), with an average yield of 23 Mg ha$^{-1}$ (205 cwt ac$^{-1}$) and an annual value of $128 million (Florida Agricultural Statistics Service, 1994). Approximately 65% of this crop was produced in the St. Johns River Water Management District (SJRWMD) tri-county area of St. Johns, Putnam and Flagler counties where this research was conducted.

From the SJRWMD Benchmark Farm irrigation requirements monitoring program, the average potato irrigation requirement was estimated to be 457 to 508 mm $yr^{-1}$ (18 to 20 inch $yr^{-1}$) (Vince Singleton, SJRWMD, personal communication). Based on these areas and irrigation requirements, this industry applies 93 to 103 million $m^3$ (90,000 to 100,000 acre-in) of irrigation water annually.

Figure 1. Water flows in ditches and water furrows with conventional seepage irrigation.

Most Florida potatoes arc irrigated using seepage irrigation (subirrigation) systems. With seepage irrigation the field water table is controlled at a depth just below the plant root zone so that water is supplied to plant roots by capillarity. Thus this irrigation method is based on water table control. Currently 380,000 ha (940,000 ac) of commercial agricultural crops are seepage- irrigated in Florida (Smajstrla et al., 1999). This irrigation method is extensively used bccausc it is low-cost and effective in locations where water shallow water tables can readily bc established and water supplies are plentiful.

In general, seepage is not as efficient as other irrigationmethods (Smajstrla et al., 1991)because water in excess of that used by crops is required to raise the field water table, and because water is lost to drainage and runoff from high water tables in the field. Several researchers have studied methods ofreducing potato irrigation requirements by improving the efficiency of seepage irrigation systems in Florida. Campbell et al. (1975) and Rogers et al. (1975) used corrugated subsurface PE tubing for both irrigation and drainage. This greatly improved seepage irrigation efficiencies, however, plugging of the drain tubes by iron ochrc and other materials rcduced their effectiveness and prevented the general adoption of this method. Smajstrla et al. (1984) automatically controlled seepage irrigation water applications using field float switches. They reported an 8% increase in irrigation efficiency with the controlled water applications, but runoff losses still occurred.

Figure 2. Little or no water flows in water furrows when the water table is controlled with SDI.

Conventional semi-closed seepage systems use shallow open field ditches (water furrows) to distribute the irrigation watcr. Clark et al. (1990) proposed the use of subsurface drip (SDI) pipelines and emitters to apply irrigation water directly into the plant beds in sufficiently large quantities to establish and maintain shallow water tables. They estimated that costs would be much less than with conventional drip irrigation systems where laterals are required for each plant row because SDI laterals could be widely spaced and permanently installed. **Also,** irrigation requirements would be expected to be reduced due to the direct water application into the plant beds as compared to conventional open-ditch seepage systems. Stanley and Clark (1991) reported that irrigation requirements for tomato production in south Florida were reduced 33% to 40% with **SDI** as compared to conventional semi-closed seepage irrigation systems due to reduced runoff rates.

Because no studies of subsurface drip irrigation of potatoes had previously been conducted in Florida, the primary objective of this research was to evaluate the use of an automatically-controlled subsurface drip (SDI) system to conserve water during irrigation of potatoes in the Hastings, FL area. Specific research objectives were to directly compare automatically-controlled **SDI** and semi-closed seepage irrigation systems (1) to evaluate the performance of the **SDI** irrigation system and the automatic irrigation control system, (2) to measure irrigation water use with both systems, (3) to measure potato yield with both types of irrigation systems, **(4)** to evaluate maintenance needs and hydraulic performance of the **SDI** system, and *(5)* to estimate the cost to convert from conventional seepage to **SDI** irrigation.

## MATERIALS AND METHODS

Direct field-scale comparisons of **SDI** and conventional semi-closed seepage irrigation systems were made. Field research plots were installed at the University of Florida Hastings Agricultural Research Center Yelvington Farm, **and** potatoes were produced for three crop years during the 1995-1997 spring growing seasons. Production practices typical of the industry were used as much as possible. Field scale research plots 18 m (60 ft) wide and **183** m (600 ft) long were installed. To statistically separate responses to site characteristics from treatment responses, treatments were blocked and replicated **3** times. Buffer plots of the same size were installed between seepage and SDI plots, requiring a total land area of **4** ha (10 ac). The soil type was Ellzey fine sand (Arenic Ochraqualfs), a typical high water table soil in that area.

The subsurface drip irrigation system consisted of microirrigation tubing (Netafim) with **4-L** $hr^{-1}$ (1 g $hr^{-1}$ emitters (Triton) spaced 1.2-m **(4** ft) apart. Laterals were spaced ***6*** m **(20** ft) apart and extended the 183-m (600 ft) length of the beds. Laterals were buried by chiseling them to a depth of 0.5 m (20 in). Each **group** of three laterals per bed was connected with a polyethylene manifold pipeline at each end of the field. At the inlet end water was supplied from a continuously pressurized main pipeline to each manifold through an automatic solenoid valve, pressure regulator, flow meter, and vacuum breaker. At the downstream end, an automatic flush valve, manual ball valve, and flow meter were installed to automatically flush the lateral pipelines at each irrigation, permit manual flushing and inspections, and to monitor the volume of flush water, respectively. The operating pressure was set at the inlet of each treatment plot using a 207 kPa (30 psi) pressure regulator. This produced an application rate of **0.44** L $s^{-1}$ **(7** gpm) per treatment plot or 11 mm $d^{-1}$ (0.45 inch $d^{-1}$).

Irrigation of each of the **six** treatment plots was independently controlled with a float-actuated mercury tilt switch located in a shallow well 18 m (60 ft) from the upper end of each plot. Float switches were initially set to schedule irrigations to maintain the field water table 50 cm (20 in) below the top of the plant row. When the water table dropped below 50 cm (20 in), the mercury switch closed, providing power to the solenoid valve for irrigation. When the water table rose above 50 cm (20 in), the microswitch opened, and irrigation was stopped. Buffer areas were irrigated at the same time and at the same rate as the adjacent treatment plot. As the plant root zones developed during the mid and latter 1/3 of the season, threshold water table levels were re-set to 55 and 58 cm (22 and 23 in).

Irrigation water was chemically treated and filtered to prevent clogging of the drip emitters. The filtration system consisted of a Y-strainer, media filters and screen filters at the well, and disk filters at each field plot. Each lateral was automatically flushed each time the irrigation system operated, while manifold and main pipelines were manually flushed each week. A commercially-available irrigation line treatment chemical ('DiSolv', Flo-Tec, Inc.) was continuously injected into the irrigation water at an average rate of about 4 mg $L^{-1}$ (4 ppm) to prevent emitter plugging by preventing chemical precipitates and biological growths.

Irrigation volumes applied were monitored with totalizing flow meters at the irrigation pump, at the inlet to each field plot, and at the flush valves. Irrigation occurrences and durations were recorded with timers, event counters, and a strip chart recorder. Data were recorded from the beginning of irrigation at plant emergence until irrigation was discontinued about one week before plant harvest each year.

Potato production field operations followed typical grower practices in the region. Most major tillage and production operations were performed by our grower-cooperator, Smith Farms, using large field-scale equipment. A cover crop of sorghum sudan was grown during the summer and early fall months each year to increase the soil organic matter. To control soil-borne plant diseases, the field was fumigated with Busan or Telone in December each year in preparation for planting. 'Atlantic' seed potatoes were planted at the rate of 2,900 Kg $ha^{-1}$ (2,600 lb $ac^{-1}$) in early February each year. At the same time Temik was applied for nematode control.

Liquid fertilizer was applied at the rate of 270-84-270 Kg $ha^{-1}$ (300-94-300 lb $ac^{-1}$) N-P-K plus micronutrients in two applications. The herbicide Sencor was applied by spraying before plant emergence. Irrigation was begun at plant emergence and continued until mid-May, about one week before harvest. Plots were sprayed as required during the growing season to control potato blight and insects. Potatoes were harvested and graded by U.S. grade standards during the last week of May each year. A total of 180 6-m (20 ft) long subplots were harvested for yield analysis each year. Ten rows were harvested at three locations (north, center, south) in each of the six treatment plots. Only grade A potatoes are reported as marketable yield in this paper.

## RESULTS AND DISCUSSION

The SDI irrigation system performed well throughout each irrigation season. The SDI field water table responded more quickly to irrigations than with the seepage system because water was applied directly into the beds with SDI within minutes of beginning irrigation, whereas it had to flow down the water furrows and then seep laterally across the beds with the seepage system. As an example, Fig. **3** shows the average field water table levels as a function of irrigation and rain for the

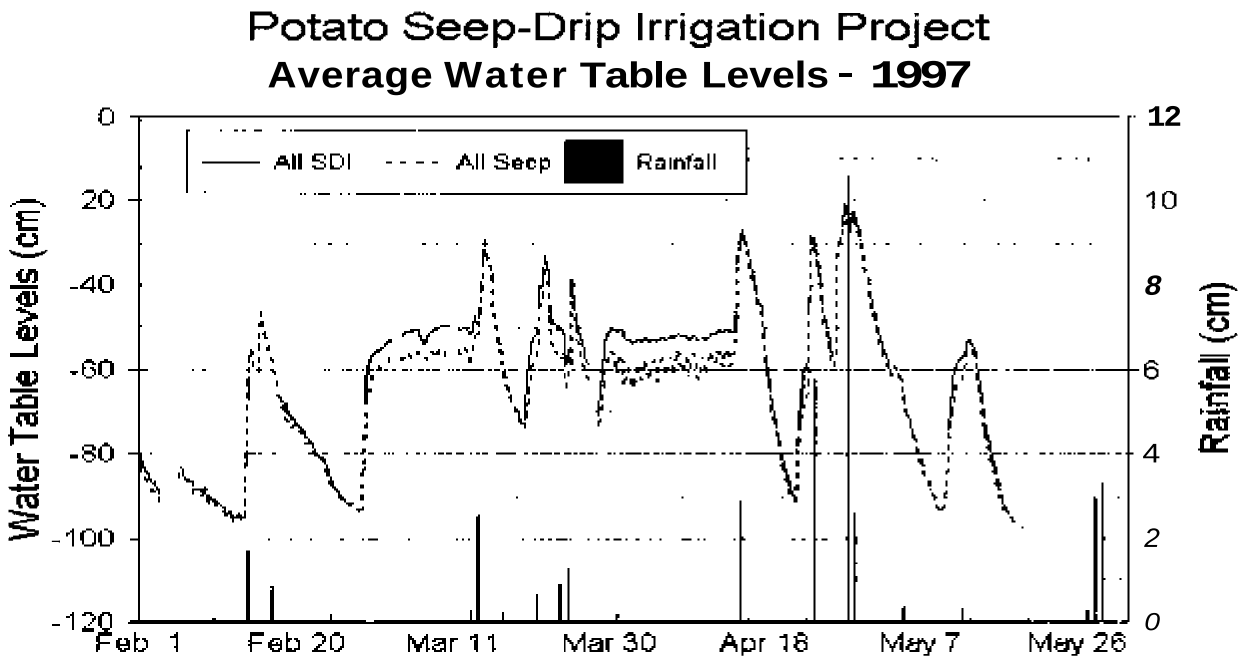


**Figure 3. Average field water table levels and rain for all SDI and seepage irrigation plots - 1997.**

**1997** growing season. With conventional seepage, the field water tables consistently lagged the SDI water tables even though float switches in all plots were set to control at the same elevation.

Irrigation requirements were significantly reduced with SDI as compared to seepage irrigation. Overall average and yearly irrigation requirements are shown in Table 1. With seepage irrigation, 27.5 to 42.6 cm **(10.8** to 16.8 in) were applied, while only **19.1** to 25.8 cm **(7.5** to **10.2** in) were applied with the SDI system. The average **SDI** application of 22.9 cm **(9.0** in) was **36** percent less than the average seepage application of 36.2 cm (14.3 in). The average seepage application was less than the industry average of **46** to 50 cm (1**8** to **20** in) because the float-controlled irrigation system we used only applied water when the field water table was below the desired level.

| Irrigation Treatment | Seasonal Irrigation Depths Applied (cm) | | | |
|---|---|---|---|---|
| | **1995** | 1996 | **1997** | *3*-year average |
| Seepage | 38.6 | 42.7 | 27.5 | 36.2 |
| SDI | 25.8 | 23.9 | 19.1 | **22.9** |
| Significance | * | * | * | * |

Monthly and seasonal total rainfall and evapotranspiration (ET) data are shown in Figs. **4** and *5*. Both individual year and long-term average (normal) values are shown. Although the seasonal rainfall was approximately equal to the normal (long-term average) value each year, distributions during the growing season were considerably different, and led to highest irrigation requirements in 1996 when little rainfall occurred during the high ET months of April and May. Irrigation applications were lowest in 1997 when rainfall was concentrated in April. Monthly ET (Fig. *5*) was not as variable as rainfall. Seasonal ET was approximately equal to the normal value in 1995, while it was below normal in both 1996 and 1997, primarily because of reduced solar radiation due to cloud cover.

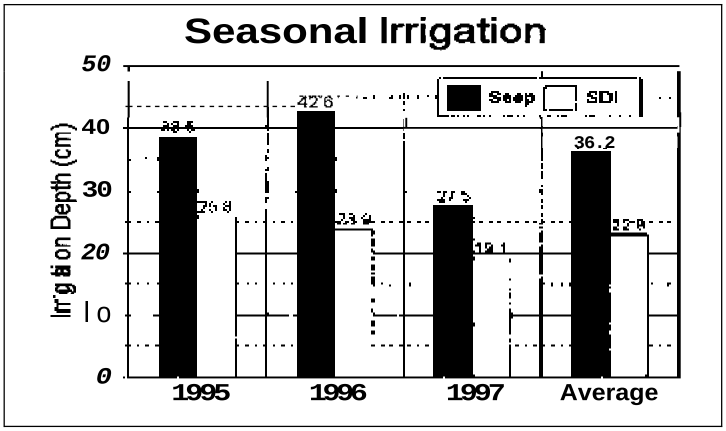


**Figure 4. Annual and 3-year average irrigation requirements with seepage and SDI irrigation systems.**

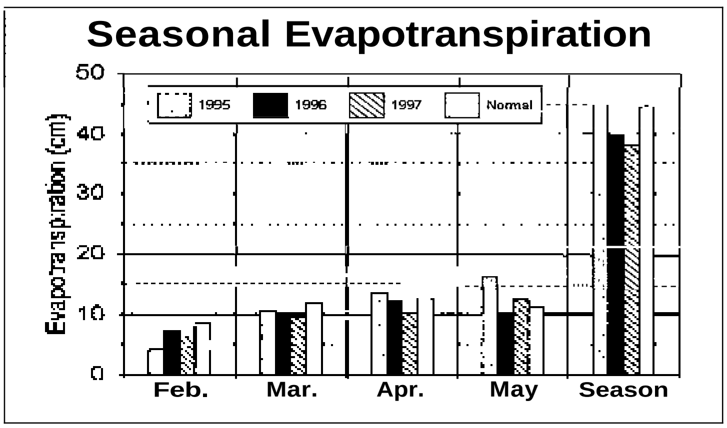


**Figure** 5. 1995-1997 and long-term **average monthly and seasonal evapotranspiration.**

The primary reason for the reduction in irrigation applications with **SDI** (Table 1) as compared to seepage was that no runoff occurred during SDI irrigation because the field water table was maintained at a level just below the bottom of the water furrows. Conversely, semi-closed seepage systems use the water furrows to distribute water within the field, and runoff occurs due to the slope of the water furrow. A small slope (typically about 0.05%) is required in the water furrows for adequate drainage of large rainstorms.

**SDI** irrigation required water to flush pipelines and filters in addition to the water applied to the crop. All filters and pipelines were manually or automatically flushed at about the same time throughout the growing season, thus the volumes of flush water were almost identical for all plots. In each case, the volume of flush water was approximately *5%* of the irrigation volume. Water was flushed directly into field drainage ditches, thus it did not help meet crop irrigation requirements.

Potato marketable yields are shown in Table 2 for all years and both irrigation systems. Yields were not statistically different in any of the three growing seasons, nor for the 3-year average. These data demonstrate that the SDI irrigation system did not restrict crop growth although **36%** less water was applied. Further, SDI treatments did not benefit from the greater uniformity of water application or the faster water table response to irrigation.

Table 2. Potato marketable yield with seepage and SDI irrigation.

| Irrigation Treatment | Potato Marketable Yield (Mg $ha^{-1}$) | | | |
|---|---|---|---|---|
| | 1995 | 1996 | 1997 | 3-year average |
| Seepage | 26.5 | 35.2 | 24.5 | 28.7 |
| SDI | 24.9 | 35.3 | 22.8 | 27.7 |
| Significance | NS | **NS** | **NS** | **NS** |

**NS = F values for comparisons were not significant at the 5% level.**

Filtration, flushing, and the continuous injection of DiSolv irrigation line cleaner chemical at an average rate of about **4** mg $L^{-1}$ (4 ppm) with the irrigation water prevented the buried irrigation emitters from plugging throughout the crop season. The **4** mg $L^{-1}$ (4 ppm) injection rate of DiSolv was higher than the **2** mg $L^{-1}$ **(2** ppm) rate recommended by the manufacturer, however, higher rates were found to be necessary because emitter flow rates declined after the system was shut off for several days. We injected a higher rate, up to 10 mg $L^{-1}$ (10 ppm) upon startup and at the beginning of the season to quickly restore emitter flow rates to design values. Emitter flow rates were often reduced by as much as 10% whenever the system was off for several days, however, the design flow rate was restored after the irrigation system operated regularly for 2-3 days.

In 1995, during the first year of this research we injected liquid chlorine at 10 mg $L^{-1}$ (10 ppm) and DiSolv as recommended by the manufacturer. Emitter flow rates in the three drip-irrigated plots decreased due to partial emitter plugging by precipitation of calcium sulfate, We determined that chlorination had caused the precipitation of sulfur, and conducted tests after the **1995** growing season that demonstrated that clogging could be prevented with DiSolv alone. We prevented plugging during the 1996 and 1997 potato growing seasons by injecting DiSolv only, however, reductions in flow rates still occurred after the system had been off for several days. In each case, emitter flow rates were quickly recovered by injecting DiSolv at higher rates of 5 to 10 mg $L^{-1}$ (5 to 10 pprn) during the first 2 to 3 days of operation.

During the fall of 1995 and 1996, we conducted extended tests of system plugging and found that the plugging was progressive with time while the system was idle. After being shut off for approximately three months, emitter flow rates dropped about 21%. This demonstrated that it is necessary to periodically operate the irrigation system during extended periods of non-use in order to inject chemicals and reduce the severity of plugging.

The energy required for irrigation pumping was about 5.9 kwh $cm^{-1}$ (1**5** kwh in-')with SDI and only 2.4 kwh $cm^{-1}$ (6 kwh $in^{-1}$)with seepage, primarily because of the higher pressure required to operate the SDI system. Thus, the energy consumption was 70% higher with SDI, despite smaller water applications with SDI. We operated the subsurface drip emitters at 172 kPa **(25** psi), which required a manifold pressure of 206 kPa (30 psi), while the seepage system required a manifold pressure of only 34 kPa ***(5*** psi).

The estimated cost to convert an existing seepage system to SDI is \$990 $ha^{-1}$ (\$400 $ac^{-1}$). This includes the cost of installing buried lateral pipelines, replacing the irrigation pump and power unit, and providing for filtration, flushing, and chemical water treatment. It assumes that existing manifold pipelines and wells are adequate for the new system.

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# WETLAND HYDROPERIOD ANALYSIS

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## ABSTRACT

Rapid urbanization can adversely impact the functional values of isolated wetlands. The hydroperiod (duration of inundation in a wetland) is one of the functional elements which must be maintained to avoid such impacts to wetlands surrounded by or adjacent to development. In the past, wetlands were typically filled in to facilitate development. Today, instead, many developments incorporate wetlands into stormwater management planning as a means to provide water quality treatment and/or attenuation. However, the hydroperiod of a wetland to be utilized in this way must be properly determined in order to avoid adverse wetland impacts, In this paper, a water budget analysis is depicted to determine the hydroperiod with special emphasis given to surface water runoff resulting from precipitation. ***An*** analytical example is included to illustrate the hydroperiod analysis.

## INTRODUCTION

Hydrology is probably the single most important determinant for the establishment and maintenance of specific types of wetlands **and** wetland processes (Mitsch & Gosselink, 1993). Land use changes and stormwater management practices usually alter hydrology within **a** watershed (Azous & Horner, 1997). Within the last decade or so of rapid urbanization, the stormwater management function of natural wetlands has been recognized by those employed in land development. As a result, rather than being destroyed and replaced by mitigation projects, wetlands are being incorporated more and more into developments' stormwater management systems for water quality treatment and attenuation purposes. Given man's inability thus far to recreate nature, this may be viewed as a godsend. However, some preliminary information must be gathered prior to project design.

The objective of this paper is to evaluate the hydrological and biological functions of a wetland by considering the pre-development and post-development conditions within a wetland watershed.

### Hydrologic Characteristics of Wetlands

The hydrological regimen is what distinguishes wetlands from aquatic and terrestrial systems. This characteristic creates the physicochemical conditions that make such an ecosystem unique.

Hydrology modifies or determines the structure and functioning of wetlands by controlling the composition of the plant community and thereby the animal community.

For the purpose of this paper, palustrine wetlands will be used. According to Cowardin et al. (1979) there are eight classes of palustrine wetlands, all nontidal (isolated, freshwater). In addition to physical shape and form, major factors that influence the hydrology of palustrine wetlands are precipitation, surface water inflows and outflows, groundwater exchange and evapotranspiration. These components will be further discussed under the water budget section.

Among the hydrological characteristics of wetlands described by Duever (1988), are flood hydrographs, water level fluctuations and hydroperiods.

## Flood Hydrograph

**A** typical hydrograph is a graph or table showing the flow rate as a function of time at a given storm event in a watershed. The hydrograph is the result **of** physiographic aspects and meteorological occurrences in the watershed. Since wetlands are one of the physical characteristics of the watershed, the wetlands influence the response of the watershed runoff for a given storm event. The actual shape and scale of a hydrograph can vary substantially depending upon physical characteristics such as slopes, vegetation coverage and ecosystem type within a watershed. There are two types of hydrographs. The first one relates discharge to time and is called a discharge hydrograph and the second relates stage to time and are called a stage hydrograph.

## Water Level Fluctuations

The fluctuations of the water level in **a** wetland are influenced by water inflows and outflows related to the meteorological conditions of the area. Another factor to consider that will cause different ranges in the fluctuation of the water levels is the location of wetlands within higher or lower areas of the watershed. Components which alter such fluctuations are the surface and groundwater inflows attributed to precipitation. However, the main control factor is the rise and fall of the groundwater table which is influenced by other surrounding topographic land features, soil type and vegetation cover.

## Hydroperiod

Wetland hydrology may be considered in the context of the hydroperiod, defined as "the seasonal occurrence of flooding and/or soil saturation, encompassing the depth, frequency, duration, and seasonal pattern of inundation" (Azous & Homer, 1997). Wetland type varies according to frequency of inundation, which may be annual, seasonal, or in some cases a daily occurrence. In addition, the water table at times may be so low that there is no apparent soil saturation or flooding (Figure 1).

Wetlands receive water from any combination of the following: precipitation, surface water and/or groundwater. These in turn influence water depth. The duration of soil saturation determines a wetland's hydroperiod.

To determine the existing hydroperiod of a wetland to be incorporated into a stormwater management system, specific hydrological characteristics and biological indicators of the wetland must be identified or field verified. The pre-development wetland watershed must be mapped and quantified so that there is known contributing acreage. The projected post-development wetland watershed must also be mapped and quantified to determine any expected changes in contributing acreage. In addition, existing normal pool (NP) and seasonal high water elevations (SHWL) of the wetland must be identified, the vegetative community described and a wetland assessment performed.

## Water Budget

It is important to understand the hydrology of a wetland system because it of its influence on chemical and biological dynamics of the wetland. For example, a significant variation especially the deficit of water associated with the hydroperiod of the wetland during the dry and/or wet seasons can result in biological changes. **A** major difficulty in managing wetland systems is the inability to distinguish shifts in the hydrological conditions resulting from human activities versus those caused by natural phenomena.

To understand the hydrological process based on the principles of conservation of mass and the continuity equation, the water budget reflects the net effect of all the processes that influence the hydroperiod of wetlands. The water budget for a wetland **can** be expressed as:

$$AS = P + SSI + SI - PR - SSO - SO - ET$$

where

| | | |
|---|---|---|
| AS | = | change in storage volume (surface and soil); |
| P | – | precipitation; |
| SSI | = | subsurface inflow (groundwater inflow); |
| SI | = | surface inflow (overland flow); |
| PR | = | percolation; |
| SSO | = | subsurface outflow (groundwater outflow); |
| SO | = | surface outflow (overland outflow); and |
| ET | = | evapotranspiration. |

In the above equation, all the parameters represent units of depth. These parameters can either be measured or analytically calculated based on information collected at a specific site. The above components of the water budget vary significantly depending upon local topography, hydrology of the site, and wetland type.

## Precipitation

Precipitation inputs to wetlands may exhibit extreme spatial variability, even over small areas during a single storm event. This variability has been synthesized and available in data sets appropriate near or within a wetland and its watershed.

For example, within the Tampa Bay area, average rainfall is 53 inches per year, much of this from June to October (the rainy season). Seasonal variation of rainfall is shown in Figure **2.**

## Subsurface Inflow-Oufflow

The subsurface (groundwater) inflow-outflow beneath a vegetation canopy may differ significantly from adjacent areas without a canopy. Interception of precipitation fiom foliage and vegetated surfaces and the re-evaporation of water can significantly reduce the amount of water reaching the water table,

In the Tampa Bay area, during the rainy season, the water table varies from zero to **2** feet below the existing ground surface, and during the dry season, the water table falls to as much as six to **8** feet below the surface,

## Percolation

Gradual percolation causes a regulating effect on wetlands and its hydroperiod. Note that the percolation rate at the wetland bed would be very low because of low hydraulic conductivity due to the relatively impermeable soil characteristics underlying a wetland as shown in Figure **3** (Eggelsmann, 1972).

## Surface Flow-Inflow-Outflow

In general, surface water movement in a wetland is the result of precipitation, surface water inflow and outflow, and losses through seepage, transpiration, and evaporation.

An important wetland characteristic is extended shallow water inundation **-** extended but not prolonged or permanent. Factors such as orientation, surrounding soil characteristics, storm characteristics, adjacent land use patterns, and man-made alterations (such as land use changes) affect wetland hydrology. During periods of high water levels, large inflows may enter a wetland, but quickly dissipate as outflows. Even several such large flood events occurring within a relatively short time **span** may substantially raise annual inputs, but have little significant impact on the hydrology of a wetland. However, these occasional peak flows are important to topographically isolated wetlands, which receive the majority of their inflows during storm events.

The water storage capacity of wetlands is intermediate between upland areas and aquatic systems. In a flood event, the runoff rate drastically increases when water levels exceed a system's normal barriers to flow. In other words, the rate of the water level **rises** and falls quickly as the runoff rates approximate the inputs, This phenomenon leads to a fairly constant year to year maximum water levels in a wetland system (Daniel, 1981).

For the Tampa Bay area, approximately 14 inches of rainfall is generated in runoff annually.

## Evapotranspiration

Evapotranspiration is the combined process of evaporation from vegetation, land, water surface and transpiration by plants, Evapotranspiration for a given wetland depends on its microclimate (relative humidity, air and water temperature, wind velocity and its duration), the soil moisture content and the type and density of the vegetation. Compared to those of other ecosystems, wetlands have among the highest evapotranspiration rates.

Evapotranspiration rates for wetlands can be measured and/or calculated by a variety of techniques. Theoretical rates are established based on regional climatic data or site specific micro climatic data.

For the Tampa Bay area, annual evapotranspiration accounts for a loss of approximately 38 inches. Average seasonal evapotranspiration data are shown in Figure 2.

## Stormwater Systems

Developers today face many pressures including state, local and regional regulations and above all the financial interest from shareholders, Land use policies specify what percentage of developable land needs to be set aside for other "non-income producing" usage. Stormwater management is one such use. Existing depressions in the land, or wetlands, are "natural" stormwater facilities, ideal locations for stormwater storage. Today, development plans increasingly incorporate wetlands into stormwater management systems to provide storage, water quality improvement and environmental enhancement.

The impact of quantity and quality of stormwater runoff on wetland processes has raised some concerns among researchers. Quantity of stormwater runoff is a driving force in the establishment and maintenance of wetlands. In fact, assuming adequate quality, and at the correct frequency, depth and duration, stormwater runoff maintains and may even upgrade the quality of wetlands previously altered,

## Attenuation (pre- and post-development runoff rate and volume)

Variation in water level in wetlands for a typical storm under both pre- and post- development conditions can be determined by using any hydrological routing program such as EPA-SWMM, HEC-HMS or HEC-1, TR-20, etc. Water levels under pre-development conditions can be established based on biological indicators or determined by a monitoring program. Under post-development conditions, water levels will rise rapidly during and after storm events but would quickly return to its operating level (pre-development level). The quick return to this operating level would be controlled by the outflow at the outlet control structure to restore the storage capacity of the wetland.

Stormwater runoff could prove to be detrimental to the wetland by causing rapid water level fluctuations and duration periods, thus altering the wetland's hydroperiod. Plant diversity, for example, is likely to be reduced if wetland hydrology is altered in this manner. Therefore, fluctuations in a wetland should be maintained at pre-development levels, Measures should be taken to protect the integrity of a wetland during and after development. Among these should be structural and non-structural works which may include but not be limited to; sedimentation vault, erosion control, vegetation management, etc. Equally important but fewer frequently recognized, adjacent, upland buffer zones must be maintained in their natural states.

## Water Quality

Urbanization and urban activities are a source of pollution in stormwater runoff. Pollutants can be removed by wetlands through a combination of: 1) incorporation into or attachment to wetland sediments or biota; 2) degradation; or 3) export to the atmosphere or groundwater. Both physical and chemical pollutant removal mechanisms occur in wetlands. These mechanisms include: sedimentation, absorption, precipitation and dissolution, filtration, biochemical interactions, infiltration, etc. These interactive mechanisms vary from wetland to wetland; therefore, the pollutant removal efficiencies also vary from wetland to wetland (Table 1).

## Guidelines

Local, state and regional governmental agencies consider "wetlands" as: lands that are seasonally or permanently covered by shallow water, as well as lands where the water level is close to or at the surface. Whatever the case may be the presence of abundant water has caused the formation of hydric soil and has favored the dominance of either hydrophytic or water-tolerant plants.

In circumstances in which it is impossible to eliminate impacts from development, affected wetlands should be incorporated into stormwater management systems or as "natural facility." enhancements.

For wetlands incorporated into stormwater management systems, government agencies, including the Southwest Florida Water Management District (SWFWMD) require pre-treatment of storm water runoff prior to discharge to a wetland. The SWFWMD (1996) allows isolated wetlands to be included in surface water management systems when it can be demonstrated that the system design will not adversely impact those wetlands. The SWFWMD requires a pre-treatment of one-fourth inch of runoff prior to release to the wetland. The SWFWMD also states that the depth, duration of frequency of inundation through changing the rate or method of discharge of water to the wetlands must be addressed to prevent adverse impacts to the functions that wetlands provide to fish and wildlife species.

The following recommendations should be considered when incorporating wetlands into designs for stormwater management facilities in new land development projects:

- Maximize natural water storage and infiltration outside of existing wetlands.
- Establish and maintain vegetative buffers in the riparian zone surrounding wetlands.
- Acquire specific management measures to avoid general urban impacts to wetlands.
- Support management of runoff water quantity by performing a hydrological assessment to estimate elements of hydroperiod and hydrodynamics under existing pre-development and anticipated post-development conditions based on the mean annual storm event.

- Manage water quality (attempt to match pre-development water quality conditions by considering both source control BMP's and treatment BMP's) by providing a water quality control facility consisting of one or more treatment BMP's (i.e., pre-treatment sediment sump to control suspended sediment, skimmer/baffle to control oil and grease, overland sheet flow length with swale, if any, etc.).
- Establish plans to protect specific biological communities.

To determine the existing and future hydroperiod, a hydrological assessment (routing programs) should be used to determine the water level fluctuation due to storm event(s) prescribed by the regulations.

Water Level Fluctuation = Crest stage - Seasonal High Water Level

To maintain the hydroperiod and hydrodynamics of a wetland, and to avoid adverse impacts to its biological and hydrological functions, water level fluctuation over time should not vary significantly. If the analysis described above predicts excessive water fluctuations, stormwater management strategies should be employed to keep fluctuations within an acceptable range. Some guidelines suggest that the duration of stage excursions above the pre-development stage should not exceed **24** hours in any event in any year **(Azous** & Homer, **1997).**

**Hypothetical Example**

The analytical example given shows the hydoperiod assessment of an isolated wetland. The following parameters are considered:

Pre-development conditions:

| | |
|---|---|
| 1) Isolated wetland area | = 0.9 ha at SHWL (2.0 acres) |
| 2) Watershed area of wetland | = 12.14 ha (30.0 acres) |
| 3) Composite curve number, CN | = 80 |
| 4) Seasonal High Water elevation | = 7.32 m (24.0' msl) |
| 5) Normal Pool elevation | = 7.16 m (23.5' msl) |
| 6) Time of concentration | = **66** minutes |

Post-development conditions:

| | |
|---|---|
| 1) Watershed area of wetland | = 9.71 ha (24 acres) |
| 2) Composite curve number | = 88.2 |
| **3)** Time of concentration | = 18 minutes |
| 4) Lake area | = 0.4 ha (1.0 acres) @ elevation 7.32 m (24.0' M.S.L.) |

The Palustrine/ Emergent wetland consists of three distinctive vegetative zones. The outer **zone** is dominated by St. John's **wort** ***(Hypericum fasciculatum).*** **A** middle zone is dominated by maidencane (***Panicum hemitomon)*** and a core zone of pickerelweed (*Pontederia cordata*). The wetland is bordered by an abrupt border of saw palmetto (*Serenoa repens*).

Several biological indicators were identified in the field to determine the SHWL and NP of the wetland. The adventitious rooting of ***H. fasciculatum*** and the ground elevation at the jurisdictional line were compared and a SHWL of 7.32 m (24.0' M.S.L.) was determined. The normal pool was determined at 7.16 m **(23.5' M.S.L.)** by comparing *H.* ***fasciculatum*** indicators with the ground elevation at the apparent change of zonation where *P. hemitomon* begins to dominate. This wetland has minimal impacts and provides significant functions and values.

## DISCUSSION

**As** indicated by the example, based on the mean annual storm event (**2.33** year - **24** hour storm), the wetland water level fluctuates from a seasonal high water elevation **(SHWL)** of **7.32** m to 7.4 m (+/-) (24.0 to 24.3 feet (+/-)) at hour eight to approximately hour **40** (i.e., it takes approximately 32 hours to return to the pre-development seasonal high water level). While in the post-development condition it takes about 50 hours to return to the pre-development level (i.e., there is approximately 18 hours longer inundation time).

During a flood storm event (25 year - 24 hour storm), the wetland water level fluctuates from 7.32 m to **7.5** m (+/-) (24.00 to 24.6 feet (+/-)) and takes approximately **35** hours to return to the pre-development SHWL elevation. While in the post-development conditions it takes about 50 hours (i.e., there is an approximate 15 hour longer inundation time).

Since the wetland will be used for the treatment and attenuation of runoff, a pre-treatment lake has been proposed. The pre-treatment lake provides removal of sediment, oils and greases prior to discharge to the wetland. To prevent oils and greases, a structure would be set at the seasonal high water elevation with a skimmer which will function as a positive/negative flow from and to the wetland from the lake. The top of the skimmer and berm elevation around wetland were considered as the routed post-development design high water level for the 25 year - 24 hour storm event.

In both storm events, the stage excursion for the wetland was under the **24** hour guideline proposed by Azous and Horner (1997). Using the proposed guideline and limited literature available concerning the tolerance of emergent vegetative species from prolonged and/or frequent inundations, the example suggests that no adverse wetland impacts would occur; however, it is strongly recommended that each wetland hydroperiod be analyzed, **as** in the example, on a case by case basis. If the proposed design exceeds the range of the pre-development staging, and adverse wetland impacts are anticipated, a stormwater management design modification or a monitoring plan for the wetland may be necessary.

## CONCLUSIONS

In summary, the following statements provide reasonable assurance that when wetlands are incorporated into stormwater management systems, the hydroperiod of the wetland will be maintained or may improve in the case of previously altered wetlands and if used for water quality treatment, will not cause adverse impacts to the functions and values provided by the wetland.

1. The hydroperiod of isolated wetlands can be determined by using the water budget analysis.

2. Wetlands can be incorporated into the stormwater management system (i.e., attenuation and treatment) provided that all necessary criteria of the governmental agencies requirements/guidelines/policies including pre-treatment (removal of sediment, oils and greases) of runoff have been met.

3. The depth, duration or frequency of inundation should be analyzed by using a mean annual storm event (2.33 year - 24 hour storm) and at least one flood storm event such as a 25 year - 24 hour or a 100 year - 24 hour storm event.

4. The duration of inundation of stage excursions above the pre-treatment stage should be limited to 24 hours in any storm event (i.e., the difference between the pre- and post-development stage hydrographs (stage versus time) should not exceed **24** hours at the SHWL stage.

5. If the wetland is used for the treatment of stormwater runoff, a water quality recovery structure(s) between the wetland and the proposed stormwater system (dry and/or wet detention) should **be** considered. The top elevation of the structure(s) should be established between the SHWL **and** NP elevations depending upon the treatment volume provided in the wetland.

6. If the overland sheet flow from the rear yard is designed to directly discharge into the wetland, a minimum of 80 to 100 feet vegetative (grassed) filter strip including the wetland's buffer should be considered.

## ACKNOWLEDGMENTS

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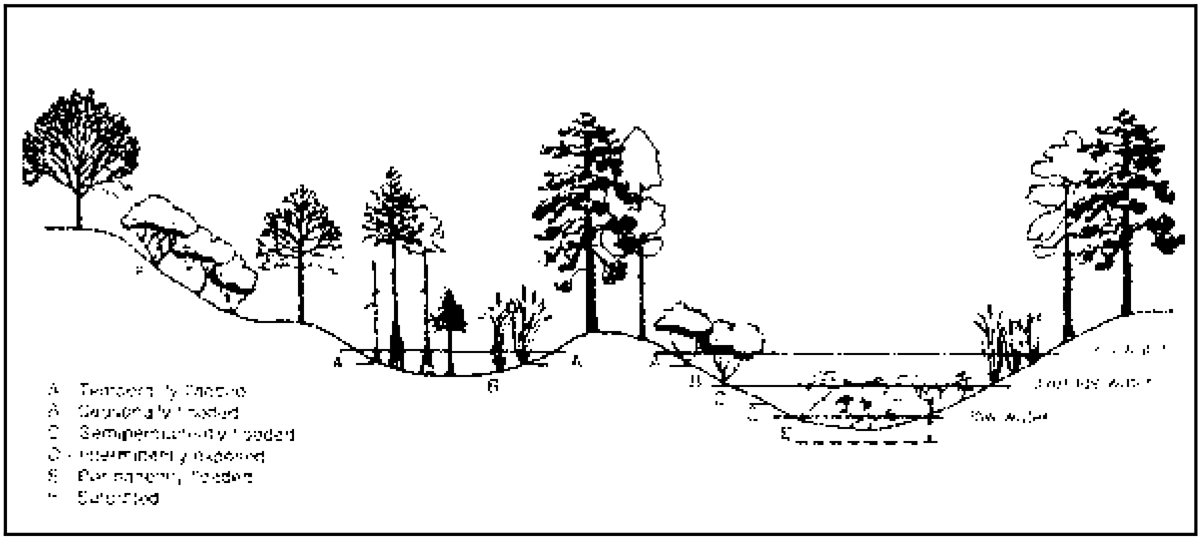

**Figure 1. Hydroperiods and hydrological indicators in nontidal wetlands. (After Cowardin et al., 1979)**

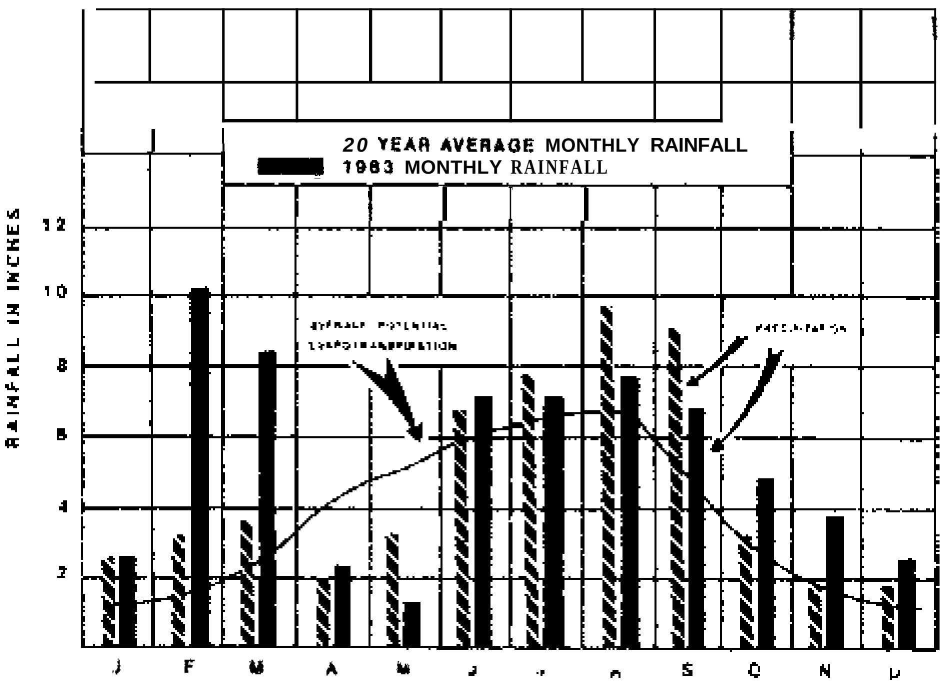


**Figure 2. Monthly rainfall averages and monthly potential evapotranspiration average.**

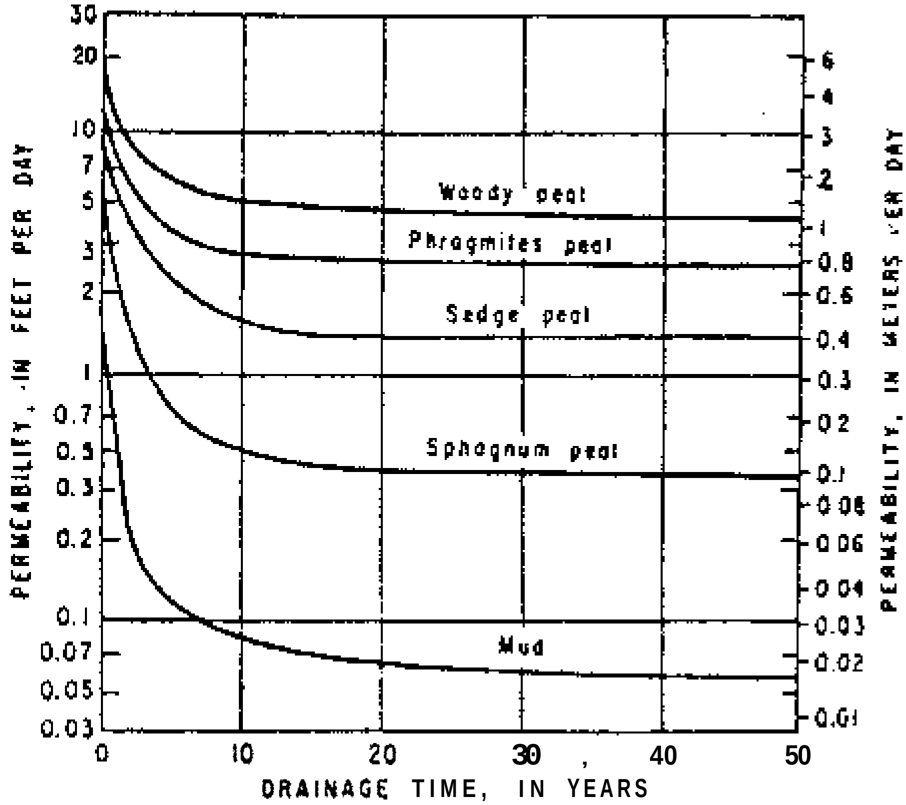


**Figure 3. Decrease** of **peat** and mud **permeability with time following** drainage. **(After Eggelsmann,** 1972)

| Parameters | Percent Average Removal in Natural Wetlands |
|---|---|
| TSS | 76 |
| TN | 24 |
| TP | 36 |
| TZn | 50 |
| TPb | 68 |

**Table 1. Average removal efficiencies of some** pollutants in natural **wetlands. (After Strecker et** al., **1992)**

# RETROFITTING AND MITIGATION OF A HIGHLY POLLUTED WETLAND IN AN URBAN AREA

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## ABSTRACT

Today, the acquisition of a good site is only the beginning of the development process and in some ways, it may be the easiest. Community comprehensive plans, concurrency requirements and regulations can guide investorsldevelopers toward property which may provide value now or in the future.

This commercially zoned property fronts the highway and because of poor drainage, was being flooded by polluted runoff, causing the centrally located, unhealthy wetland to expand to twice its original size (almost 9 acres). The recovery of the wetland, if at all possible, would have taken years. Toward the back of the property, however, was healthy wetland of almost 7 acres. North Dale Development's idea was to relocate the unhealthy wetland so as to connect with the healthier one, thus creating a larger total area while allowing maximum of his highly valuable property. While much smaller wetlands had been relocated, no one had ever tried one of this magnitude.

Ultimately, they not only moved the unhealthy wetlands, but also enhanced it in its new location by adding thousands of trees, plants and shrubs creating a net environmental and economic benefit by:

- Improving the wetland function
- Providing a noise barrier for residential areas nearby
- Adding more functional habitats for endangered and protected species
- Improving drainage for the area
- Improving water quality
- Enhancing recharge capability for water supply
- Permitting a fair development of an economically-desirable development site

This innovative approach leads the way for those who want to gain from their property investments while protecting the environment.

# RETROFITTING SMALL URBAN PARKS FOR WATER QUALITY ENHANCEMENT, TOM'S AND BOGGY BAYOUS, CHOCTAWHATCHEE BAY

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## ABSTRACT

The City of Valparaiso, Florida has several small urban parks that border two bayous of the Choctawhatchee Bay. Several of these parks are situated such that they encompass the outfalls for small watersheds that discharge into the bayous. The watershed areas are primarily comprised of older residential and light commercial development. Most of the runoff water entering the parks is runoff from public streets and private lawns. Degradation of water quality in the neighboring bayous has been documented, and traced to nonpoint pollution arising from the urban nature of the drainage areas. In an effort to begin the process of improving conditions within the bayous, the NWFWMD has designed systems tailored to each park that will provide at least some treatment of the runoff currently passing though the parks. The objective of these designs is to begin improvement of the water quality in the surrounding bayous by using existing park land and minimal disturbance to the public access and recreational uses of the parks by incorporating the design into the existing infrastructure to the extent possible.

## INTRODUCTION

The City of Valparaiso (City) and the Northwest Florida Water Management District (District), in conjunction with the Florida Department of Environmental Protection (FDEP), have jointly developed a project to reduce nonpoint source (NPS) pollution and mitigate its effects within both the Tom's Bayou and the Boggy Bayou basins, parts of Choctawhatchee Bay. This was a joint project funded by an EPA Section 319 grant awarded to the District. The project is also a part of the Surface Water Improvement and Management (SWIM) Program and the Florida Pollution Recovery Program for the ecological restoration of the Choctawhatchee Bay watershed. Nonpoint source pollution will be reduced through the design and implementation of BMP's in City owned parks bordering the bayous by retrofitting their existing stormwater outfalls. These BMP's include vegetated buffers, the creation of wetland communities, restoration of stream riparian zones, and the creation of small detention ponds that will function as created wetlands. The main objective of the project is to reduce the nutrient enrichment and sedimentation loading that have occurred as a result of urban runoff, in the process creating functional wetlands that will improve the quality of waters entering the Choctawhatchee Bay, while maintaining the recreational function of the existing parks

The District's Surface Water Management Bureau conducted an assessment of six park sites within the City of Valparaiso for their potential inclusion in the design project. The park sites were analyzed using the following criteria:

1. Ability to reproduce pre-development hydrological conditions as it relates to documented downstream aquatic habitat and/or severe erosion,
2. Ability to provide pollution removal capability,
3. Site feasibility (i.e., physical restrictions, total contributing watershed area and infiltration rate of soils),
4. Cost effectiveness (i.e., construction costs, BMP's, economies-of-scale),
5. Future maintenance,
6. Assurance that construction of BMPs would not cause adverse impacts on the environment.
7. Ability to incorporate water treatment strategies into existing park structure with minimal disruption to current park function.

No action was recommended on three of the six assessed sites based on the lack of water quality impacts, construction costs, physical restrictions, total contributing watershed areas and/or maintenance feasibility. The three sites selected for design, Lincoln Park, Glen Argyle Park, and Clearwater Park, were determined to be the ones that would provide the greatest opportunity to facilitate the achievement of the objectives for this project.

**Project Site Locations and Descriptions**

The project sites are located in small watersheds within the central portion of the Tom's and Boggy Bayous watershed. All the project sites are located within the City of Valparaiso, Okaloosa County, Florida (Figure 1). Locations and descriptions of the individual park sites and their associated design parameters for stormwater management are presented in the following sections.

**Lincoln Park.** Lincoln Park is the smallest of the three project sites with a drainage area of approximately 8.9 hectares (22 acres). The park itself has an area of about 4 hectares (10 acres). (Figure 2). It is located in Section 7 of Township 1 South, Range 22 West. Land use within the Lincoln Park watershed is well developed, consisting predominantly of single-family residential development. Stormwater runoff from this watershed occurs both as sheet flow and as channelized flow through an existing drainage pipe system. The drainage system connects to a ditch that runs north-south, paralleling Bay Shore Drive within the park, and out through a culvert into Boggy Bayou. The existing channel is approximately 122 meters (400 feet) long and is located in the western portion of the park adjacent to Bay Shore Drive. The area of the swale that provides water quality treatment and flood attenuation is approximately 43.6 cubic

**Proposed Stormwater Treatment Facilities and Habitat Enhancement**

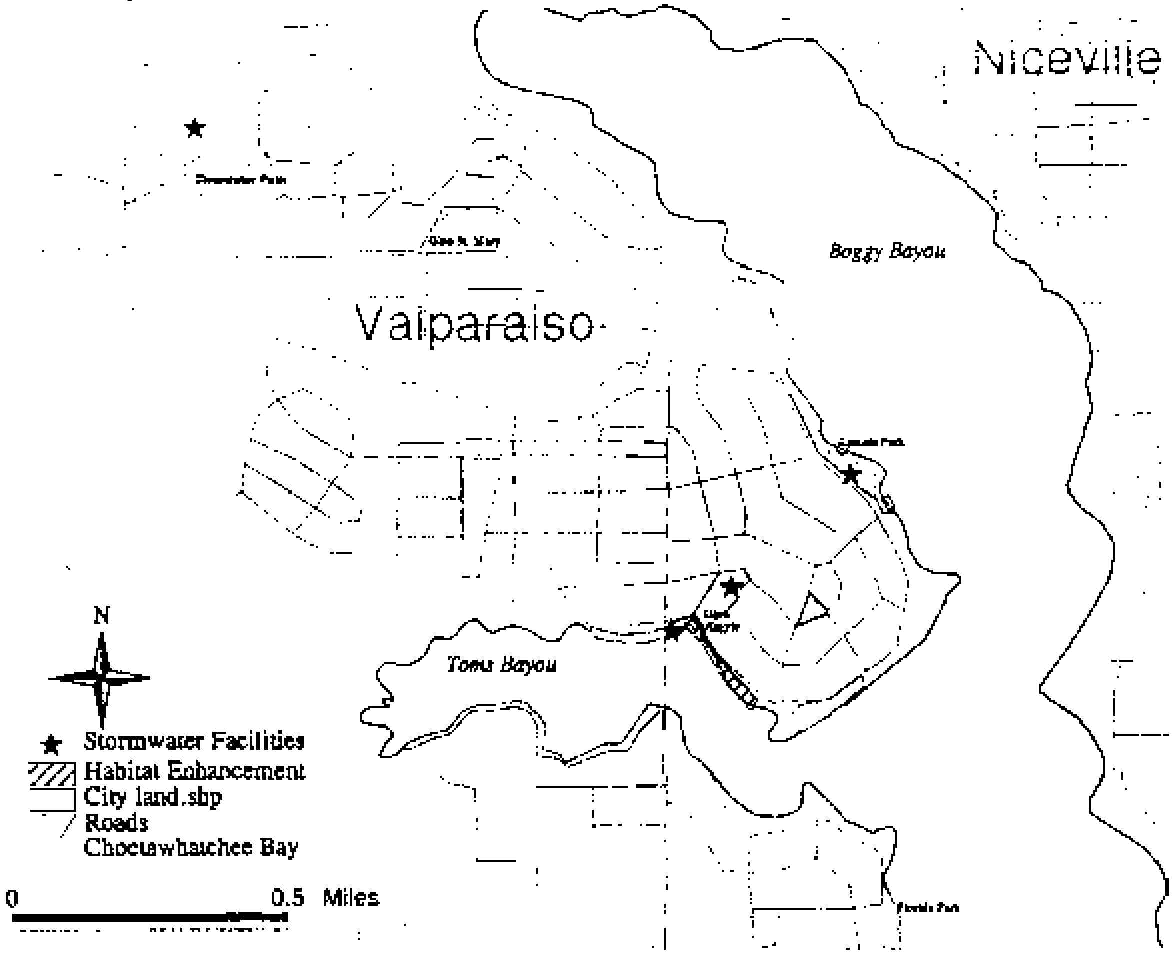



Figure 1.

meters (**1,540** cubic feet **or** 0.035 acre-feet) **with an** average depth of 0.3 to 0.6 meters (one to two feet) and with a bottom elevation of 0.7 to **2.1** meters (2.28 to **6.74** feet) above sea level.

The Lincoln **Park** wetland restoration design project consists of watershed water quality and habitat improvements. The stormwater management treatment facility has been designed as a pocket wetland and will expand the current facility in **an urban** sub-basin. The proposed design, with a total storage of 1,357 cubic meters (1.1 acre-feet), will consist of meandering channels, low marsh terraces along the swale, a small detention pool, wetland buffers, stormwater diversion and erosion **and** sediment controls. Pollutant removal capability for this design type is moderate due to the probability of resuspension and groundwater displacement. Plant diversity and wildlife habitat value will be improved by control of water levels and a planting schedule.

Glen Argyle Park. Glen Argyle **Park** has an existing drainage channel that **runs** the length of the park from Glenview Avenue to Bay Shore Drive and out into Tom's Bayou (Figure 3). It is located in Section 18 of Township 1 South, Range **22** West. The **2.8** hectare (7 acres) park is located in a well- developed residential subdivision and, in it's current state, has been determined

## LINCOLN PARK

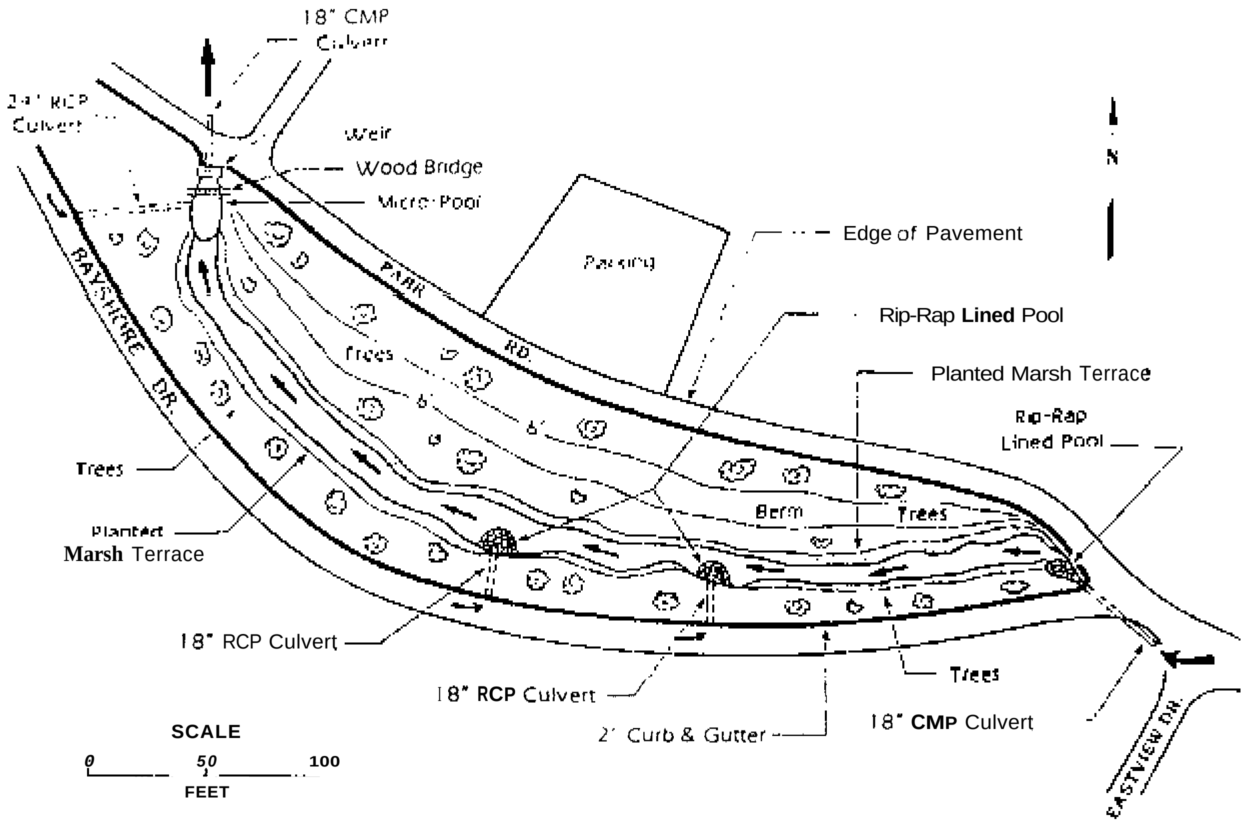



Figure 2,

to provide suitable access, existing habitat and neutral impacts to the environment. Runoff comes from a Combination of baseflow and stormwater that is delivered to the site via a single discharge pipe. The project drainage area is about 20 hectares (50 acres). In order to increase the efficiency of the stormwater treatment for this project, additional drainage areas were included in the design. A stormwater diversion on Bay Street was included, and a baffle box is proposed adjacent to the park to treat additional stormwater runoff that currently is being deposited directly into Tom's Bayou at the bridge approach from a section of John Simms Parkway.

Water quality treatment for Glen Argyle Park wetland restoration project will be provided by a shallow marsh wetland system. The system has a relatively large surface area and requires a reliable source of baseflow or groundwater supply to mitigate in impacts of stormwater quality and quantity that occurred during the process of development.

Stormwater will be temporarily stored in shallow, terraced pools. The park wetlands surface allocation has been approximately divided into high marsh (40%), low marsh (40%) and deep pool (20%). The majority of the shallow marsh system is from zero to 0.457 meters (18 inches) deep, which will create favorable conditions for the growth of emergent wetland plants. Shallow marsh systems have been shown to have moderate, reliable pollutant abatement capabilities.

Stormwater runoff will be captured in a forebay and micropool. Water quality treatment will be furnished in the upper reaches of the system. Flood attenuation will be provided in the lower reach and will be controlled by an elbow pipe and culvert. A wetland buffer landscaped with native plants will be incorporated into the park for both the removal of urban pollutants and habitat potential.

**GLEN ARGYLE PARK**

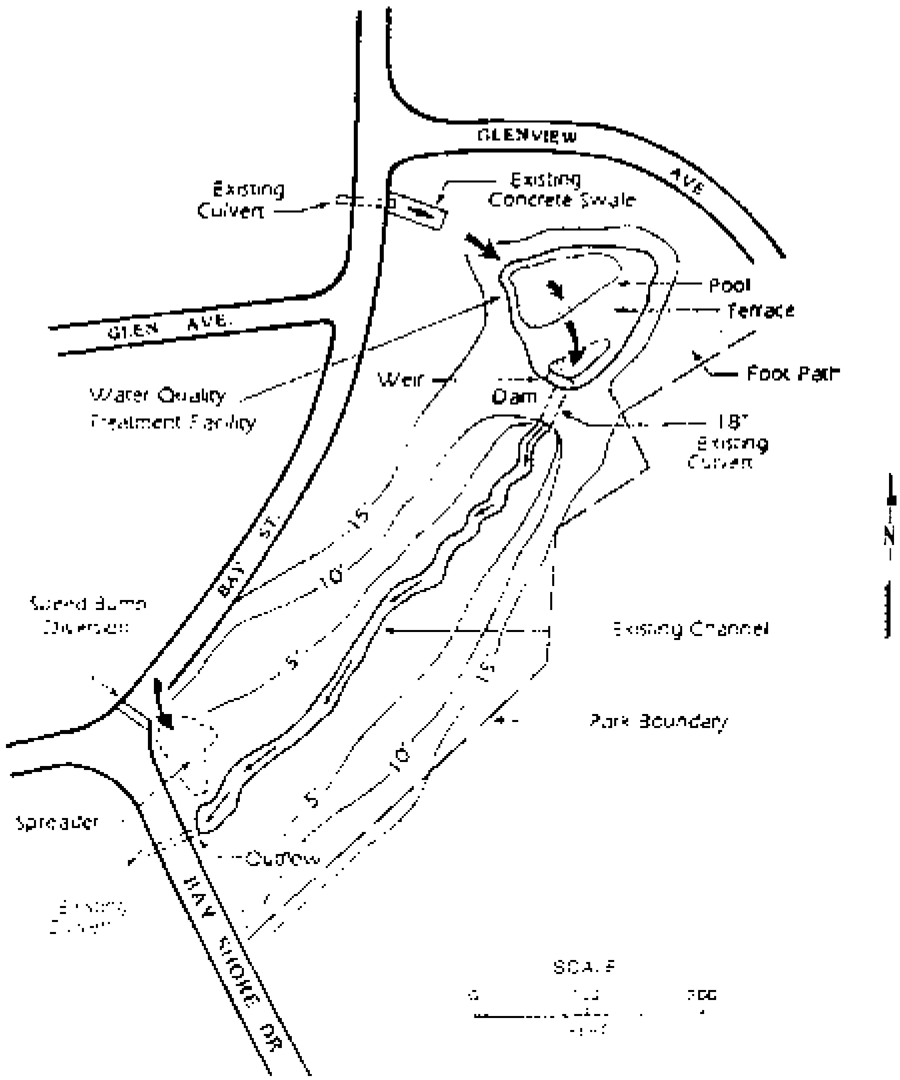
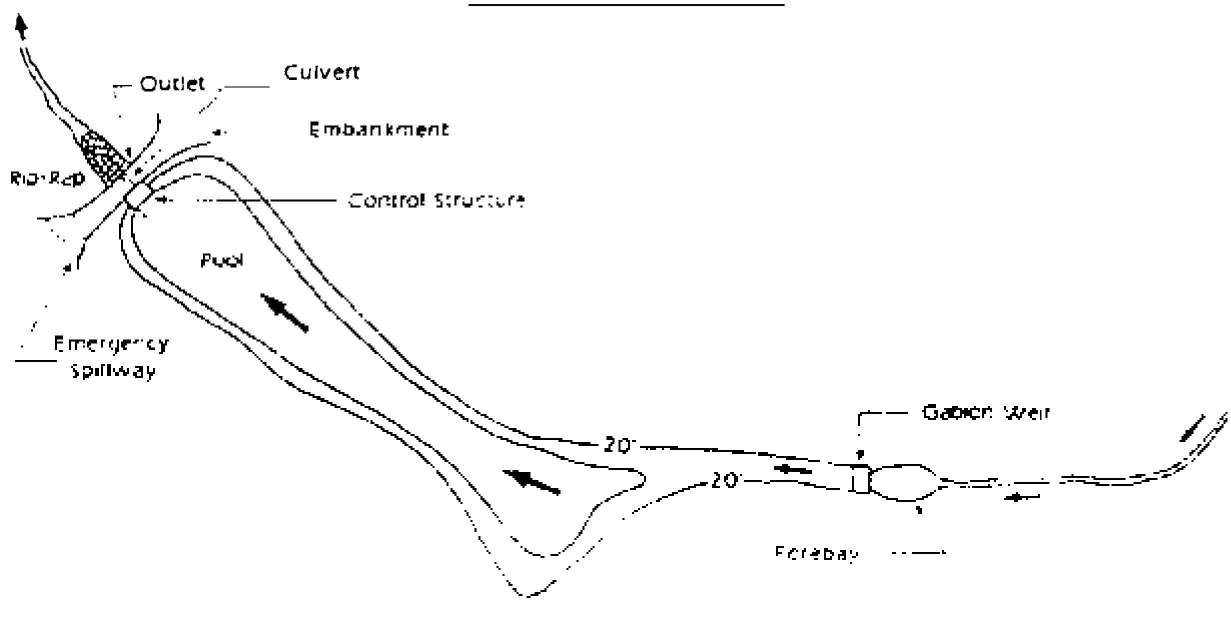



**Figure 3.**

**Clearwater Park.** Clearwater Park is the location of a former pond, prior to an embankment failure, that now handles stormwater runoff and baseflow via a shallow marshy channel that runs the length of the park (Figure 4). The 2.8 hectare (7 acres) park has a drainage area of about 58 hectares (143 acres). It is located in Section 12 of Township 1 South, Range 23 West. The park is located between Eglin Air Force Base and the City of Niceville. Land use within the Clearwater Park watershed is well developed, consisting of single-family residential development and steep woodlands. This park showed the greatest potential for abating nonpoint source pollution through treatment in a wet extended detention pond system. Best management practices include a multi-stage discharge structure, extended detention basin, shoreline protection and sediment and erosion control.

The capability for pollutant removal is moderate but not always reliable for wet extended detention pond systems. Extra runoff storage will be created above the shallow marsh by the temporary detention of runoff. A new growing zone will be created along the slopes of the extended detention wetlands that extends from the normal pool elevation to the maximum ED water surface elevation. Environmental concerns are that fluctuating water levels would impose physiological constraints on native plant diversity but at the same time provide a potential buffer for wildlife habitat.

**CLEARWATER PARK**




Plan View

Figure 4.

## Design Methodology

The principle concerns for the restoration of wetland habitats within Tom's and Boggy Bayous center on degraded water and sediment loading. These issues derived, partly, from a long history of use as a receiving water body for NPS pollution from both urban runoff from the vicinity of the bay and basin-wide NPS pollution from the river watershed. Some contribution was found from point sources in the river watershed. The problem of cultural enrichment and resulting degraded water and sediment quality is particularly severe with and adjacent to the urbanized western bayous. The bayou currently receives concentrated stormwater runoff from the uplands and direct runoff from adjacent commercial and residential land uses. The principal focus of the Tom's and Boggy Bayous project was to define the nature and extent of current problems affecting the three parks and their watersheds, and to implement a program that will result in improved water and sediment quality in the bayous. This can be accomplished by retrofitting the parks existing stormwater flow characteristics to optimize treatment prior to discharging into the bayous. **A** technical assessment of current park conditions was conducted prior to the design and implementation of the restoration program, in order to understand the nature of existing problems. This assessment involved a determination of the current quality and quantity of stormwater inflows and an assessment of the current quality of water and sediment. This technical information was used to identify and rank feasible restoration alternatives that would be implemented in order of priority.

**A** system of practices, an integrated BMP treatment train, will be designed and implemented to facilitate the achievement of the objectives identified by the project partners for water and sediment quality, biological resources, public awareness and basin-wide coordination. The three parks, Glen Argyle, Lincoln and Clearwater, are located along the city's waterfront and/or adjacent to roads and residential areas, and have functioned as unobstructed routes for NPS pollution via sheet flow and stormwater conveyances. BMP's will include swales to convey storm flows, landscaped detention basins, wetland buffers and erosion and sediment controls. The proposed wetland restoration projects consist of watershed quality and habitat improvements. The anticipated functions of the general design concept of retrofitting existing public use parks are as follow.

1. Retain and transform nutrients from the stormwater;
2. Reduce velocity of stormwater entering the wetlands to allow sediments to settle;
3. Use long term detention to allow greater biological processing of nutrients in the water column;
4. Employ wetland plants to remove nitrogen and phosphorous and provide the functional resistance to incoming runoff and enhance nutrient retention by burial; and
5. Furnish habitat for wildlife, including waterfowl, mammals and unique vegetation;
6. Maintain existing uses of the parks with minimal disruption to the public.

Incorporation of wetland systems in the three Valparaiso parks for stormwater treatment provides a management technique for addressing stormwater quality, as well as flood mitigation, habitat creation, and aesthetics. These systems were integrated into the drainage paths of all three existing park developments. They include retrofitting an existing swale as a pocket wetland at Lincoln Park, transforming an intermittent marshy swale to a shallow marsh system at Glen Argyle Park, and constructing a wet detention pond with fringe wetlands to provide water quality treatment and flood attenuation at Clearwater Park.

## RESULTS

Retrofitting a design for incorporation into existing parks to optimize the stormwater treatment for runoff through these sites shows there can be measurable benefits to this procedure. In the Lincoln Park design the total phosphorus (TP) loading for this area was estimated to be 35.7 kilograms (78.64 pounds) per year and the total nitrogen (TN) loading was estimated to be 450 kilograms (990.32 pounds) per year. The projected, average pocket wetland system pollutant removal rate for TP is calculated to be 25% and TN is calculated to be 15%. It is therefore anticipated there will be an overall loading reduction to the bayous system in TP of around 8.9 kilograms (19.66 pounds) per year and a reduction of about 67.4 kilograms (148.55 pounds) per year of TN. The pollutant removal rate for total suspended solids is estimated at 60%. For the Glen Argyle Park site it is estimated that the park alone currently contributes approximately 20.9 kilograms (46 pounds) **per** year of TP and 261.5 kilograms (576 pounds) per year of TN to the bayous system. Pollutant removal rates achievable with best design are 60% for TP, 45% for TN and ***85%*** for total suspended solids. This would mean approximately 12.7 kilograms (28 pounds) per year of TP and 58.1 kilograms (128 pounds) per year of TN would be removed from the runoff currently entering the Bayou. It is estimated that Clearwater Park currently contributes approximately 232 kilograms (511 pounds) per year of TP and **2922** kilograms **(6437** pounds) per year of TN. Pollutant removal rates achievable with best design are 65% for TP, 40% for TN and **76%** for total suspended solids. By incorporating the construction and maintenance of the stormwater management facilities within the existing parks, approximately 151 kilograms (332 pounds) per year of TP and 1169 kilograms (2575 pounds) per year of TN will be removed from the runoff entering the Bayou.

## CONCLUSIONS

Stormwater wetlands situated in urban areas offer challenges to the designer to integrate social factors, safety, recreational uses and maintenance, along with hydrology and wetland plant ecology. **A** stormwater system would function as a natural wetland while addressing **the** needs of the adjacent community. The selection of these particular wetland designs was dependent on four factors: contributing watershed, available space, desired environmental function for the wetland and capital. The primary goal of the stormwater wetlands projects in Valparaiso is to maximize pollutant removal with minimum construction and disturbance to the existing parks, and to create generic wetland habitats that will meet the greatest needs. When properly designed, constructed and maintained, stormwater wetlands have many advantages as an urban BMP, including reliable pollutant removal, longevity, adaptability to many development sites, excellent wildlife habitat potential, while maintaining public recreational benefits.

## *ACKNOWLEDGMENTS*

The authors and staff of the District would like to acknowledge the assistance of the City of Valparaiso for their willingness to provide the sites for this design project **and** cooperation in providing surveying and other information regarding these sites.

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# STORMWATER RETROFIT OF THE ABANDONED JAN-PHYL WASTEWATER TREATMENT PLANT SITE

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## ABSTRACT

The Jan Phyl Stormwater project was completed in January 1998 to retrofit the abandoned wastewater treatment plant site to provide treatment of stormwater runoff through nutrient and sediment removal. The project also created storage volume to reduce localized flooding for a 90 acre portion of the watershed of the Winter Haven Chain of Lakes which is a SWIM Waterbody. Of the seven acre total project area, four acres were utilized for stormwater treatment. The existing wastewater percolation ponds were retrofitted as wet detention ponds to provide stormwater treatment of the runoff from the first 1.25 inches of rainfall, Sediment excavated from the existing percolation ponds was tested for Fecal Coliform, nutrients and Toxic Contaminant Leaching Potential (TCLP) to verify the material met the criteria established under Chapter **17-640** of the Florida Administrative Code (FAC) for the disposal of waste water residuals. It was originally estimated that nine tons of Total Nitrogen and six and one-half tons of Total Phosphorus was removed with the sediment from this site and disposed of in accordance with FDEP approval. Over 25,000 aquatic plants were placed in the littoral zone. The remaining three acres of property have been sodded to allow for passive recreation and to educate visitors through the use of signs depicting native fish, water fowl and aquatic vegetation. Water quality monitoring is being performed to determine the pollutant load reductions achieved at this facility.

## INTRODUCTION

The Jan Phyl retrofit project was developed as an innovative way to solve a local flooding problem. Stormwater runoff from the adjacent residential neighborhood discharged to a ditch on the project site property boundary with no treatment being provided. Abandoned ponds from a County operated wastewater treatment plant (WWTP) remained following the relocation of the sanitary sewer facilities. The engineering **firm** of Bromwell & Carrier Inc. (BCI) was hired to design a stormwater treatment facility with sufficient capacity to reduce flooding of County roads. The result was a system which also addressed water quality improvements which otherwise would not have been considered.

The project entailed removal of wastewater sediments which had accumulated over the 30 years the sewage treatment plant was in operation. Over **8,800** cubic yards of material was removed from the pond area for disposal off-site. Removal of the abandoned plant superstructure and renovation of the ponds allowed for treatment of stormwater prior to being discharged to Lake Howard. The pond bottoms were re-contoured to provide littoral shelves for planting wetland vegetation for nutrient removal. More than 25,000 herbaceous wetland plants were placed at the site. **A** mitigation monitoring program was established to verify the success of the wetland system.

Stormwater monitoring was initiated following establishment of the littoral vegetation. Automatic samplers are used to collect flow weighted composite samples during rain events. Stations were designated immediately upstream and downstream of the ponds to obtain untreated and treated samples, respectively. Additional grab samples are collected from the downstream sampler at regular intervals to determine the recovery period for the ponds. Base flow samples were collected at the start of the monitoring program to identify pollutant contributions from ground water sources. The results are to be used to determine the pollutant removal efficiency of the system.

Federal matching funds were obtained through a Section **3** 19 grant administered by the Florida Department of Environmental Protection (FDEP). Additional funding was provided by the Southwest Florida Water Management District (SWFWMD) through the Surface Water Improvement and Management (SWIM) program as Lake Howard is a designated SWIM water body. The Polk County Board of County Commissioners funded 50% of the $368,565.14 total project cost with the remaining 50% being matched by the SWFWMD and the FDEP.

## Project Team / Objectives

The Jan Phyl project was initiated through discussions between the County's Natural Resources Division, the Drainage Division and the local FDEP representative. The Natural Resources Division managed the contracts for matching funds with the SWFWMD and the FDEP, and hired the firm of BCI for design and construction supervision. The primary objective was to solve the problem of local flooding. Utilization of existing ponds from the abandoned wastewater plant allowed for this to be accomplished while also addressing water quality improvements to the Winter Haven Chain of Lakes.

## Existing Conditions

Jan Phyl Village was developed in the late 1950's **as** a residential community. It is located in Polk County just west of the City of Winter Haven, The seven acre Jan Phyl wastewater plant site was fenced and had remained abandoned following construction of the County's Central Regional wastewater facility. Portions of the concrete sludge digester unit remained along with iron pipe and other material. Vegetation consisted primarily of Bahia grass which stabilized the pond berms with some exotic hardwoods mixed with the few pine trees on the site.

A perimeter ditch along the west and south sides of the property conveys stormwater runoff from the 90 acre watershed approximately one mile to Lake Howard. Drainage from the adjacent residential neighborhood discharged east to this ditch without receiving any treatment, contributing a significant amount of sediment to the storm sewer system. Tail water effects from the ditch system resulted in frequent flooding of roads.

The wastewater treatment plant operated until 1989, discharging effluent to three percolation ponds on the seven acre site. Samples were collected from the sediment within the ponds and determined to meet the domestic Wastewater residuals criteria for land disposal under Chapter 62-640 **FAC.**

## Design & Permitting

Attenuation of peak flood levels and a reduction in the duration of roadway flooding was achieved through the improvements to the existing storm sewer system, and the additional storage designed into the 3 acres of pond area. The original three pond design was modified through interconnection. **A** 24 inch outlet was installed in the first pond (Pond **A),** on the end opposite the inlet structure, to connect the second and third ponds which were combined into a single, larger pond (Pond B). **A** diversion structure was installed at the end of the 48 inch storm sewer pipe to direct flow to the Pond A through two 24 inch pipes. **A** third 24 inch pipe was installed with the invert one foot above the other two pipes so that any additional flow would be discharged directly to Pond B. The diversion structure was designed with an overflow one foot above the top of this pipe to allow peak flows to be routed to the perimeter ditch. The top of the pond berms included two feet of freeboard as a safety factor to avoid over topping. Figure 1 shows the "as-built" plan of the treatment ponds prepared by BCI.

Retrofitting the storm sewer system provided the opportunity to address water quality, The facility was designed to provide treatment of stormwater runoff from the first 1.25 inches of rainfall over the entire contributing basin. The original berm between the second and third ponds was lowered to provide additional littoral zone area. An outlet structure was located on the opposite end of the newly formed Pond B to provide the maximum detention time for treatment. This outlet consists of an eight inch pipe which acts as a bleed down for the ponds. **A** cap with a four inch orifice was installed on the pipe to control the discharge rate so that the required 120 hour detention time could be achieved. Littoral shelves were designed on a 8:1 horizontal:vertical slope. Stormwater treatment was to be provided using herbaceous vegetation at various depths along the littoral zone,

Permitting for this project followed the standard Environmental Resource Permit (ERP) application procedure. Application was made to the SWFWMD in March **1997** with the final permit being issued in June. **A** Notice of Intent (NOI) to comply with the National Pollutant Discharge Elimination System (NPDES) general permit for construction was completed. The NOI was submitted to the U.S. Environmental Protection Agency (EPA) prior to the initiation of construction. A Stormwater Pollution Prevention Plan ($SWP^3$) was prepared for the project according to federal requirements.

## Project Construction

Construction began in August 1997 with the initial stages involving demolition of the remaining WWTP components. **A** large portion of the concrete sludge digester had to be crushed and removed along with cast iron sewer pipe that was scattered around the site. Erosion controls were placed along the perimeter ditch to reduce off-site sediment transport. De-watering of the ponds was required to remove the sludge material which remained from the WWTP operation. The original intent was to utilize this material on the pond berms to provide substrate for the sod which was needed to stabilize the side slopes. Limited working area prohibited stockpiling of the material and removal off-site became necessary. The material was pushed into earthen cells for drying prior to loading for disposal.

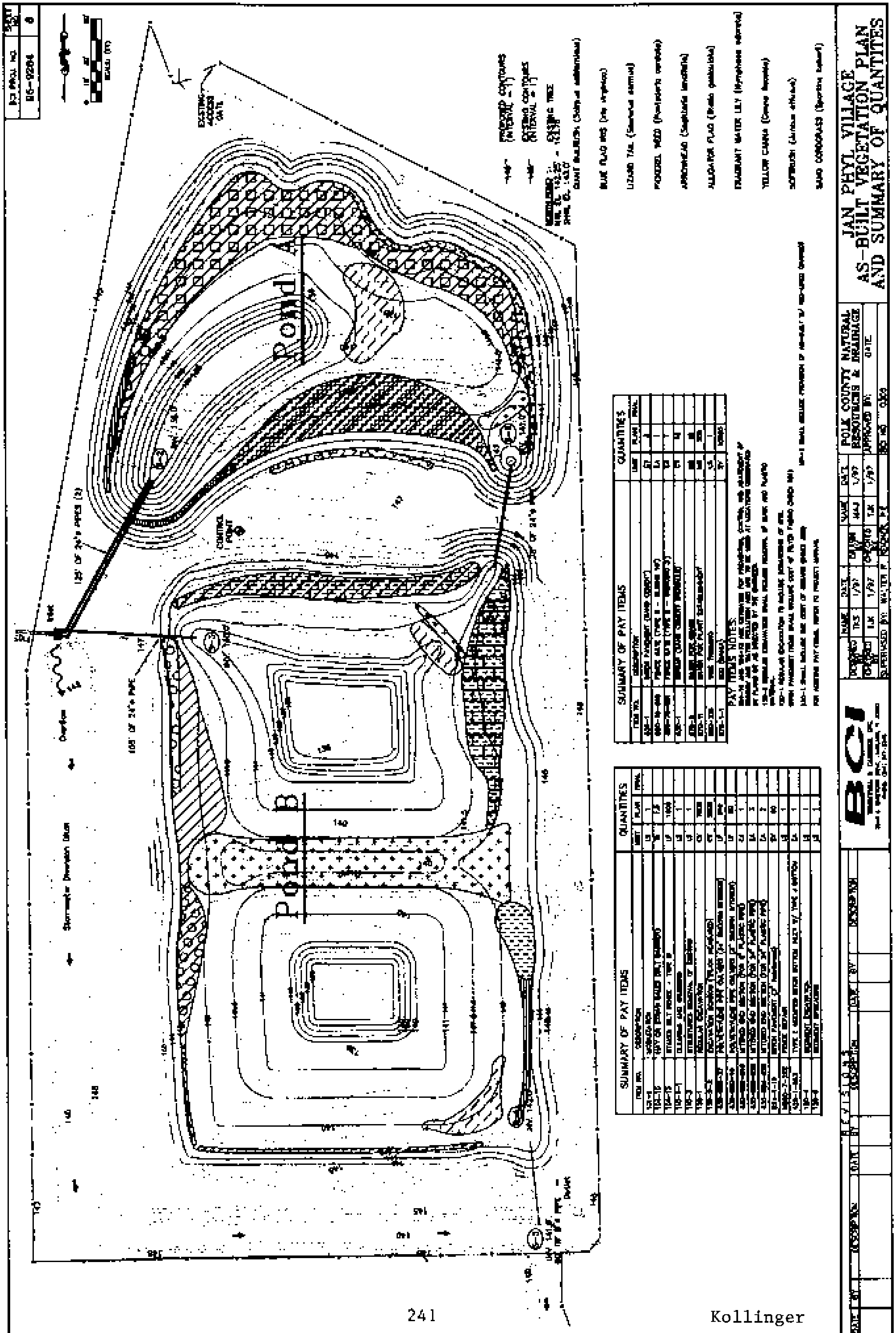


Figure 1 - As-built plan of treatment ponds.

TCLP testing of the bottom material was required by FDEP so the material could be disposed of by land application. Results of the analysis for heavy metals indicated the material met the disposal criteria for Class AA wastewater residuals. Chapter 62-640 FAC requires stabilization of the material for pathogen reduction in order to meet the Class **AA** standards. Since the ponds had been inactive for eight years, the County opted not provide further stabilization. The material therefore needed to be land applied onto a site with limited public access.

**A** summary of the analytical results for heavy metals is provided below for comparison with the regulatory standard:

| **Parameter** **Analyzed** | **Class AA Standard** Chapter 62-640-(mg/kg dry wt.) | **Analytical Results** Average Conc. (mg/kg dry wt.) |
|---|---|---|
| Cadmium | < 30 | < 0.5 |
| Copper | < 900 | **89.9** |
| Lead | < 1000 | 10.3 |
| Nickel | < 100 | **3.4** |
| Zinc | < 1800 | 282.3 |

Two sites were evaluated for disposal subject to FDEP approval. The first location was the County's North Central Landfill, approximately *6* miles from the project. The second site was the Polk County Skeet and Trap Club located 1.5 miles from the project site. This property is owned by the County and leased to the skeet and trap club. The property was originally used as a borrow pit for road construction materials. It was later operated as a solid waste disposal facility by the County until the early 1970's. Field investigations showed this second location to be acceptable and the site was selected for land disposal of residuals. The shot fall zone for the skeet & trap range was essentially bare of suitable vegetation. Exposed soils and debris covered an area of approximately 9.7 acres.

Chapter 62-640 FAC specifies loadings for nitrogen as well as metals which are land applied. The application rate of nitrogen for Bermuda grass is 250 lbs./acre/year. Based on the nitrogen concentration in the residuals, and the estimated volume of material to be removed, it was calculated that a total of 1850 Ibs. of nitrogen would be applied, This equates to a loading of 190 lbs./acre, which is within the regulatory requirement. Calculation of the metals loading confirmed the amount of material applied was below the maximum allowable cumulative total required by the rule. The FDEP agreed to the suitability of the site and approved the request for land application of the material.

The pond sediment was applied by dump truck and spread using a bull dozer at a depth of approximately **8** inches. Over 8,800 cubic yards of sediment were actually removed from the site with an average solids content of **28%.** The application area was contoured and provided with an earthen curb to reduce the potential for runoff to an adjacent wetland area during storm events. Recruitment of Bermuda grass from the surrounding field minimized the need for seeding and a permanent ground cover was established naturally. An adjustment in the calculation for nitrogen loading using the geometric mean of the sample results revealed that a total of 4.5 tons of total nitrogen was actually removed from the ponds and made available for use in establishing this ground cover.

It was interesting to note that the maximum amount of lead which was land applied is insignificant in comparison to the lead shot deposited during a single shoot by the club. In fact, lead shot is readily visible on the ground surface in the shot fall area. During an average club event with 150 entrants, 100 rounds of 1 oz. loads per day may be shot by each participant. Over a three day period this results in 2,800 lbs. of lead shot being deposited. The amount of lead deposited with the pond sediment was calculated at 80 pounds.

Pond construction commenced with the sediment removal. Heavy equipment was used to re-contour the three ponds to the design elevations specified for Ponds **A** and B. **A** nine foot deep sump was created at the inflow to Pond **A** to provide for particle settling and provides access for sediment removal during maintenance. The side slopes were set at a **4:**1 slope from top of berm down to the water's edge in accordance with the SWFWMD rules since the area is open to the public. **A** shortage in available fill from calculated cuts however, resulted in less littoral shelf being established in Pond B. The shortage of fill required the use of borrow material from off-site. This created a separate problem in that the clay content of the source material resulted in excessive turbidity when Pond B was filled. The problem was solved initially with the application of aluminum sulfate (alum) which allowed for flocculation of solids which settled, clearing the water. **A** long term solution to this problem is expected from stabilization of the pond with the establishment of the vegetative cover.

Ten different species of wetland plants were installed following completion of the ponds. Common names for these plants include: giant bulrush, blue flag iris, lizard tail, pickerel weed, arrowhead, alligator flag, fragrant water lily, yellow canna, soft rush, and sand cordgrass. The survival rate during the first year of operation varied by plant species. Replacements in subsequent replanting were done according to species viability. The giant bulrush installed bare root on the submerged berm in the center of Pond B did not survive. This **was** apparently due to the higher clay content of the material used in construction of the berm as the species did establish in deeper water at the outlet of Pond **A.**

The project site was fenced and gated to allow limited public access for passive recreational activities including walking, jogging and fishing. Kiosks were erected at four locations on the project site to provide information to the public. One display explains graphically how stormwater is treated at the facility. The other three displays identify the fish, birds and types of vegetation commonly associated with wetland areas, Project construction was complete as of the end of January 1998.

## Wetland Mitigation

**A** total of **2** ½ acres of herbaceous wetland from this site was subsequently identified as a mitigation offset for the Lake Deeson water level control project. Florida Permitting, Inc. was hired by the County for semi-annual monitoring and quarterly maintenance of the wetland vegetation. **A** supplemental planting was completed in March 1999 in which the giant bulrush (*Scirpus californicus*) lost from the submerged berm in Pond B was replaced with a smaller bulrush (*Scirpus validus)*. Fully rooted plants in **4** inch pots were used rather than bare root to offset the effects of the clay substrate. Spatterdock (*Nuphar lutem*) was planted along the outer edges of this berm and is doing very well. The supplemental planting also included replanting of soft rush, canna lily arrowhead and the introduction of prairie iris.

Pond A has been most successful with the overabundance of pickerelweed which has established. An inspection of the site following completion of the first mitigation report indicates this pond meets the criteria for release from monitoring.

**Project Costs**

This project was completed at a total cost of $368,565.14 which included engineering design, permitting, construction and public education. Construction costs of $288,139.30 were paid to the site contractor, Royal Construction, Inc. of Tampa. Engineering design and project management fees in the amount of $21,500.00 were paid to BCI. **As** a cooperatively funded project, the SWFWMD permit application fees were waived and $2,775.84 was paid by the FDEP for construction of the public education kiosks. The remainder of the costs include the assessed value of the property and additional materials provided by the Polk County Board of County Commissioners.

The cost of maintenance and monitoring of the site will continue to be incurred by the County throughout the life of the project, Mowing of the grassed berm area is performed twice per month during the wet season in order to allow access to the site by the public. Quarterly inspections of the area are performed under contract and semi-annual reports are prepared to document the success of the mitigation project. Maintenance is performed as required to control invasive species, and desirable species are replanted as needed. In April 1999 an additional $5,374.00 was spent to replace plants to meet the mitigation density criteria.

**Stormwater Monitoring**

Sample collection began in February 1999 to monitor water quality for determination of the treatment efficiency of the ponds. An American Sigma Model 900 automatic sampler was installed at the 48 inch storm sewer system outfall from Jan Phyl to the project to monitor the runoff from the adjoining residential neighborhood. **A** second Sigma sampler was stationed at the 8 inch outfall from the ponds to the perimeter ditch to determine the quality of treated water which is discharged from the site. Velocity sensors were provided in order to determine flow for calculation of the volume of water treated.

Samples at the inflow and outflow from the project site are collected on a flow weighted basis during the first three hours following initiation of storm events with an intensity greater than 0.1 inches in 30 minutes. Follow-up grab samples are collected at the pond discharge at regular intervals to determine the average time for the ponds to return to base flow levels in terms of water quality. The intervals chosen initially are the 12, 24, and 48 hour periods following the onset of the qualifying storm event. These intervals were selected based on previous experience with the Derby Avenue stormwater treatment pond system previously monitored by the County. Table I provides a summary of the results from the events monitored to date. Base flow samples were also collected at the start of the monitoring program to identify pollutant contributions from ground water sources.

Treatment Efficiency of the ponds will be determined through comparison of results of samples collected during the first year of operation. Information from the Derby Avenue project suggests the efficiency is expected to vary depending on the parameter. Removal of nutrients in the 30-50% range, with slightly higher efficiencies of up to **75%** for heavy metals were reported for this project (King 1997). This information was supported by the averages obtained through literature reviews.

## CONCLUSION

It is feasible to retrofit existing storm drainage systems to address water quality with conventional treatment techniques where land is available. Although the cost associated with construction and maintenance of retrofits is significant, there are direct benefits to receiving waters that can be measured in terms of pollutant load reductions. It is difficult to attach a monetary value to the opportunity to see plants and wildlife in an urban neighborhood setting that a project such as this provides.

This project was able to convert idle public property which was previously used to treat domestic wastewater, and allow for passive recreation which was not permitted as a utility site. The project provides an opportunity to educate the public by demonstrating Best Management Practices for proper stormwater management. Efficiency of the treatment system in reducing pollutant loads to Lake Howard will be determined upon completion of the monitoring program,

## ACKNOWLEDGMENTS

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**Table I - Jan Phyl Storm Event Monitoring**

A comparison of analytical results of stormwater runoff at the Project site inlet to levels in the treated water discharged from the outlet.

| Parameter | Baseline | Inlet Initial Rain event 2/2/99 | Outlet Initial Rain event 2/2/99 | Outlet 12 hour Rain event 2/3/99 | Outlet 24 hour Rain event 2/3/99 | Outlet 48 hour Rain event 2/4/99 | Inlet Initial Rain event 4/28/99 | Outlet Initial Rain event 4/28/99 | Outlet 12 hour Rain event 4/29/99 | Outlet 24 hour Rain event 4/29/99 | Outlet 48 hour Rain event 4/30/99 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| BOD (mg/l) | NT | NT | NT | NT | NT | NT | >13.7 | 7.24 | 5.96 | 5.64 | 7.13 |
| COD (mg/l) | 29.5 | 125.2 | 58.7 | 36.7 | 65.1 | 40.1 | 188 | 74 | 66.3 | 63.7 | 68.8 |
| TOC (mg/l) | 9.577 | 17.43 | 12.67 | 9.749 | 12.26 | 10.27 | 13.56 | 13.73 | 12.1 | 12.31 | 12.59 |
| TSS (mg/l) | 4.4 | 160.4 | 30.4 | 15.6 | 29.6 | 25.6 | 279.6 | 50.3 | 37.5 | 84.0 | 76.7 |
| TDS (mg/l) | 168 | 134 | 168 | 160 | 159 | 149 | 71 | 189 | 165 | 168 | 168 |
| $NO_{2+3}$ (mg/l as N) | 0.000 | 0.312 | 0.003 | 0.006 | 0.004 | 0.018 | 0.749 | 0.124 | 0.194 | 0.060 | 0.034 |
| TKN (mg/l as N) | 0.782 | 3.489 | 1.847 | 0.857 | 1.398 | 1.191 | 6.773 | 2.178 | 3.394 | 1.838 | 1.983 |
| Total $PO_4$ (mg/l) | 0.367 | 0.842 | 0.412 | 0.169 | 0.302 | 0.276 | 1.677 | 0.691 | 1.689 | 0.596 | 1.292 |
| Dissolved O-$PO_4$ (mg/l as P) | 0.231 | 0.252 | 0.016 | 0.018 | 0.020 | 0.019 | 0.395 | 0.051 | 0.079 | 0.061 | 0.055 |
| Arsenic (ug/l) | <2 | <1 | <1 | <1 | <1 | <1 | <2 | <2 | 2.1 | <2 | <2 |
| Cadmium (ug/l) | 0.48 | <2 | <2 | <2 | <2 | <2 | 1 | 0.6 | 0.3 | 0.2 | 0.4 |
| Chromium (ug/l) | <2 | <5 | <5 | <5 | <5 | <5 | 14.3 | 3.4 | 3.4 | 2.2 | 8.7 |
| Copper (ug/l) | <10 | <20 | <20 | <20 | <20 | <20 | 40.1 | 12.1 | 14.3 | 13 | 19.1 |
| Lead (ug/l) | <3 | 12.5 | <5 | <5 | <5 | <5 | 33.9 | 4 | 3.5 | <3 | 9.5 |
| Nickel (ug/l) | <10 | <20 | <20 | <20 | <20 | <20 | 17.9 | <10 | 13.7 | <10 | 11.2 |
| Selenium (ug/l) | <2 | 5.6 | 3.9 | <1 | 4.1 | <1 | <2 | <2 | <2 | <2 | <2 |
| Silver (ug/l) | <2 | <5 | <5 | <5 | <5 | <5 | <2 | 2.02 | <2 | <2 | <2 |
| Zinc (ug/l) | 24 | 130 | 22 | 12.3 | 17.4 | <10 | 203.4 | 21.2 | 15.9 | 10 | 49.3 |

NT = not tested

**Table 1- Jan Phyl Storm Event Monitoring** (continued)
**A** comparison of analytical results of stormwater runoff at the Project site inlet to levels in the treated water discharged **from** the outlet.

| Parameter | Baseline | Inlet Initial Rain event 5/9/99 | Outlet Initial Rain event 5/9/99 | Outlet 12 hour Rain event 5/10/99 | Outlet 24 hour Rain event 5/10/99 | Outlet 48 hour Rain event 5/11/99 | Inlet Initial Rain event 5/12/99 | Outlet Initial Rain event 5/12/99 | Outlet 12 hour Rain event 5/13/91) | Outlet 24 how Rain event 5/13/99 | Outlet 48 hour Rain event 5/14/99 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| BOD (mg/l) | NT | 8.38 | *5.02* | 4.37 | 5.29 | *5.65* | 7.49 | 5.88 | 4.84 | **7.4** | 6 6 |
| COD (mg/l) | 29.5 | 10 3 | 46.3 | 44.6 | 55.8 | **43** | 59.5 | **56.4** | **59.5** | 72.3 | 80.2 |
| TOC (mg/l) | 9.577 | 14.08 | 12.38 | 12 99 | 13.12 | 12.59 | 12.5 | 11.74 | 11.76 | 14.05 | 14.23 |
| TSS (mg/l) | 4.4 | 41 3 | 17.3 | 30.0 | **23.7** | 17.3 | 14.7 | 19.3 | 28.7 | 26.0 | 52.7 |
| TDS (mg/l) | 168 | 144 | 178 | 188 | 190 | 174 | 154 | 186 | 184 | 204 | 208 |
| $NO_x$ (mg/l as N) | *0.000* | 0.911 | 0.001 | 0.014 | 0.014 | 0.015 | 0.162 | 0.000 | 0.005 | 0.023 | 0.017 |
| TKN (mg/l as N) | 0.782 | 2.585 | 1.691 | 1.805 | **1.313** | 1.254 | 1.366 | 1.357 | 1.415 | 1.462 | 1 559 |
| Total $PO_4$ (mg/l) | 0.367 | 0 561 | 0.263 | 0.505 | **0.244** | 0.186 | 0.348 | **0.239** | 0.300 | 0.280 | **0.343** |
| Dissolved 0-$PO_4$ (mg/l as P) | 0.231 | 0.182 | 0.011 | 0.016 | 0.013 | 0.007 | 0.149 | 0.007 | 0.011 | 0.011 | 0.017 |
| Arsenic (ug/l) | <2 | 2 9 | 2 | <2 | <2 | <2 | <2 | <2 | <2 | 2.7 | 2 |
| Cadmium (ug/l) | 0.48 | 3.1 | 0 4 | <0.1 | **0.3** | 0 1 | *0.5* | **0.3** | <0 1 | <0.1 | <0.1 |
| Chromium (ug/l) | <2 | 3 3 | <2 | 2 | **2.1** | <2 | <2 | <2 | <2 | 2.4 | 3.1 |
| Capper (ug/l) | <10 | 20.5 | <10 | <10 | 18.8 | <10 | <10 | <10 | <10 | <10 | <10 |
| Lead (ug/l) | <3 | 5.6 | <3 | <3 | 3.3 | <3 | <3 | <3 | 3.3 | <3 | <3 |
| Nickel (ug/l) | <10 | <10 | <10 | <10 | <10 | <10 | 17.3 | <10 | <10 | 13.2 | <10 |
| Selenium (ug/l) | <2 | 2.5 | <2 | <2 | <2 | <2 | <2 | <2 | <2 | 2 | <2 |
| Silver (ug/l) | <2 | <2 | <2 | <2 | <2 | <2 | <2 | <2 | <2 | <2 | <2 |
| Zinc (ug/l) | 24 | 78.8 | <10 | <10 | 12.3 | 11.9 | 27.8 | <10 | <10 | <10 | <10 |

NT = not tested

# STORMWATER RETROFIT OF THE ABANDONED JAN-PHYL WASTEWATER TREATMENT PLANT SITE

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## ABSTRACT

The Jan Phyl Stormwater project was completed in January 1998 to retrofit the abandoned wastewater treatment plant site to provide treatment of stormwater runoff through nutrient and sediment removal. The project also created storage volume to reduce localized flooding for a 90 acre portion of the watershed of the Winter Haven Chain of Lakes which is a SWIM Waterbody. Of the seven acre total project area, four acres were utilized for stormwater treatment. The existing wastewater percolation ponds were retrofitted as wet detention ponds to provide stormwater treatment of the runoff from the first 1.25 inches of rainfall, Sediment excavated from the existing percolation ponds was tested for Fecal Coliform, nutrients and Toxic Contaminant Leaching Potential (TCLP) to verify the material met the criteria established under Chapter **17-640** of the Florida Administrative Code (FAC) for the disposal of waste water residuals. It was originally estimated that nine tons of Total Nitrogen and six and one-half tons of Total Phosphorus was removed with the sediment from this site and disposed of in accordance with FDEP approval. Over 25,000 aquatic plants were placed in the littoral zone. The remaining three acres of property have been sodded to allow for passive recreation and to educate visitors through the use of signs depicting native fish, water fowl and aquatic vegetation. Water quality monitoring is being performed to determine the pollutant load reductions achieved at this facility.

## INTRODUCTION

The Jan Phyl retrofit project was developed as an innovative way to solve a local flooding problem. Stormwater runoff from the adjacent residential neighborhood discharged to a ditch on the project site property boundary with no treatment being provided. Abandoned ponds from a County operated wastewater treatment plant (WWTP) remained following the relocation of the sanitary sewer facilities. The engineering **firm** of Bromwell & Carrier Inc. (BCI) was hired to design a stormwater treatment facility with sufficient capacity to reduce flooding of County roads. The result was a system which also addressed water quality improvements which otherwise would not have been considered.

The project entailed removal of wastewater sediments which had accumulated over the 30 years the sewage treatment plant was in operation. Over **8,800** cubic yards of material was removed from the pond area for disposal off-site. Removal of the abandoned plant superstructure and renovation of the ponds allowed for treatment of stormwater prior to being discharged to Lake Howard. The pond bottoms were re-contoured to provide littoral shelves for planting wetland vegetation for nutrient removal. More than 25,000 herbaceous wetland plants were placed at the site. **A** mitigation monitoring program was established to verify the success of the wetland system.

Stormwater monitoring was initiated following establishment of the littoral vegetation. Automatic samplers are used to collect flow weighted composite samples during rain events. Stations were designated immediately upstream and downstream of the ponds to obtain untreated and treated samples, respectively. Additional grab samples are collected from the downstream sampler at regular intervals to determine the recovery period for the ponds. Base flow samples were collected at the start of the monitoring program to identify pollutant contributions from ground water sources. The results are to be used to determine the pollutant removal efficiency of the system.

Federal matching funds were obtained through a Section **3**19 grant administered by the Florida Department of Environmental Protection (FDEP). Additional funding was provided by the Southwest Florida Water Management District (SWFWMD) through the Surface Water Improvement and Management (SWIM) program as Lake Howard is a designated SWIM water body. The Polk County Board of County Commissioners funded 50% of the $368,565.14 total project cost with the remaining 50% being matched by the SWFWMD and the FDEP.

## Project Team / Objectives

The Jan Phyl project was initiated through discussions between the County's Natural Resources Division, the Drainage Division and the local FDEP representative. The Natural Resources Division managed the contracts for matching funds with the SWFWMD and the FDEP, and hired the firm of BCI for design and construction supervision. The primary objective was to solve the problem of local flooding. Utilization of existing ponds from the abandoned wastewater plant allowed for this to be accomplished while also addressing water quality improvements to the Winter Haven Chain of Lakes.

## Existing Conditions

Jan Phyl Village was developed in the late 1950's **as** a residential community. It is located in Polk County just west of the City of Winter Haven, The seven acre Jan Phyl wastewater plant site was fenced and had remained abandoned following construction of the County's Central Regional wastewater facility. Portions of the concrete sludge digester unit remained along with iron pipe and other material. Vegetation consisted primarily of Bahia grass which stabilized the pond berms with some exotic hardwoods mixed with the few pine trees on the site.

A perimeter ditch along the west and south sides of the property conveys stormwater runoff from the 90 acre watershed approximately one mile to Lake Howard. Drainage from the adjacent residential neighborhood discharged east to this ditch without receiving any treatment, contributing a significant amount of sediment to the storm sewer system. Tail water effects from the ditch system resulted in frequent flooding of roads.

The wastewater treatment plant operated until 1989, discharging effluent to three percolation ponds on the seven acre site. Samples were collected from the sediment within the ponds and determined to meet the domestic Wastewater residuals criteria for land disposal under Chapter 62-640 **FAC.**

## Design & Permitting

Attenuation of peak flood levels and a reduction in the duration of roadway flooding was achieved through the improvements to the existing storm sewer system, and the additional storage designed into the 3 acres of pond area. The original three pond design was modified through interconnection. **A** 24 inch outlet was installed in the first pond (Pond **A),** on the end opposite the inlet structure, to connect the second and third ponds which were combined into a single, larger pond (Pond B). **A** diversion structure was installed at the end of the 48 inch storm sewer pipe to direct flow to the Pond A through two 24 inch pipes. **A** third 24 inch pipe was installed with the invert one foot above the other two pipes so that any additional flow would be discharged directly to Pond B. The diversion structure was designed with an overflow one foot above the top of this pipe to allow peak flows to be routed to the perimeter ditch. The top of the pond berms included two feet of freeboard as a safety factor to avoid over topping. Figure 1 shows the "as-built" plan of the treatment ponds prepared by BCI.

Retrofitting the storm sewer system provided the opportunity to address water quality, The facility was designed to provide treatment of stormwater runoff from the first 1.25 inches of rainfall over the entire contributing basin. The original berm between the second and third ponds was lowered to provide additional littoral zone area. An outlet structure was located on the opposite end of the newly formed Pond B to provide the maximum detention time for treatment. This outlet consists of an eight inch pipe which acts as a bleed down for the ponds. **A** cap with a four inch orifice was installed on the pipe to control the discharge rate so that the required 120 hour detention time could be achieved. Littoral shelves were designed on a 8:1 horizontal:vertical slope. Stormwater treatment was to be provided using herbaceous vegetation at various depths along the littoral zone,

Permitting for this project followed the standard Environmental Resource Permit (ERP) application procedure. Application was made to the SWFWMD in March **1997** with the final permit being issued in June. **A** Notice of Intent (NOI) to comply with the National Pollutant Discharge Elimination System (NPDES) general permit for construction was completed. The NOI was submitted to the U.S. Environmental Protection Agency (EPA) prior to the initiation of construction. A Stormwater Pollution Prevention Plan ($SWP^3$) was prepared for the project according to federal requirements.

## Project Construction

Construction began in August 1997 with the initial stages involving demolition of the remaining WWTP components. **A** large portion of the concrete sludge digester had to be crushed and removed along with cast iron sewer pipe that was scattered around the site. Erosion controls were placed along the perimeter ditch to reduce off-site sediment transport. De-watering of the ponds was required to remove the sludge material which remained from the WWTP operation. The original intent was to utilize this material on the pond berms to provide substrate for the sod which was needed to stabilize the side slopes. Limited working area prohibited stockpiling of the material and removal off-site became necessary. The material was pushed into earthen cells for drying prior to loading for disposal.

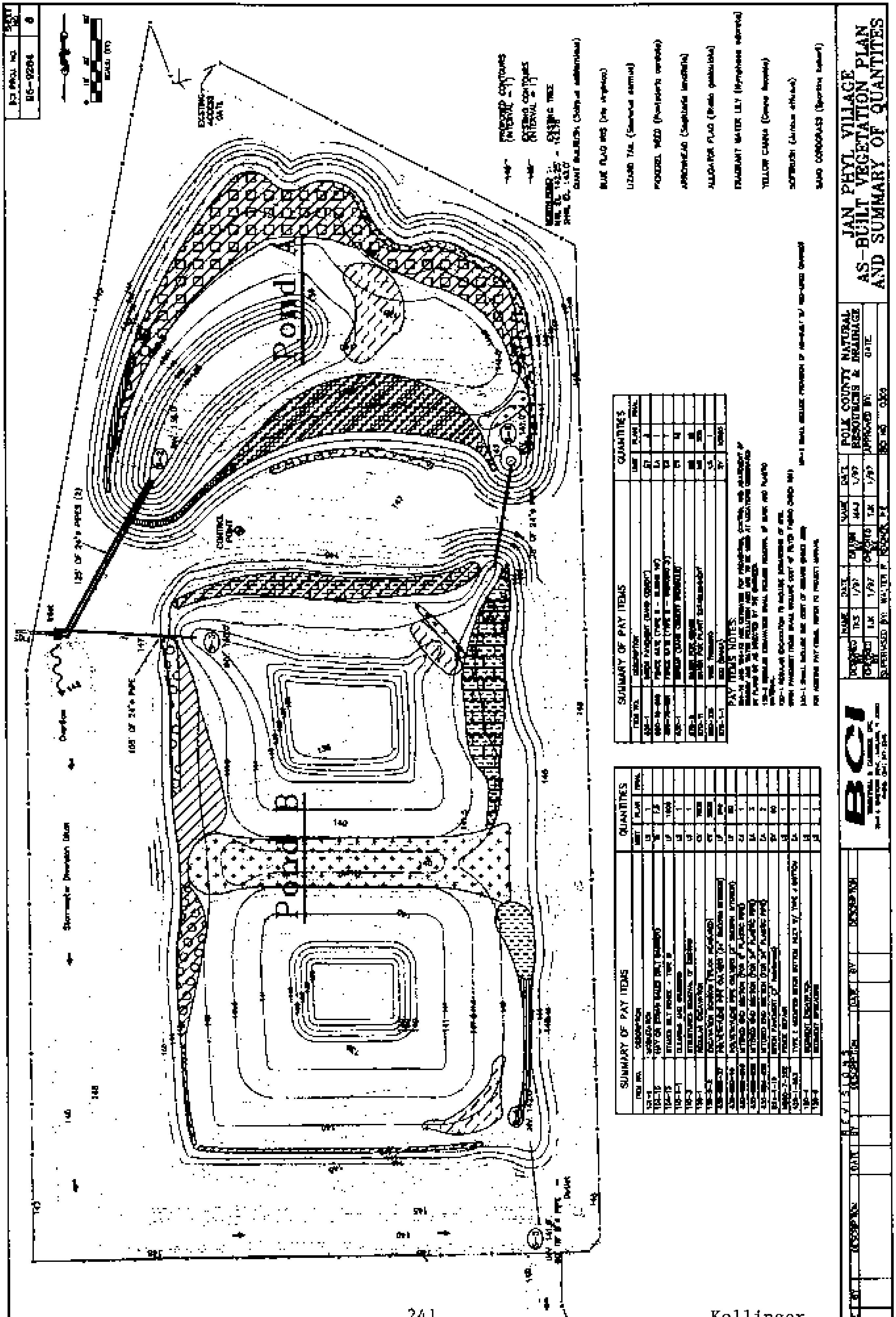


Figure 1 - As-built plan of treatment ponds.

TCLP testing of the bottom material was required by FDEP so the material could be disposed of by land application. Results of the analysis for heavy metals indicated the material met the disposal criteria for Class AA wastewater residuals. Chapter 62-640 FAC requires stabilization of the material for pathogen reduction in order to meet the Class **AA** standards. Since the ponds had been inactive for eight years, the County opted not provide further stabilization. The material therefore needed to be land applied onto a site with limited public access.

**A** summary of the analytical results for heavy metals is provided below for comparison with the regulatory standard:

| **Parameter Analyzed** | **Class AA Standard** Chapter 62-640-(mg/kg dry wt.) | **Analytical Results** Average Conc. (mg/kg dry wt.) |
|---|---|---|
| Cadmium | < 30 | < 0.5 |
| Copper | < 900 | **89.9** |
| Lead | < 1000 | 10.3 |
| Nickel | < 100 | **3.4** |
| Zinc | < 1800 | 282.3 |

Two sites were evaluated for disposal subject to FDEP approval. The first location was the County's North Central Landfill, approximately *6* miles from the project. The second site was the Polk County Skeet and Trap Club located 1.5 miles from the project site. This property is owned by the County and leased to the skeet and trap club. The property was originally used as a borrow pit for road construction materials. It was later operated as a solid waste disposal facility by the County until the early 1970's. Field investigations showed this second location to be acceptable and the site was selected for land disposal of residuals. The shot fall zone for the skeet & trap range was essentially bare of suitable vegetation. Exposed soils and debris covered an area of approximately 9.7 acres.

Chapter 62-640 FAC specifies loadings for nitrogen as well as metals which are land applied. The application rate of nitrogen for Bermuda grass is 250 lbs./acre/year. Based on the nitrogen concentration in the residuals, and the estimated volume of material to be removed, it was calculated that a total of 1850 Ibs. of nitrogen would be applied, This equates to a loading of 190 lbs./acre, which is within the regulatory requirement. Calculation of the metals loading confirmed the amount of material applied was below the maximum allowable cumulative total required by the rule. The FDEP agreed to the suitability of the site and approved the request for land application of the material.

The pond sediment was applied by dump truck and spread using a bull dozer at a depth of approximately **8** inches. Over 8,800 cubic yards of sediment were actually removed from the site with an average solids content of **28%.** The application area was contoured and provided with an earthen curb to reduce the potential for runoff to an adjacent wetland area during storm events. Recruitment of Bermuda grass from the surrounding field minimized the need for seeding and a permanent ground cover was established naturally. An adjustment in the calculation for nitrogen loading using the geometric mean of the sample results revealed that a total of 4.5 tons of total nitrogen was actually removed from the ponds and made available for use in establishing this ground cover.

It was interesting to note that the maximum amount of lead which was land applied is insignificant in comparison to the lead shot deposited during a single shoot by the club. In fact, lead shot is readily visible on the ground surface in the shot fall area. During an average club event with 150 entrants, 100 rounds of 1 oz. loads per day may be shot by each participant. Over a three day period this results in 2,800 lbs. of lead shot being deposited. The amount of lead deposited with the pond sediment was calculated at 80 pounds.

Pond construction commenced with the sediment removal. Heavy equipment was used to re-contour the three ponds to the design elevations specified for Ponds **A** and B. **A** nine foot deep sump was created at the inflow to Pond **A** to provide for particle settling and provides access for sediment removal during maintenance. The side slopes were set at a **4:**1 slope from top of berm down to the water's edge in accordance with the SWFWMD rules since the area is open to the public. **A** shortage in available fill from calculated cuts however, resulted in less littoral shelf being established in Pond B. The shortage of fill required the use of borrow material from off-site. This created a separate problem in that the clay content of the source material resulted in excessive turbidity when Pond B was filled. The problem was solved initially with the application of aluminum sulfate (alum) which allowed for flocculation of solids which settled, clearing the water. **A** long term solution to this problem is expected from stabilization of the pond with the establishment of the vegetative cover.

Ten different species of wetland plants were installed following completion of the ponds. Common names for these plants include: giant bulrush, blue flag iris, lizard tail, pickerel weed, arrowhead, alligator flag, fragrant water lily, yellow canna, soft rush, and sand cordgrass. The survival rate during the first year of operation varied by plant species. Replacements in subsequent replanting were done according to species viability. The giant bulrush installed bare root on the submerged berm in the center of Pond B did not survive. This **was** apparently due to the higher clay content of the material used in construction of the berm as the species did establish in deeper water at the outlet of Pond **A.**

The project site was fenced and gated to allow limited public access for passive recreational activities including walking, jogging and fishing. Kiosks were erected at four locations on the project site to provide information to the public. One display explains graphically how stormwater is treated at the facility. The other three displays identify the fish, birds and types of vegetation commonly associated with wetland areas, Project construction was complete as of the end of January 1998.

## Wetland Mitigation

**A** total of **2** ½ acres of herbaceous wetland from this site was subsequently identified as a mitigation offset for the Lake Deeson water level control project. Florida Permitting, Inc. was hired by the County for semi-annual monitoring and quarterly maintenance of the wetland vegetation. **A** supplemental planting was completed in March 1999 in which the giant bulrush (*Scirpus californicus*) lost from the submerged berm in Pond B was replaced with a smaller bulrush (*Scirpus validus)*. Fully rooted plants in **4** inch pots were used rather than bare root to offset the effects of the clay substrate. Spatterdock (*Nuphar lutem*) was planted along the outer edges of this berm and is doing very well. The supplemental planting also included replanting of soft rush, canna lily arrowhead and the introduction of prairie iris.

Pond A has been most successful with the overabundance of pickerelweed which has established. An inspection of the site following completion of the first mitigation report indicates this pond meets the criteria for release from monitoring.

**Project Costs**

This project was completed at a total cost of $368,565.14 which included engineering design, permitting, construction and public education. Construction costs of $288,139.30 were paid to the site contractor, Royal Construction, Inc. of Tampa. Engineering design and project management fees in the amount of $21,500.00 were paid to BCI. **As** a cooperatively funded project, the SWFWMD permit application fees were waived and $2,775.84 was paid by the FDEP for construction of the public education kiosks. The remainder of the costs include the assessed value of the property and additional materials provided by the Polk County Board of County Commissioners.

The cost of maintenance and monitoring of the site will continue to be incurred by the County throughout the life of the project, Mowing of the grassed berm area is performed twice per month during the wet season in order to allow access to the site by the public. Quarterly inspections of the area are performed under contract and semi-annual reports are prepared to document the success of the mitigation project. Maintenance is performed as required to control invasive species, and desirable species are replanted as needed. In April 1999 an additional $5,374.00 was spent to replace plants to meet the mitigation density criteria.

**Stormwater Monitoring**

Sample collection began in February 1999 to monitor water quality for determination of the treatment efficiency of the ponds. An American Sigma Model 900 automatic sampler was installed at the 48 inch storm sewer system outfall from Jan Phyl to the project to monitor the runoff from the adjoining residential neighborhood. **A** second Sigma sampler was stationed at the 8 inch outfall from the ponds to the perimeter ditch to determine the quality of treated water which is discharged from the site. Velocity sensors were provided in order to determine flow for calculation of the volume of water treated.

Samples at the inflow and outflow from the project site are collected on a flow weighted basis during the first three hours following initiation of storm events with an intensity greater than 0.1 inches in 30 minutes. Follow-up grab samples are collected at the pond discharge at regular intervals to determine the average time for the ponds to return to base flow levels in terms of water quality. The intervals chosen initially are the 12, 24, and 48 hour periods following the onset of the qualifying storm event. These intervals were selected based on previous experience with the Derby Avenue stormwater treatment pond system previously monitored by the County. Table I provides a summary of the results from the events monitored to date. Base flow samples were also collected at the start of the monitoring program to identify pollutant contributions from ground water sources.

Treatment Efficiency of the ponds will be determined through comparison of results of samples collected during the first year of operation. Information from the Derby Avenue project suggests the efficiency is expected to vary depending on the parameter. Removal of nutrients in the 30-50% range, with slightly higher efficiencies of up to **75%** for heavy metals were reported for this project (King 1997). This information was supported by the averages obtained through literature reviews.

## CONCLUSION

It is feasible to retrofit existing storm drainage systems to address water quality with conventional treatment techniques where land is available. Although the cost associated with construction and maintenance of retrofits is significant, there are direct benefits to receiving waters that can be measured in terms of pollutant load reductions. It is difficult to attach a monetary value to the opportunity to see plants and wildlife in an urban neighborhood setting that a project such as this provides.

This project was able to convert idle public property which was previously used to treat domestic wastewater, and allow for passive recreation which was not permitted as a utility site. The project provides an opportunity to educate the public by demonstrating Best Management Practices for proper stormwater management. Efficiency of the treatment system in reducing pollutant loads to Lake Howard will be determined upon completion of the monitoring program,

## ACKNOWLEDGMENTS

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## LITERATURE CITED

King, C. J., 1997. Derby Ditch Water Quality Enhancement Project. Polk County Natural Resources and Drainage Division. Bartow, Florida.

**Table I - Jan Phyl Storm Event Monitoring**

A comparison of analytical results of stormwater runoff at the Project site inlet to levels in the treated water discharged from the outlet.

| Parameter | Baseline | Inlet Initial Rain event 2/2/99 | Outlet Initial Rain event 2/2/99 | Outlet 12 hour Rain event 2/3/99 | Outlet 24 hour Rain event 2/3/99 | Outlet 48 hour Rain event 2/4/99 | Inlet Initial Rain event 4/28/99 | Outlet Initial Rain event 4/28/99 | Outlet 12 hour Rain event 4/29/99 | Outlet 24 hour Rain event 4/29/99 | Outlet 48 hour Rain event 4/30/99 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| BOD (mg/l) | NT | NT | NT | NT | NT | NT | >13.7 | 7.24 | 5.96 | 5.64 | 7.13 |
| COD (mg/l) | 29.5 | 125.2 | 58.7 | 36.7 | 65.1 | 40.1 | 188 | 74 | 66.3 | 63.7 | 68.8 |
| TOC (mg/l) | 9.577 | 17.43 | 12.67 | 9.749 | 12.26 | 10.27 | 13.56 | 13.73 | 12.1 | 12.31 | 12.59 |
| TSS (mg/l) | 4.4 | 160.4 | 30.4 | 15.6 | 29.6 | 25.6 | 279.6 | 50.3 | 37.5 | 84.0 | 76.7 |
| TDS (mg/l) | 168 | 134 | 168 | 160 | 159 | 149 | 71 | 189 | 165 | 168 | 168 |
| $NO_{2+3}$ (mg/l as N) | 0.000 | 0.312 | 0.003 | 0.006 | 0.004 | 0.018 | 0.749 | 0.124 | 0.194 | 0.060 | 0.034 |
| TKN (mg/l as N) | 6.782 | 3.489 | 1.847 | 0.857 | 1.398 | 1.191 | 6.773 | 2.178 | 3.394 | 1.838 | 1.983 |
| Total $PO_4$ (mg/l) | 0.367 | 0.842 | 0.412 | 0.169 | 0.302 | 0.276 | 1.677 | 0.691 | 1.689 | 0.596 | 1.292 |
| Dissolved O-$PO_4$ (mg/l as P) | 0.231 | 0.252 | 0.016 | 0.018 | 0.020 | 0.019 | 0.395 | 0.051 | 0.079 | 0.061 | 0.055 |
| Arsenic (ug/l) | <2 | <1 | <1 | <1 | <1 | <1 | <2 | <2 | 2.1 | <2 | <2 |
| Cadmium (ug/l) | 0.48 | <2 | <2 | <2 | <2 | <2 | 1 | 0.6 | 0.3 | 0.2 | 0.4 |
| Chromium (ug/l) | <2 | <5 | <5 | <5 | <5 | <5 | 14.3 | 3.4 | 3.4 | 2.2 | 8.7 |
| Copper (ug/l) | <10 | <20 | <20 | <20 | <20 | <20 | 40.1 | 12.1 | 14.3 | 13 | 19.1 |
| Lead (ug/l) | <3 | 12.5 | <5 | <5 | <5 | <5 | 33.9 | 4 | 3.5 | <3 | 9.5 |
| Nickel (ug/l) | <10 | <20 | <20 | <20 | <20 | <20 | 17.9 | <10 | 13.7 | <10 | 11.2 |
| Selenium (ug/l) | <2 | 5.6 | 3.9 | <1 | 4.1 | <1 | <2 | <2 | <2 | <2 | <2 |
| Silver (ug/l) | <2 | <5 | <5 | <5 | <5 | <5 | <2 | 2.02 | <2 | <2 | <2 |
| Zinc (ug/l) | 24 | 130 | 22 | 12.3 | 17.4 | <10 | 203.4 | 21.2 | 15.9 | 10 | 49.3 |

NT = not tested

**Table 1- Jan Phyl Storm Event Monitoring** (continued)

**A** comparison of analytical results of stormwater runoff at the Project site inlet to levels in the treated water discharged **from** the outlet.

| Parameter | Baseline | Inlet Imtial Rain event 5/9/99 | Outlet Initial Rain event 5/9/99 | Outlet 12 hour Rain event 5/10/99 | Outlet 24 hour Rain event 5/10/99 | Outlet 48 hour Rain event 5/11/99 | Inlet Initial Rain event 5/12/99 | Outlet Initial Rain event 5/12/99 | Outlet 12 hour Rain event 5/13/91) | Outlet 24 how Rain event 5/13/99 | Outlet 48 hour Rain event 5/14/99 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| BOD (mg/l) | NT | 8.38 | *5.02* | 4.37 | 5.29 | *5.65* | 7.49 | 5.88 | 4.84 | **7.4** | 6 6 |
| COD (mg/l) | 29.5 | 10 3 | 46.3 | 44.6 | 55.8 | **43** | 59.5 | **56.4** | **59.5** | 72.3 | 80.2 |
| TOC (mg/l) | 9.577 | 14.08 | 12.38 | 12 99 | 13.12 | 12.59 | 12.5 | 11.74 | 11.76 | 14.05 | 14.23 |
| TSS (mg/l) | 4.4 | 41 3 | 17.3 | 30.0 | **23.7** | 17.3 | 14.7 | 19.3 | 28.7 | 26.0 | 52.7 |
| TDS (mg/l) | 168 | 144 | 178 | 188 | 190 | 174 | 154 | 186 | 184 | 204 | 208 |
| NO„ (mg/l as N) | *0.000* | 0.911 | 0.001 | 0.014 | tl.O14 | 0.015 | 0.162 | 0.000 | 0.005 | 0.023 | 0.017 |
| TKN (mg/l as N) | 0.782 | 2.585 | 1.691 | 1.805 | **1.313** | 1.254 | 1.366 | 1.357 | 1.415 | 1.462 | 1 559 |
| Total $PO_4$ (mg/l) | 0.367 | 0 561 | 0.263 | 0.505 | **0.244** | 0.186 | 0.348 | **0.239** | 0.300 | 0.280 | **0.343** |
| Dissolved 0-PO, (mg/l as P) | 0.231 | 0.182 | 0.01 1 | 0.016 | 0.013 | 0.007 | 0.149 | 0.007 | 0.011 | 0.011 | 0.017 |
| Arsenic (ug/l) | <2 | 2 9 | 2 | <2 | <2 | <2 | <2 | <2 | <2 | 2.7 | 2 |
| Cadmium (ug/l) | 0.48 | 3.1 | 0 4 | <0.1 | **0.3** | 0 1 | *0.5* | **0.3** | <0 1 | <0.1 | <0.1 |
| Chromium (ug/l) | <2 | 3 3 | <2 | 2 | **2.1** | <2 | <2 | <2 | <2 | 2.4 | 3.1 |
| Capper (ug/l) | <10 | 20.5 | <10 | <10 | 18.8 | <10 | <10 | <10 | <10 | <10 | <10 |
| Lead (ug/l) | <3 | 5.6 | <3 | <3 | 3.3 | <3 | <3 | <3 | 3.3 | <3 | <3 |
| Nickel (ug/l) | <10 | <10 | <10 | <10 | <10 | <10 | 17.3 | <10 | <10 | 13.2 | <10 |
| Selenium (ug/l) | <2 | 2.5 | <2 | <2 | <2 | <2 | <2 | <2 | <2 | 2 | <2 |
| Silver (ug/l) | <2 | <2 | <2 | <2 | <2 | <2 | <2 | <2 | <2 | <2 | <2 |
| Zinc (ug/l) | 24 | 78.8 | <10 | <10 | 12.3 | 1 1.9 | 27.8 | <10 | <10 | <10 | <10 |

NT = not tested

# SUCCESS STORIES IN STORMWATER RETROFITTING

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## ABSTRACT

In 1990 Brevard County created a Stormwater Utility under the Surface Water Improvement Division (SWID) for the purpose of retrofitting stormwater facilities for water quality and quantity benefits. Since that time over 200 retrofit projects have been constructed at a cost of over $11,000,000, This paper will discuss some of the types of projects implemented and the lessons learned.

## INTRODUCTION

Brevard County's Stormwater Utility was one of the first such Utilities in Florida. At the time, there was considerable debate as to what our function was and whether we would be successful. There was no NPDES program, but the environmental movement was in full swing and the main emphasis was for cleaning stormwater runoff to protect the Indian River Lagoon, which is part of the National Estuary Program.

Brevard County is predominantly rural with approximately 2,414 km2 (1,500 square miles) of area. The urbanized sections of the County are concentrated along the coastline and barrier islands. Unlike other sections of Florida where most of the stormwater drains to a relatively small number of lakes or rivers through large collection systems, Brevard County has over 2,000 stormwater outfalls to the Indian River Lagoon, most of which are fairly small and undersized. Very few of these systems have permitted treatment facilities. As with most low-lying coastal communities there are numerous areas which experience frequent flooding due to the extremely flat grades.

The big question was where do we start eating this elephant? The standard engineering approach to this challenge was to perform a master stormwater study which identified projects to be designed and constructed. Consulting engineers gladly gave us estimates of $6,000,000 - $8,000,000 and 2 years to perform this grandiose study. This presented two significant problems: 1) we did not have the funds and 2) if we waited two years for a dusty study to throw on the shelf, our program would not survive.

We knew there were more projects than we could possibly build in 20 years and the emphasis was on immediate construction to win public support of our program. Therefore, we decided to start eating the elephant in small pieces. It was obvious where the impacted areas of the Lagoon were located by the lack of seagrasses and fish, so we began performing small basin studies of .8-8 km2 (200-2000 acres) in these areas and identifying projects to retrofit. We were principally looking at water quality projects but also addressed flooding concerns. As soon as we had projects located, we began design and construction. The first project was built about 1 year after program inception.

The first studies performed by our highly paid consultants identified a number of large-scale projects that ended up not being viable or having marginal benefits. We soon learned not to give consultants open checkbooks; rather we looked very hard at proposed projects during the conceptual stage and provided the engineers with substantial direction in the feasibility analysis.

We also learned that there were many small, simple projects that consulting firms were not real thrilled about designing due to the low costs and fees. Fortunately, we had engineers in the Surface Water Improvement Program who could design these small projects in-house. Constructing a large number of small projects was a way to obtain positive public support in many neighborhoods as well as answer the question of "What am I getting for this new tax?"

We still used consultants to design large and medium sized projects. There is a never ending need for improving existing conveyance systems and solving perennial flooding in Florida. While flooding control was not our main emphasis, it was necessary to solve some of these problems to maintain public and political support. Generally, permitting requirements will mandate ponds for attenuation when making conveyance improvements, so the projects end up providing water quality and quantity improvements.

Our program had the enviable problem of having more money than we could spend the first couple of years. We collected about $3,000,000 a year and about $2,000,000 was available for construction projects. With a staff of one engineer and six people, there was much more of a demand for projects than we could supply.

Once word got out that we had money, every real estate broker in town beat a path to our door to give us a deal on wetlands that they could not develop but wanted to sell to **us** at a premium. We also became the target of other City, County, and State agencies which needed projects funded for their own agendas.

At this point, the Brevard County Board of County Commissioners made a wise decision and determined that Stormwater Utility funds were to be used for Capital Improvements only. These funds were not used to fund normal maintenance activities of other Public Works or County Departments. This prevented the shell game of supplementing general funding shortfalls with Stormwater Utility revenues. These funding games were one of the most persistent and difficult struggles of the program.

Another controversial subject was the splitting of costs between water quality and water quantity projects. Our initial goals were to lean more toward water quality projects, but as different Commissioners came and went and various tropical storms hit, we would get mixed messages for project direction. The Board of County Cornmissioners made another wise decision after Hurricane Erin and established guidelines that our funds were to be split 50-50 for water quality and water quantity projects.

## PROJECTS

Several years of committee meetings were spent in trying to come up with selection criteria and complex matrixes to be used in project selection. After this agony, we inevitably found two factors which determined project selection: 1) availability of land and **2)** public demand for work on their problem! The selection matrices eventually faded away.

After 3-4 years of growing pains we settled into an aggressive program of project construction, The best way to achieve the maximum treatment bang for the buck is with ponds serving large drainage basins. Fortunately, the unincorporated areas of Brevard County are lightly urbanized for the most part and there are quite a few areas where land is available for ponds. Working in conjunction with other County agencies such as Parks and Recreation or Wastewater Department often provided a source of free land. Since the State mandates no sewer disposal to the Indian River Lagoon, most of the sewer plants have shut down and several of those sites have since been converted to stormwater ponds.

The following list highlights some of the projects Brevard County has completed over the last seven years:

## Sea Park Pond

One of our first projects was converting the abandoned Sea Park Sewer Plant into a partnered multi-use facility. This facility consisted of a 8,093 m2 (2 acre) regional detention pond, soccer fields for Parks and Recreation Department, and a new piping system installed in the adjacent road in conjunction with a road reconstruction plan. This pond served a drainage basin of .25 km2 (61 acres) of residential area flowing untreated to the Banana River. **A** small existing pond had been used years ago as a percolation pond for the sewer plant. Although testing showed no contamination in this pond, the State required us to dredge muck out of the pond at a cost of $186,000, This pond was redesigned as a stormwater reuse pond to provide irrigation for the soccer fields. While stormwater reuse was a laudable goal, it was rarely viable due to competition with wastewater reuse mandated by the State. The cost for constructing this project was $361,070.

## Scottsrnoor Masterplan

In 1991 north Brevard County received over 45.7 cm (18") of rainfall in a 5-day period resulting in widespread flooding in the Scottsmoor area. In response to this disaster, a Master Drainage Study was conducted for 84.7 km2 (20,933 acres) of rural land.

The drainage systems in Scottsmoor principally consist of ditches along dirt roads and agricultural ditches built to dewater cropland. Most of this area drains to Class 2 Waters of the Indian River and Mosquito Lagoon. The St. Johns River Water Management District was naturally very concerned about introducing additional flows to these environmentally sensitive waters through upstream conveyance improvements. In order to make necessary upstream improvements, it was necessary to construct downstream detention ponds, which attenuated increased flows and provided stormwater treatment.

The initial analysis was to provide 25 year (22.9 cm or 9" of rainfall) flood protection for these areas, but cost estimates for this level of service were around $12,000,000. Since this was a low-density rural area, it was decided to design improvements for the annual storm (12.7 cm or 5" of rainfall). These proposed improvements consisted of enlarging pipes and ditches and constructing 18 detention ponds at an estimated cost of $6,156,000.

To date, SWID has constructed detention ponds along Johns Road (two ponds of **6,880** m2 or .7 acres and 809 m2 or 0.2 acres), Flounder Creek Road (12,950 m2 or 3.2 acres), and Huntington Avenue (13,759 m2 or 3.4 acres), These ponds served drainage basins totaling 698 hectares (1,725 acres) and removed an estimated 32,728 kg (72,152 lbs.) of pollutants per year. They were constructed at a cost of $948,722. An EPA grant of $94,535 was obtained for the construction of the Huntington Road Pond.

**Port St. John Masterplan**

Port St. John is a medium density residential area adjacent to 'Body C' on the Indian River Lagoon. 'Body ***C'*** is an environmentally sensitive waterbody providing 90% of Florida's harvested clams, one of the highest wintering populations of manatees along Florida's east coast, and large expanses of seagrasses.

In 1993, a Master Stormwater Study was undertaken for the 1.39 **km2** (343 acres) of Port St. John which drain to the Indian River. There were almost no stormwater treatment facilities in this area. Four (4) outfalls were identified with proposed retrofits for stormwater quality benefits. Three **(3)** baffle boxes were soon installed for interim protection while land acquisition was pursued for larger projects. Baffle boxes are sediment trapping devices which will be explained in further detail later in this report.

The first baffle box was constructed at the Sunrise Village Condominiums on a 152.4 cm (60") pipeline and has remained the largest installed by the program. It was downstream of a deep, highly eroded ditch and collected up to 22,680 kg (50,000 lbs). of sediment per month. Due to right-of-way constraints, there was no room to improve the ditch so 408 m of the ditch were piped and a **5,261** m2 (1.3 acre) detention pond was constructed. **A** partnering opportunity presented itself when a hospital approached the County for a joint project. The hospital donated the land for a pond, provided its own pretreatment ponds, and SWID constructed the detention facilities for the hospital and the upstream properties of our study area.

We again partnered with a new Publix store at the second outfall when it came in for development. Publix provided **an** access road across a ditch and we constructed a baffle box in the ditch under the road. We also stabilized the ditch adjacent to their parking lot, working with their proposed fill grades. Investigation of this ditch showed it was deeper than necessary and caused significant groundwater drawdown in the area. The flow lines of 1,000 m of ditch were raised as much as 1 m feet allowing for sideslope reconstruction at much flatter slopes. These flatter side slopes provided for grass stabilization as well as routine mowing for the first time. Erosion from this ditch has been significantly reduced as a result of these improvements.

Along a third ditch in the area, 7 lots were purchased and a **6,880** m2 (1.7) acre detention pond was constructed which allowed for another 433 m of highly eroded ditch to be piped, again reducing significant sediment loadings to the River. Other parts of the ditch were fabriformed for erosion protection where sideslopes were too steep to stabilize.

Monitoring of the baffle box on the fourth outfall showed minimal sediment loadings for that drainage basin so a difficult pond construction project in the Indian River was not pursued. Almost all of the areas in Port St. John draining to the Indian River have been retrofitted with treatment facilities at a cost of $1,489,253.

## Merritt Ridge Alum Treatment Plant

In the Merritt Island Mall area, there are 3 main outfall ditches draining 1.27 km2 (3 14 acres) of shopping centers, residential, and industrial land uses with minimal storm water treatment facilities. The Plumosa ditch has a very restricted outfall causing Plumosa Street and Fortenberry Road to flood with almost every rainfall, A 13-acre tract of land has been acquired and a detention pond with an alum treatment plant has been constructed at this site. In addition, 579 m of double 121.9 cm (48") pipe has been constructed to provide an outfall from the Plumosa ditch to the alum treatment pond.

Again partnering was used effectively on this project. The system was designed to provide for future widening of Fortenberry Road, allowing our Transportation Department to participate in the pond costs. In exchange for easements, 2 private properties with permitted stormwater systems had their permits and ponds removed and the regional pond permit covered these sites. The estimated cost of these facilities is $3,070,000.

## Indialantic Masterplan

In the Indialantic area, a Master Stormwater Study was conducted for 7.28 km2 (1,800 acres) along the barrier island. There were numerous small outfalls to the Indian River in this basin. Combinations of converting a ditch to a pond, 14 baffle boxes, 4 exfiltration systems, and 6 inlet devices have been constructed to provide stormwater quality treatment for 24 outfall pipes. The ditch reconstruction was unique with the use of stair stepped Geoweb to stabilize the slope. This allowed a strong maintenance berm to be built for access and ended continuous erosion problems. The cost for constructing these improvements was $1,008,625, with $280,362 being funded through DEP grants.

## Johnson Jr. High

A rare partnering opportunity was used with the School Board at Johnson Jr. High School. They provided a 20,234 m2 (5 acre) easement to construct a regional pond in return for giving them the excavated dirt to regrade flooding ball fields. The pond also provided a positive outfall for flooding in their building areas. This project was constructed at a cost of $292,921 and served a drainage basin of .275 km2 (68 acres).

## Hurricane Erin

Hurricane Erin hit Brevard County in August 1995 and painfully pinpointed many older areas with inadequate drainage facilities, Since then, two large Master Stormwater Studies for the Crane Creek basin and the Upper Eau Gallie Creek basin have been completed which proposed $10,701,167 of improvements. Implementation of a double 2.4m x 3m (8'x10') box culvert and 213 cm (84") RCP culvert upgrade projects has been completed and more will be constructed over the next several years.

## Treatment Techniques

**As** previously stated, Brevard County has a large number of small drainage basins and outfalls. Several economical, innovative treatment techniques have been developed for these small outfalls. The most successful is the baffle box, a sediment trapping device constructed in-line with existing pipes. It has multiple chambers for sediment trapping and swiveling screens for trash removal. Thirty four of these boxes have been constructed on pipes up to 152.4 cm (60") in diameter. A total of 314,030 kg (692,316 lbs.) of sediment have been cleaned out of these baffle boxes between 1991 and 1998. The average cost of a baffle box is $25,000.

Grated inlet baskets are ***$595*** fiberglass inserts which fit into existing grated inlets. They effectively trap dirt, trash, leaves and debris which flow through the inlet, with no loss of hydraulic capacity.

Curb inlet baskets are expandable fiberglass units designed to fit inside existing curb inlets without head losses, They trap leaves, **grass,** paper, and trash in a removable basket, The cost for one of these devices is ***$695.***

Another sediment trapping device successfully used is the **CDS** unit; a circular box with screens that works on the vortex principal. The CDS unit effectively **traps** about 50% of suspended soils and virtually 100% of floating trash. Brevard County installed the first unit in the United States in August 1997 at a cost of *$55,000.* Since that time 4632 kg (10,213 lbs.) of sediment and 0.96 m3 (34 cubic feet) of trash, leaves, and floating debris have been removed from the unit,

Table 1 shows the cleanout records and costs for these different BMP's. They each have a role in stormwater treatment, depending on the pollutant targeted, size of drainage basin, availability of land, and project budget. Our costs for maintenance of these projects runs about 1% of the budget.

| Type of BMP | Number Installed | Average Weight Cleaned | Average Cost Per Cleaning | Average Cost/kg Sediment Removal | Average Cost/kg TP Removal | Average Cost/kg TN Removal |
|---|---|---|---|---|---|---|
| BB | 31 | 1925 | $450 | $.023 | | |
| CIB | 50 | 4.6 | $3.50 | | ***$3.87*** | $1.51 |
| GIB | ***39*** | 16.3 | $45 | ***$2.76*** | | |
| CDS | 1 | 1544 | $400 | $0.26 | | |

Table 1

Table 2 provides a matrix for BMP selection based upon targeted pollutants and drainage basin size.

| Basin Size | Sediment | Trash | Nutrients from Grass & Leaves | Nutrients from Other Sources |
|---|---|---|---|---|
| Small | GIB, BB, **CDS** | GIB, CIB, CDS | GIB, CIB | Other |
| Medium | GIB, BB, **CDS** | GIB, CIB, BB,CDS | GIB, CIB | Other |
| Large | Other | Other | GIB, CIB | Other |

Table 2

**Public Education**

Another important aspect of the program are the public education efforts. Holding several public meetings during the planning and design stages involves the citizens of the area and reduces resistance to projects and "change" in their neighborhoods. They do not like Big Brother to show up unannounced with backhoes to "save" them.

We also take advantage of these meetings to educate the citizens how they can help in the battle against stormwater pollution. The problem is so large that there will never be enough money to clean all of the runoff once it becomes dirty. Without the help of citizens to keep pollutants out of the water to begin with, the job is hopeless. Simple actions such as not dumping grass clippings, leaves and trash in the gutters, reducing mowing and fertilizer usage, recycling oils and chemicals, and bagging trash are important activities all citizens can participate in.

## CONCLUSION

To date Brevard County's Stormwater Utility has constructed over 200 retrofit projects totaling about $11,000,595. Of these projects, approximately $6,657,020 **was** spent on water quality projects and $4,355,575 on flood control projects. In the next 2 years approximately $5,000,000 of retrofit projects are scheduled for implementation. These projects will make a significant improvement in the water quality of the Indian River Lagoon and will hopefully save it from the fate of many other severely damaged bays and rivers throughout the country. Using a Stormwater Utility has successfully provided a dedicated source of funds for water quality and flooding improvements in Brevard County. This funding source is relatively insulated from the budget cutbacks most other governmental agencies are experiencing allowing for long term planning for many projects,

# ESTIMATING SOIL STORAGE CAPACITY FOR STORMWATER MODELING APPLICATIONS

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## ABSTRACT

The capacity of surface soil layers to store infiltrated stormwater becomes **an** important parameter in many practical situations (e.g., when the water table reaches the ground surface, which is a common occurrence throughout Florida). Current methods for estimating soil storage capacity are often based on very general soil characteristics. For instance, the Natural Resources Conservation Service **(NRCS,** formerly the Soil Conservation Service, SCS) curve number method uses a predetermined soil storage volume based on the assigned hydrologic soil group. Since soil storage may be more dependent on other variables (e.g., depth *to* groundwater table and fillable porosity), some methods currently used to determine soil storage capacity might not be appropriate for certain areas or applications.

This paper presents a method for estimating the storage capacity of specific soil types using data that are readily available in tabular and digital format. This method is applied to NRCS **Soil Survey** data for various counties in Central and Northwestern Florida. Finally, results are compared with other methods for a case study in Polk County, Florida.

# HARNESSING THE POWER OF MICROSOFT ACCESS FOR THE MANAGEMENT OF NPDES PERMIT COMPLIANCE DATA IN A MULTI-PERMITTEE SCENARIO

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## ABSTRACT

Full implementation of the United States Environmental Protection Agency's (EPA) NPDES **MS4** permit program imposes extensive compliance and reporting requirements on municipalities. Typical requirements include development, revision, and implementation of a comprehensive Storm Water Management Program (SWMP). **An** overall Annual Report needs to be submitted to EPA, which qualitatively and quantitatively describes the specific task accomplishments and compliance status of each permittee.

EPA is sued a MS4 Permit to Pinellas County and **22** co-permittees, effective November 1,1997. Pinellas County contracted PBS&J to develop a user-friendly and comprehensive data management system to collect, compile and summarize permit compliance data from all the 23 co-permittees and generate the annual **summary** report for submission to EPA.

In order to facilitate the collection, analysis and compilation of a vast amount of data from these permittees, Pinellas County contracted PBS&J to develop a user-friendly and comprehensive data management system to perform this challenging task. Though this task appears to be simple in concept, implementation was complicated due to these factors: the portions of permit applicable for each co-permittee vary significantly; the type of data to be collected and reported varied for each co-permittee; no common data collection, storage or analysis methodology existed among the permittees at the time of this project; and the GIS/Database system of the County is not used by several of the co-permittees.

PBS&J successfully accomplished this task through a team of MS4 Permit Experts, GIS Analysts, and Database Designers in conjunction with the staff from all the co-permittees. The Pinellas County NPDES Permit Tracking System was developed as a flexible stand-alone application in Microsoft Access, with a simple and intuitive graphical user interface that even an inexperienced computer user could use with minimal training. Each co-permittee department has the ability to specify their own performance measures without affecting the performance of other departments or co-permittees. Generation of annual summary reports is as simple as a click of a button. The database was designed for use in both a multi-user network environment and a single-user desktop setting, and can be expanded to incorporate spatial intelligence using GIS. This project is a demonstration of the power of applying the new information technology tools in conjunction with specialized functional knowledge of MS4 permits to simplify an otherwise daunting task of tracking NPDES permit compliance activities.

## INTRODUCTION

In response to the need for comprehensive National Pollutant Discharge Elimination System (NPDES) requirements for discharge of storm water, Congress amended the Clean Water Act in 1987 to require the U.S. EPA to establish phased NPDES requirements for storm water discharges. To implement these requirements, EPA published the initial permit application requirements in November 1990 for certain categories of storm water discharges associated with industrial activity and discharges from municipal separate storm sewer systems serving populations of 100,000 or more. Municipal categories were classified as medium or large if they serve populations greater than 100,000 or more and 250,000 or more respectively. Applications for these permits were submitted by large and medium municipalities by November 1992 and May 1993 respectively. Many permits have been issued to date throughout the country.

Full implementation of the NPDES MS4 permit program has imposed extensive compliance and reporting requirements on municipalities throughout the country. Requirements for a typical MS4 permit include the development, revision, and implementation of a comprehensive Storm Water Management Program (SWMP) including pollution prevention measures, treatment or removal techniques, storm water monitoring, use of legal authority and other appropriate means to control the quality of storm water discharge from the MS4.

Pinellas County and 22 co-permittees were issued an MS4 permit, which became effective November 1,1997. Part V of the MS4 Permit requires submission of an overall Annual Report at the end of each permit year, which describes in both narrative and quantitative terms, the task accomplishments and compliance status of each permittee with reference to permit requirements. Summarizing the permit activities and preparing an annual summary report is a challenging task even in a single permittee scenario. With the need to summarize and report the activities of 23 different permittees, the complexity of this task increased many folds. Some of the factors that made this data compilation task more challenging were:

- The applicable permit parts were different for each co-permittee
- The type of data to be collected for each co-permittee varied in terms of what is reported and how the data was tracked
- There was no common approach for data collection, storage and analysis among the co-permittees
- The County's preferred database system was not used by other co-permittees
- The resources available for the MS4 permit compliance activities were significantly different among the co-permittees, and
- A suitable application was to be developed on a short notice due to the time constraints for submission of annual report

The NPDES project managers for Pinellas county and the co-permittees were knowledgeable of the complexity of this task and contracted PBS&J to develop a simple, straight forward method to collect, analyze, and summarize the data related to the MS4 permit compliance, and to automatically generate the Annual Report from the collected data. The County staff also had a vision to expand the NPDES data management system in the future to automate data transfer and data exchange operations between this system and the County's Maximo Work Management System, Oracle system and the GIS system.

**Technical Approach**

The overall objective of this project was study the **MS4** permit requirements and develop a comprehensive and user-friendly data management system to collect, compile and summarize the permit compliance activities from all the permittees and their departments. The product of this data management system is the Annual Summary Report to be submitted to the **EPA.** In order to best meet the requirements of the County and the Co-Permittees, PBS&J developed a two-phased approach for this project.

- Phase I - Application Development in MS Access: In this phase a custom relational database application was developed in Microsoft Access with standard data input screens for all users and a standard report module for all users to automatically generate Annual Summary Reports from the input data,

- Phase II – Automated Input Routines for the County: This is a proposed phase in which a Bi-Directional Interface would be developed between the County's database systems (Maximo, Oracle, GIS) and the NPDES Application developed in Phase I, to facilitate seamless data exchange and eliminate duplicate data entry operations.

This paper discusses the design and development of the NPDES Permit Management System implemented to meet the Phase-I requirements discussed above.

**Relational Database Design Fundamentals**

We are constantly dealing with different types of ***data*** in our daily life, Data is everywhere, but data is not ***information.*** Information is data that is organized in a meaningful form with a well-defined structure. Good data management provides the structure necessary for transforming a maze of data into information, **A** relational database is composed of a number of data tables related to each other through common fields. This facilitates in searching for information across several tables efficiently, economically, and accurately. This makes the data more accessible, easy to maintain, update and use, **A** relational database management system (***RDBMS***) is a collection of programs that enables users to create and maintain a relational database (Simpson and Olson, **1997).**

Prior to the information revolution, such database management needed a high level of computer knowledge and programming skills. Engineers seldom had such level of skills and therefore had to resort to traditional data management methods, The advent of Windows-based database software with simple Graphical User Interfaces (GUI) virtually eliminated the need of programming knowledge to harness the power of database systems. Database systems also facilitate the implementation of a security protocol for data access.

We all have our own data management systems in place (predominantly spreadsheets). Though they may seem to work fine at the individual level or within small groups, they are likely to be corrupt the data when multiple users start managing the same data. We end up in situations where we have multiple copies of the same information and have difficulty in identifying the latest and most accurate data. In contrast to spreadsheets, relational database systems are easy-to-use tools for setting up a good data management practice. Using a RDBMS, we can quickly create queries to perform tasks that would have been very complicated to do with spreadsheets and generate a high quality report to summarize your analysis, The number of records (rows) one can have in a file is also a major limitation of spreadsheets. Lotus 1-2-3 (Release **5)** allows **8192** rows and Excel **97** allows 65,536 rows. So if we have large data sets like historical rainfall data or lake levels data or canal stage data, we now have the capability of using databases. We at PBS&J successfully used Access for data sets up to 10 million records.

Microsoft Access is the most popular desktop database in the market today. It is a part of the MS Office Professional Edition. It is easy to use yet powerful enough to dramatically improve our traditional data management systems. Due to its popularity, simplicity of use and its capabilities, **MS** Access was chosen as the RDBMS environment for developing the NPDES Permit Tracking System.

### Setting up the Database Design

Any RDBMS is only as good as the design of the underlying tables and their relationships. In order to develop a good database design, it is essential to have a development team that has sound, functional knowledge of the problem as well as good database software designers. It is also imperative to discuss the needs of the clients in detail and get their approval prior to embarking on the design process (Elmasri and Navathe, 1994).

PBS&J assembled a team of MS4 Permit Experts, Storm Water Engineers, GIS Analysts, and Database Designers in order to develop the database design for the NPDES Permit Management System. Meetings were held with the responsible staff from all the **23** co-permittees. The MS4 permit experts studied the permit in great detail to understand the key items and requirements. **A** typical MS4 permit consists of nine (9) major program elements:

- Structural Controls and Stormwater Collection System Operation
- Areas of New Development and Significant Redevelopment
- Roadways
- Flood Control Projects
- Municipal Waste Treatment, Storage or disposal facilities not Covered by an NPDES Storm Water Permit
- Pesticide, Herbicide, and Fertilizer Application
- Illicit Discharges and Improper Disposal
- Industrial and High Risk Runoff
- Construction Site Runoff

Each of the program elements requires a set of tasks to be performed in order to achieve compliance. Each such task was assigned a task number and the activities required to be performed under each task to achieve compliance were outlined by the **MS4** experts. These activities were designated as performance measures and the activities performed by each co-permittee under each performance measure were to be summarized accordingly. The performance measures were grouped under four major categories namely date, inventory, project and compliance status. Each of these groups is briefly discussed below.

***Date Dependent Activities:*** Date dependent activities are those actions that need to be completed by a specific date provided within the NPDES permit. These are actions such as completing a report by the end of the first permit year or implementing a specific program within 24 months of the effective date of the permit.

***Inventory Driven Activities:*** Inventory driven activities are those activities for which a count will be provided in the annual report to **EPA.** These are activities such as screening a percentage of your total outfalls for potential pollutants or recording the number of public education activities provided. The system continues to keep track of all data entries so that at any point in time the user can see the total number reported to date prior to entering new records.

***Project Related Activities:*** Project related activities are those activities where the compliance action is tied directly to individual projects and is managed on a project by project basis. These are such activities as keeping track of new development activities or making sure new projects comply with applicable best management practices as outlined in the NPDES permit document. All individual data entries are maintained within the system and the annual report is designed to generate a summary table showing how many projects were reviewed and found to be in compliance and how many were reviewed and found not to be in compliance.

***Compliance Status Activities:*** Compliance status check activities are those activities for which a simply yes or no answer is sufficient to satisfy the action required by the NPDES permit. These are such activities as maintaining internal records or form a committee. The status of compliance may change throughout the permit year. **As** the compliance status changes and that information is recorded within the system, all individual entries will be maintained, however, the last change will be the status that is recorded within the **annual** report.

MS4 permit requirements were broken down into simple tasks. Each task was provided with a list of suggested performance measures, which would help in ensuring permit compliance. This formed the basis for the development of **a** database system in Microsoft Access environment to track all the compliance actions. The database was designed for use in both a multi-user network environment and a single-user desktop setting. The relationship between each of these database elements is depicted in Figure 1.

## Development of Graphical User Interface

In order for the RDBMS to be utilized effectively by Pinellas County and the co-permittees it had to be created with **an** intuitive and user friendly graphical interface, It was the goal of this program to ensure ease of use due to the varied computer experience of many of the co-permittees. To accomplish ease of use, one of the most important considerations is the logical flow of information review and input. Additionally, no one form can contain more information than **a** typical user can digest quickly.

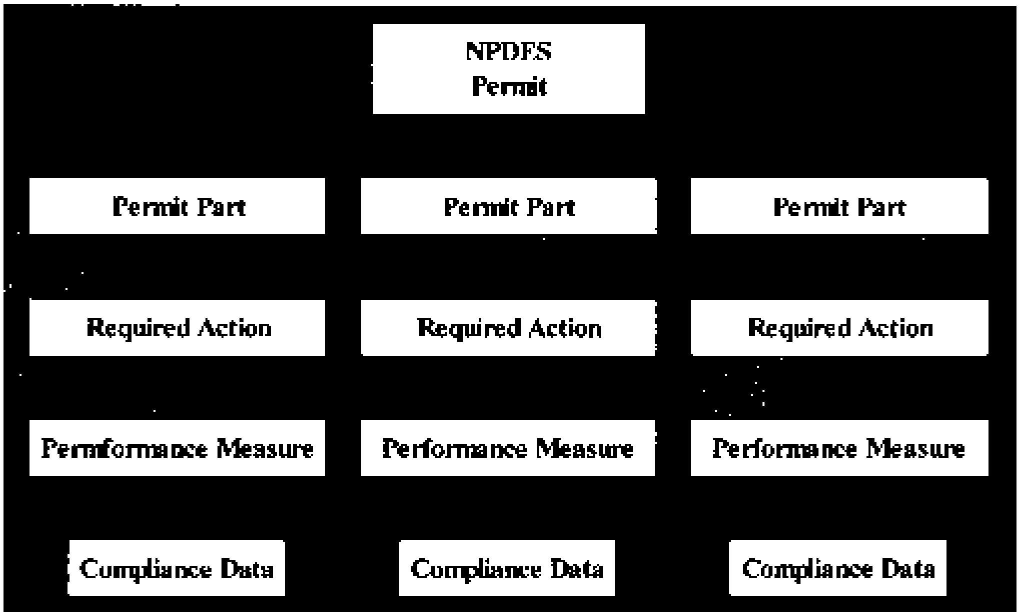



**Figure 1. Database Relationships**

Additional considerations included the ability to incorporate security measures. Initial login screens were developed to allow the user to choose their appropriate group and input a password before having access to any data entry screens. These forms were developed with consistent look and feel as the data entry forms to give the user a consistent interface from which to work.

Finally, the creation of the annual report documents needed to be accessed through these forms, The users needed to be provided flexibility to modify the reporting period. Custom coding was implemented in order to provide this flexibility while still preventing the user to modify the report format and design. Sample log-in, data entry and report creation screens are presented in Figure **2** to illustrate the easy-to-use visual interface developed for the NPDES Permit Management System.

## Database Usage

Database usage follows a logical progression. The user logs into the database by choosing their appropriate group and entering the approved password. Based upon who logs in, the system automatically sub-sets the permit requirements based upon guidelines established at each participating municipality, The user can immediately upon entering the system review these overall requirements for which they have responsibility, generate *summary* report documents, or begin to add or edit data. If the user chooses to add or edit data they are provided the option to choose the particular required action for which they would like to enter data. Once the compliance data has been entered into the system, many different report formats are available for permit managers to review the data at various levels of **summary.** Certain users have additional access to modify the performance measures for required actions in order to better accommodate their business practices. The user navigation flow chart is documented in Figure 3.

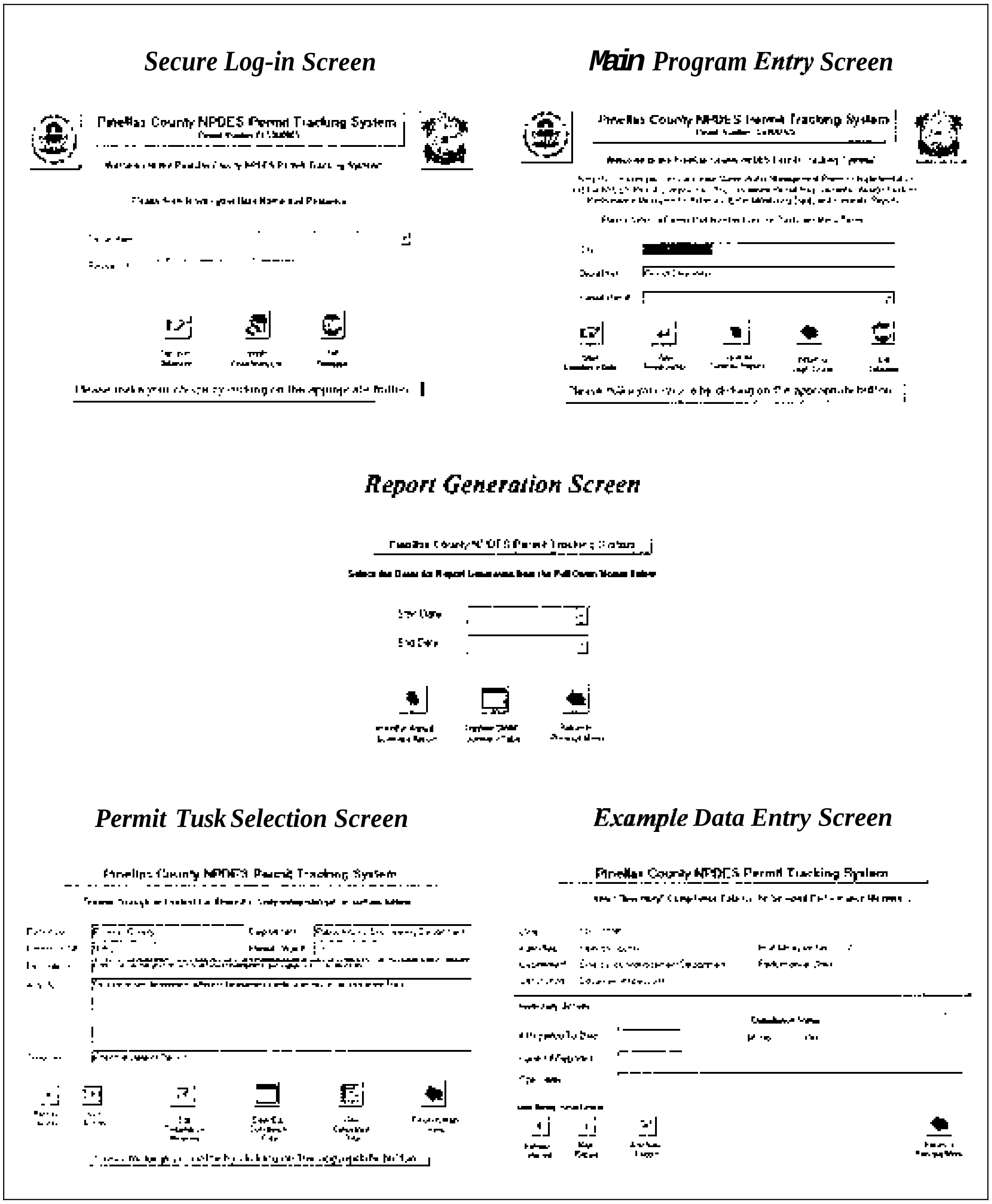
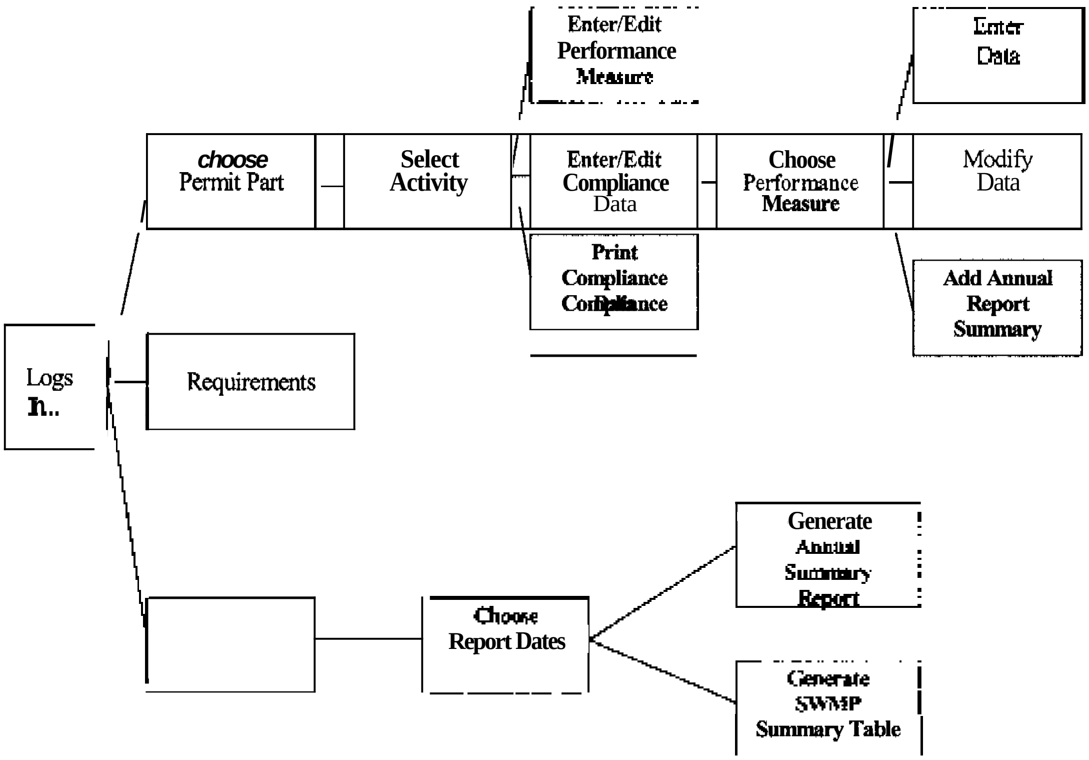



**Figure 2. Database User Interface Screens**




## CONCLUSIONS

This database system is emerging as a valuable tool for the County's NPDES Program Coordinator who has the onerous task of preparing the Annual Permit Compliance Summary Report. The database can be expanded in the future to incorporate spatial intelligence using GIS or can be integrated with other County databases. This project is a demonstration of the power of applying state-of the art information technology tools in conjunction with specialized functional knowledge of MS4 permits to simplify an otherwise daunting task of tracking permit compliance activities of 23 different co-permittees and their departments. It is imperative for the civil engineers of the next millennium to be aware of the new developments in information technology and be able to harness the power of the new software tools. This will result in developing innovative and more efficient solutions to many of our project tasks.

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# HYDRAULIC PERFORMANCE OF A NEW STORMWATER CONTROL STRUCTURE

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## ABSTRACT

This paper details a performance analysis of a new outlet stormwater control structure proposed for use by The Florida Department of Transportation. This attenuator comprises a skimmer and weir, enclosed in standard precast elements to protect against mowing accidents and vandalism. An experimental study using a one-quarter scale physical model was performed in a simulated detention facility. The results of this investigation have been reduced to an overall discharge coefficient as a means of predicting full scale performance for design purposes.

## INTRODUCTION

Management of stormwater runoff is an important issue impacting satisfactory pavement drainage. The typical stormwater pond outfall structure is a precast box with an opening in the side to release water at a controlled rate. These structures are installed out from the edge of the pond, near the toe of the berm. The boxes are difficult to mow around and are hard to access due to elevation from the berm slope. Often the structures are so tall that a ladder is required. **As** a result of location, shape, and maintenance difficulties the structures are not usually aesthetically pleasing. An oil skimming device is almost always attached to the exterior of the structure which further reduces aesthetics and is subject to theft.

The Florida Department of Transportation Roadway Design Section has been developing a novel outfall structure which would address the concerns mentioned above and could be used in most situations. Referred to here as the "attenuator", this device was originally suggested by Frank Chupka of the Department [1]. The proposed configuration resembles a U-endwall specified as in FDOT Index 261 [2] conforming to the pond berm slope and incorporating a traversable grate over the opening. The skimmer and the control opening (the weir and/or orifice) **are** located at the back of the U-endwall, within the pond berm, The U-endwall is joined at the weir to a drop box, from which direct discharge may occur or the box may subsequently be connected to a discharge pipe. Maintenance personnel will be able to mow over the structure since it conforms to the berm slope and has a traversable grate. The skimmer is internal to the structure and therefore hidden, so that the only component clearly visible is the grate itself. Theft and vandalism usually associated with externally mounted skimmers should be eliminated. In some situations requiring high capacity, tandem units could be employed.

The principal goal of this research effort is to examine the hydraulic performance characteristics of the proposed outfall structure and present design parameters prior to adoption. This information (as well as the general operating characteristics ) is essential in order to properly size the stormwater facility and minimize liability due to flooding.

### Description of The Attenuator

The attenuator discussed in this report is shown in Figure 1 as originally installed. **A** schematic diagram of the attenuator as currently envisioned and the experimental facility used in this investigation are presented in Figure 2. The device consists of a conventional culvert endsection modified by the addition of a weir set in the end of the channel, and a skimmer at the end of the mitered entrance. **A** receiving box has been added to form a transition for the flow into a drainline for eventual disposal. As designed, the control (discharge limiting) point along the flow path is intended to be the weir, and it is the elevation of the detention pond that is to be regulated.

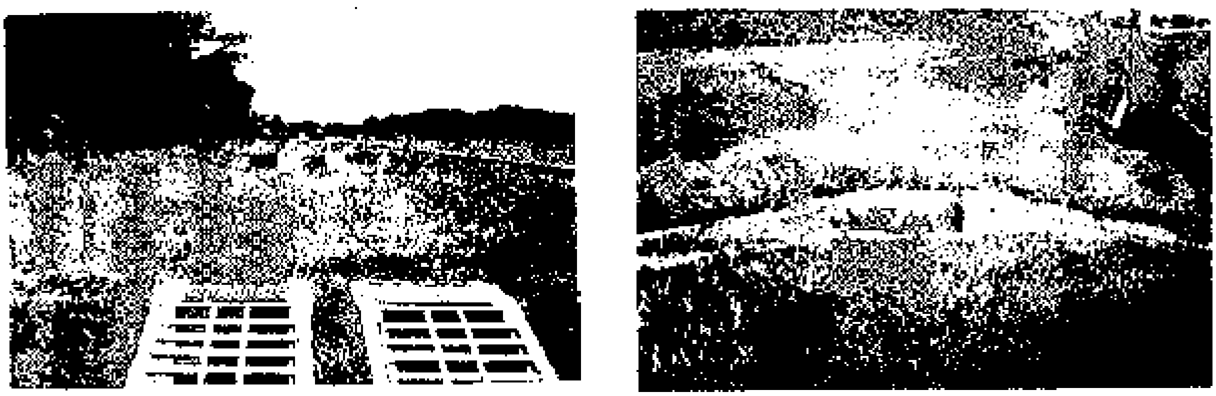

Figure 1: Field installation of attenuator as originally conceived, in a tandem installation with grate in place (left) and with grating removed to show skimmer and weir (right). Photographs courtesy of Frank Chupka, FDOT.

It is assumed that no significant velocity develops in the pond, except near the entrance to the attenuator. Beginning at this point, the operation of the structure may be described as follows. Water flows along the rectangular channel from the entrance to the skimmer with the elevation reduced in accord with the specific energy relation accounting for losses along the flow path, then under the skimmer forming a submerged jet. Downstream of the skimmer the flow through the weir is similar to that of a conventional weir but some consideration must be given to residual velocity of the persistent jet in the region between the skimmer and the weir opening. The average velocity in the channel may be estimated from the continuity relation in terms of the flow rate from the pond. Ignoring channel friction because the channel is short, only losses at the grating and the skimmer need to be considered. Thus, the discharge can ultimately be related to the water elevation in the pond H (measured above the weir crest).

$$Q = C_D L \sqrt{2g}\, H^{\frac{3}{2}} \qquad (1)$$

Here the discharge coefficient is assumed to be a function of several geometrical ratios accounting for the configuration of the weir, and possibly other factors. The extent of the weir crest in the flow direction, $B_w$ (thickness) has been chosen as the nondimensionalizing length for geometrical ratios. **As** written in this form the discharge coefficient is an empirical parameter including all significant losses and contractions in the attenuator and not simply related to the conventional discharge parameter associated with a weir. It is also emphasized that $C_D$ is nondimensional, not to be confused with the dimensional weir coefficient, which is the product of $C_D$ and $(2g)^{1/2}$. This discharge coefficient can be developed by experimental measurement or modeling. Once developed, it is assumed that $C_D$ is invariant with scale, and can be applied directly to the design of full size structures.

It is important to note however that several important features are incorporated in the attenuator that require special consideration in evaluating performance. The first of these issues is the fact that as proposed, both the skimmer and the weir will be fabricated in concrete, necessitating an extended wall thickness in the direction of flow. This aspect of control weirs was examined in Reference **3,** where it was reported that the discharge can be substantially modified, especially at lower heads or if the leading edge of the weir has been rounded.

Another type of complication occurs as a result of the relative size of the weir aperture, L, and the approach channel dimension, b. When weirs are utilized for measurement of flow rate (rather than a control function), specific requirements are made for the height of the weir crest as well as the side clearance (in the case of a contracted weir). While these specifications can be violated, special consideration must then be given to modification of the discharge relationship. For example, a weir formed with a very low crest would not be expected to obey the conventional empirical correlations. For the attenuator, the spacing of the weir sides away from the side wall of the channel will be important. For very wide weirs, performance may reflect a modification of the flow at the edge. This region has been already determined to be influential due to the variability of attachment as the flow turns around the corner at the edge [3]. Furthermore, it is noted that earliest designs for the control structure proposed installing suppressed weirs (a crest extending from wall to wall in the channel with no side lip). If employed, additional concerns for nappe ventilation and reproducible performance will be important.

Normally, control structures are intended to operate with a free discharge from the weir. However, special consideration should be given to performance when substantial water elevation is present in the receiving box due to tailwater conditions or flow limitation in the drainline from the box. If the weir flow is partially submerged then the tailwater affects the discharge. This issue has been previously examined [5], where it was determined that the reduction in discharge can be adequately represented by the Villamonte relationship even for weirs with extended crests.

Finally, in recent years, regulating bodies have often required the addition of skimmers to block the flow of oily waste and floating debris from passing through the weir. The skimmer must be positioned somewhat below the crest of the weir (typically 0.15 m) to preclude passage of this material as the pond elevation drops due to discharge or losses. The effect of skimmer placement on discharge has not been thoroughly investigated however. Other related issues include trash blockage of the flow apertures and the possible addition of bleed orifices to the weir plate.

## Experimental Investigation

As part of this study, an experimental simulation and rating of the control device was performed. Using standard scaling relationships this information can be transformed into design data. This data will eventually be used in the development of a predictive model, and the identification of other potential problems.

The experimental investigation of the performance of the attenuator was conducted using a 1/4 scale model. **As** shown in Figure 2, a simulated detention pond was constructed from resined plywood with the attenuator located at one end, discharging through a short length of pipe to a second ponding area **(for** tailwater control). Water pumped from a reservoir sump was introduced into the upstream pond through **a** large tee fitting behind a multiple V-notch baffle to intended to minimize the motion in the pond. Water elevation in both the upper and lower basin could be independently controlled.

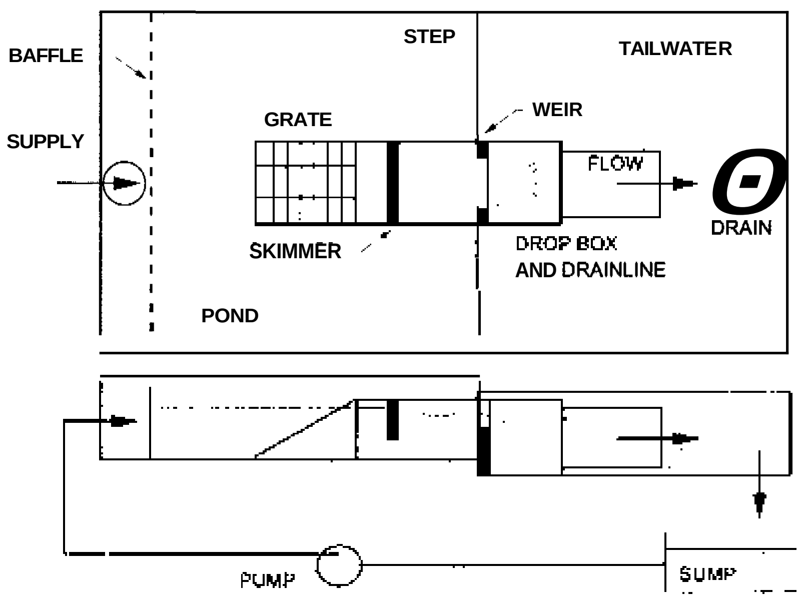


Figure 2: Schematic of the experimental arrangement used in this investigation showing the plan and side view of the attenuator as currently specified. The receiving box was connected to drainline in this study.

The model attenuator was also fabricated from resin coated plywood. In accordance with current FDOT plans, the entrance consisted of a nominal 4:1 sloped culvert end section resembling FDOT Index 261[2]. The receiving box bottom was dropped slightly, representing an optional configuration. The model included a removable grating and provision for an adjustable skimmer. Although both the grating and skimmer would be installed in field applications, some testing was conducted without these components to investigate modifications to hydraulic performance.

Experiments were conducted by first establishing a datum at the weir crest. A secondary reference elevation was taken at the channel bottom. Upstream and downstream piezometers were fitted with stilling tubes and measured directly with a sharp pointed scale to observe pond elevations. Measurements of the water depth before and after the skimmer were made directly with a scale for additional comparisons, but the water surface downstream of the skimmer was considerably disturbed and difficult to measure accurately. Flow into the pond was measured by a paddle wheel type flowmeter measuring inlet pipeline velocity. A calibration of the flowmeter output was made by inserting a metal weir in the flow channel (without the skimmer and grate) and utilizing standard weir relations.

## EXPERIMENTAL RESULTS AND DISCUSSION

In a series of tests of the operation of the model attenuator, observations of water depth and discharge were made for several weir widths, L, as indicated in Table 1. In these tests the crest height, $P_w$ was 0.127 m, the crest breadth, $B_w$ was **0.038** m and the skimmer thickness $B_s$ was 0.063 m positioned at $P_S = 0.038$ m below the weir crest (except as noted). The width of the approach channel b, was 0.263 m. Information gathered during these tests was reduced to an overall discharge coefficient utilizing Equation 1. The results of these experiments are summarized in Figures **3** and 4. For comparison, the information developed in Reference **3** for weirs with extended crests has been superposed on this diagram. Although the configuration of the attenuator differs in the flow channel, the data exhibit a similar trend except that the discharge coefficient declines for higher flow rates rather than remaining constant. For design purposes, trend lines have been developed by combining all data above $H/B_W=1.7$. Data taken with a very narrow weir ($L/b=0.19$) clearly exhibit an increasing trend and more closely followed the correlation developed for the unconfined weir. Some experiments were conducted with a full width (suppressed) weir and while the results of these tests were **as** expected, some fluctuation in performance was noted, due to poor ventilation of the nappe, as is often observed for this configuration.

Table 1: Configuration of weirs tested (dimensions in meters).

| WEIR | HEIGHT | WIDTH | THICKNESS |
|---|---|---|---|
| A | 0.127 | 0.184 | 0.038 |
| B | 0.203 | 0.184 | 0.038 |
| C | 0.127 | 0.127 | 0.038 |
| D | 0.127 | 0.263 | 0.038 |
| E | 0.127 | 0.051 | 0.038 |
| CAL | 0.127 | 0.127 | THIN |

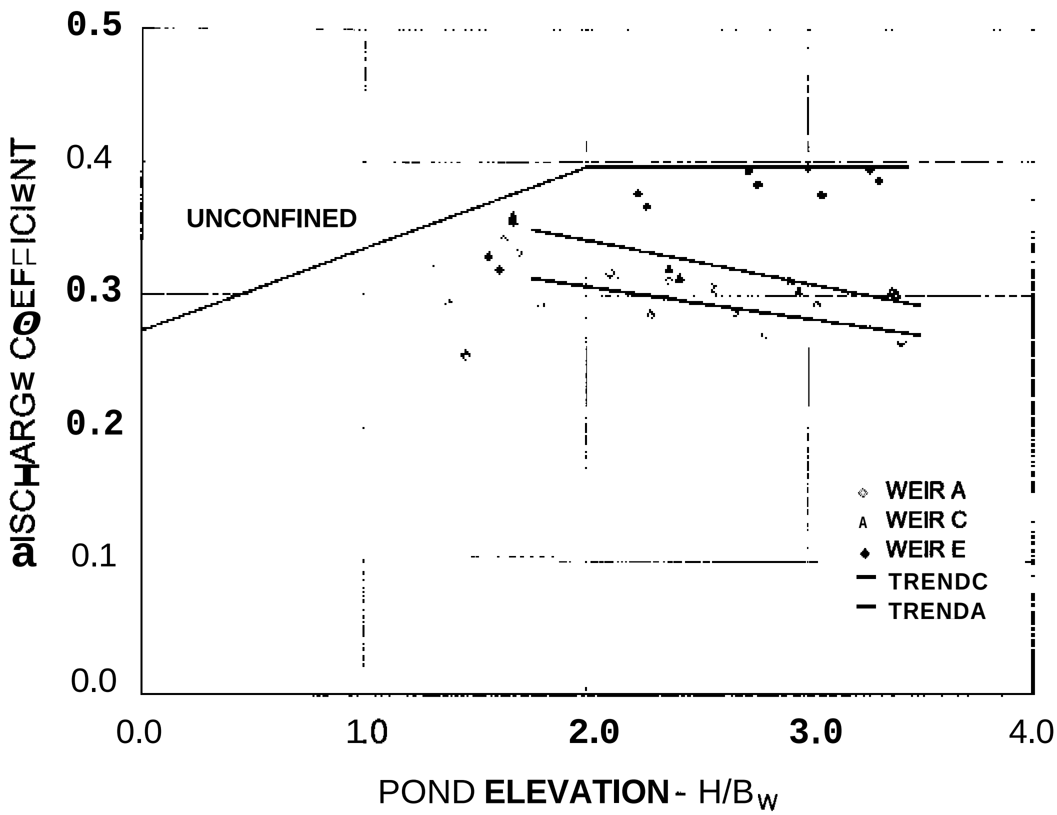


Figure 3: Experimental determination of the discharge coefficient based on pond elevation above the weir crest (Equation 1). Correlation for unconfined extended weirs **[3]** and trend lines for L/b= 0.19 and 0.48 added (cf. Table2).

To better understand of the flow modification at each point along the flow channel, a second series of experiments was conducted for variable skimmer position and configuration. In this manner the influence of each element could be isolated and independently assessed. Furthermore, similar tests were conducted with weirs with rounded edges and also with the metal calibration weir in place to eliminate the effect of the weir. The following observations were made.

1. With the grate and skimmer removed, the attenuator behaved much like a simple weir in a flow channel, with discharge modified by the thickness of the weir, especially at lower heads. The addition of the skimmer substantially reduced the coefficient of discharge. The effect of the grating was very small.

**2. As** observed in previous studies [3], rounding the weir edge elevated the overall performance.

3. The position of the skimmer was observed to be quite important. When the skimmer was lowered (increasing the velocity underneath) the coefficient of discharge was substantially reduced, Water flowing in the region between the skimmer and the weir was quite disturbed.

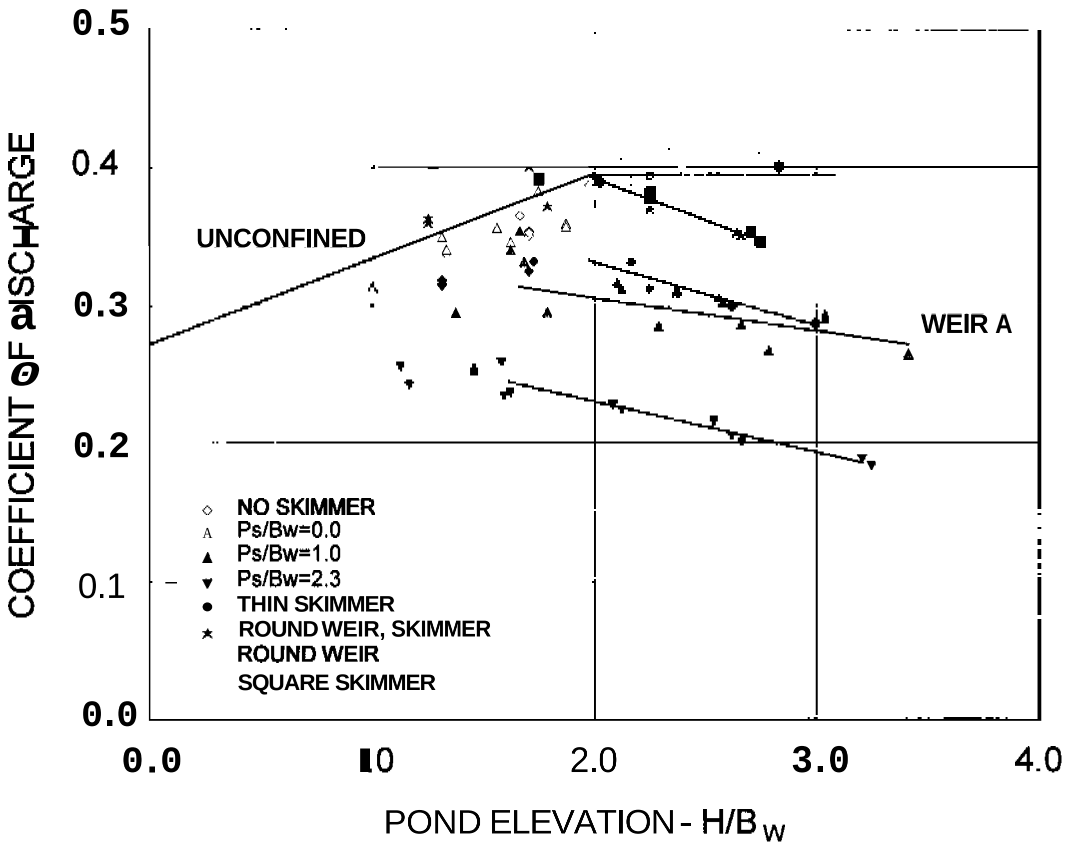


Figure **4:** Comparative performance of several alternate configurations. Correlation trend lines added (cf. Table 2).

Based on the experimental observations, a simplified picture of the flow path in the attenuator emerges. All along the channel, the kinetic energy of the flow and consequent losses are small enough to be neglected except under the skimmer. **A** substantial amount of energy **is** lost at this point, reducing the head at the weir. **A** residual flowjet from under the skimmer rises along the weir plate and may interfere with the flow through the opening, Contraction through the weir is much like that observed in the static case [3], in that some reattachment at the edges may be observed and that the condition of the leading edge is especially important.

Table 2: Recommendations for unconfined rectangular notch weirs and empirical correlations for the attenuator developed in this investigation. For the attenuator, the discharge coefficient should be merged with the unconfined value at lower heads. In all cases, it may be necessary to interpolate to obtain a smooth relationship for the discharge coefficient.

UNCONFINED WEIRS (excerpted from [3]),

**SHARP** LEADING EDGE (LOWER LIMIT)

| | |
|---|---|
| $(.25<H/B_w<2.0)$ | $C_D=0.053(H/B_w)+0.278$ |
| $(H/B_w>2.0)$ | $C_D=0.39$ |

ROUNDED OR BEVELED EDGE

| | |
|---|---|
| $(H/B_w<2.0)$ | ELEVATE SHARP EDGED VALUE ABOVE BY 10% |
| $(H/B_w>2.0)$ | $C_D=0.42\text{-}0.5$ |

ATTENUATOR (this investigation) $P_s/B_w=1$ (for use above $H/B_W=1.7$)

| | | |
|---|---|---|
| WEIR **A** | $L/b=0.70$ | $C_D=0.355-0.024(H/B_w)$ |
| WEIR C | $L/b=0.48$ | $C_D=0.406-0.0325(H/B_w)$ |
| WEIR D | $L/b=1.0$ | $C_D=0.364-0.0290(H/B_w)$ |
| WEIR E | $L/b=0.19$ | USE UNCONFINED RELATION |

WEIR **A** MODIFICATIONS $L/b=0.70$ $P_s/B_w=1$, unless noted

| | |
|---|---|
| SKIMMER POSITION ($P_s/B_w=2.3$) | $C_D=.303-.0365(H/B_w)$ |
| THIN PLATE SKIMMER | $C_D=.422-.0452(H/B_w)$ |
| ROUNDED EDGE | $C_D=.512-.0597(H/B_w)$ |

(rounded and square edged skimmer data combined)

## Performance of a Full Size Attenuator

**As** an example of the use of the data obtained in the experiments reported here, the hydraulic capacity of a full size structure (as specified in Table 3) has been predicted (Figure 5) using the empirical relationships summarized in Table 2.

Table **3:** Proposed dimensions of full size attenuator.

| | |
|---|---|
| Crest height | 0.508 m |
| Channel width | 1.050 m |
| Lower lip of skimmer | 0.152 m below weir crest |
| Weir width | 0.737, 0.508, 0.203 m |
| Weir crest breadth | 0.150 m |
| Skimmer thickness | 0.250 m |

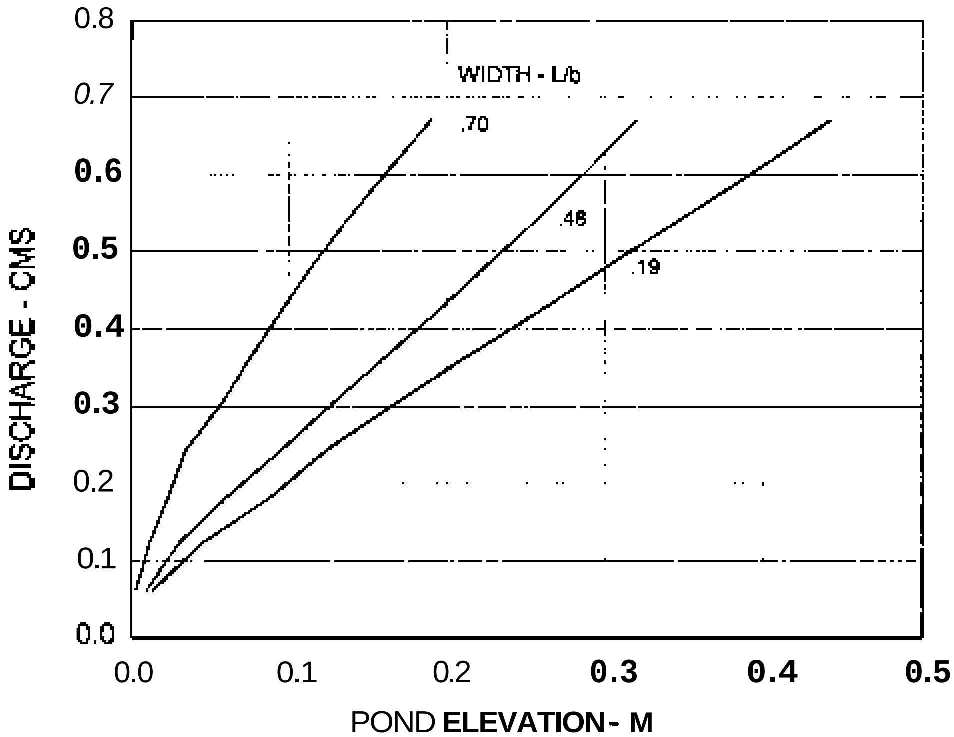


Figure 5: Predicted hydraulic performance of full size attenuator; discharge vs. pond head above weir crest for three different weir apertures.

## CONCLUSIONS AND RECOMMENDATIONS

The principal conclusions and recommendations resulting from the research reported here are as follows:

1. Experiments have been conducted for a one-quarter scale model of a proposed stormwater attenuator and the results have been reduced to a overall discharge coefficient for the integrated structure utilizing the conventional weir equation. Width of the weir aperture is an important parameter affecting the discharge coefficient. Overall the performance of the attenuator was acceptable.

**2.** It was found that the skimmer contributes substantially to a reduction in capacity of the attenuator from that which might be expected from a simple weir arrangement. This fact should not limit application, however, if suitable discharge relationships are employed. It does not appear that the thickness or the condition of the leading edge of the skimmer exert substantial influence on capacity.

**3.** As observed in previous investigations, the rounding of the leading edge of the weir contributes to a substantial improvement in discharge and should be accounted for in design.

4. The use of suppressed (full width weirs) is not recommended due to potential reproducibility problems.

5. Design capacities for an example of a full size attenuator as currently proposed have been calculated using the results of this investigation.

Several important issues (submerged discharge, trash considerations, overtopping, etc.) were not considered here. **An** extended study including an analytical approach to the computation of hydraulic performance is underway and will be presented elsewhere [4].

## ACKNOWLEDGMENTS

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# INCORPORATING THE CONCEPT OF RISK IN STORMWATER MANAGEMENT

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## ABSTRACT

Stormwater runoff has been identified as a major source of pollution to Tampa Bay. This led SWFWMD to the development of a water quality management plan to control and reduce pollutant loading from the McKay Bay watershed. Traditionally, water quality management plans have been developed using approaches that lend little consideration to the relative risk associated with those contaminants. In addition, the identification of the type of treatment required for both baseflow and stormwater runoff has been done based on generalizations of water quality conditions in the incoming flows. This paper discusses the application of a risk-based approach for identifying pollutants of concern in the McKay Bay watershed, as well as the methodology used for identifying pollution control methods.

## INTRODUCTION

Previous studies have demonstrated that concentrations of contaminants in water, sediment, and biota in Tampa Bay are elevated and in certain areas exceed regulatory or guidance levels designed to protect ecological resources (Brooks and Doyle 1992; Long et al 1991, 1994) and/or human health (Frithsen et al, 1995 and Parson **ES** 1996). One of the most important areas of concern is McKay Bay, the 1,400-acre urban estuary located on the northeast portion of Tampa Bay.
**As** part of the effort to control pollution in Tampa Bay, SWFWMD elected to conduct a water quality management study of the McKay Bay watershed. This watershed is an intensely urbanized area covering approximately 30-square mile area that collects predominantly untreated stormwater from concentrated industrial areas and residential developments. An important source of the pollution load entering McKay **Bay** is from stormwater runoff. Pollution impacts in the Bay have involved changes in the chemical, physical, and/or biological integrity of the system. In addition, impacts may include increased risk to human health with exposure to toxic pollutants through ingestion of fish or shellfish.

The overall objectives for the water quality management plan were to identify, analyze, and recommend control measures to control pollutants. **A** general risk-based framework was used to meet these objectives. To optimize the removal efficiency of potential control measures, it was necessary to first identify chemicals of potential concern (COPCs). Previous studies have identified control of nitrogen loads as a major water quality objective for the entire Tampa Bay. Other COPCs considered in this study were those chemicals that may cause adverse (toxic) effects in aquatic or terrestrial organisms, or that pose potential human health risks. **A** preliminary risk analysis was used to identify the COPCs and prioritize discharge locations for further study. The methodology and results used for identification of COPCs and basin prioritization based on contaminant risks versus total loads have been presented previously.

Based on the results of preliminary analysis, a limited baseflow, stormwater, and sediment sampling program was implemented to further characterize stormwater discharges, The results of this sampling effort were then used to develop appropriate measures to decrease pollutant loads and reduce risks associated with nonpoint discharges of COPCs. Typical stormwater management projects focus on total loads of conventional parameters like nutrients and metals, and recommend treatment methods on removal of particulates. While traditional methods may be adequate for reduction of pollutant loads, the results of this study suggest that additional consideration of pollutant form is required to address potential risks associated with stormwater pollutants. The sampling results for this study suggest that for both baseflow and stormwater discharges, the dissolved fraction of numerous stormwater pollutants represented a significant portion of the total discharge, **and** should be considered when selecting appropriate treatment methods. The methods and results for the baseflow and stormwater sampling plan and subsequent development of BMPs to reduce pollutant loadings are discussed in this paper. Sediment sampling results are also discussed briefly as they relate to stormwater discharges.

**Methodology**

For the McKay Bay study, baseflow, stormwater, and sediment samples were collected from strategic locations within the watershed on the preliminary risk evaluation of historical water and sediment quality data. Baseflow and sediment sampling were performed in June **1997;** wet weather sampling was performed in December 1997 and January 1998. All field and laboratory measurements were performed in accordance with the FDEP/EPA approved Comprehensive Quality Assurance Project Plan.

**A** total of five baseflow grab samples were collected at the end of the dry season. One field duplicate and one equipment blank were also collected. Samples were collected at stations located in the lower to middle portions of three drainage basins considered the highest priority for pollutant control purposes, the 29th St., 43$^{rd}$ St. and 50$^{th}$ St. basins.

Stormwater samples were collected near the outfalls of two of the priority basins, the 29$^{th}$ St. and 50$^{th}$ St. drainage basins. Flow-weighted composite samples were collected for representative storm events at each sampling location (total of two sampling events). Rainfall data prior to and during storm sampling events were also evaluated as part of the stormwater sampling plan to estimate rainfall versus runoff relationships. General criteria for qualifying rain events were set in accordance with **U.S. EPA** criteria: the range of acceptable rainfall events was set at **0.24** to **0.71** inches, a minimum of three hours of rainfall had to fall within this range, a minimum of three hours of rainfall had to occur, and qualifying storm events had to be preceded by 72 hours of dry weather. Some deviation was allowed on the high end of the rainfall range **as** long as it fell after the first three hours,

In addition to baseflow and stormwater data, limited sediment data were collected in depositional areas to help identify critical outfalls/discharges that represent potential sources of COPCs. These data will also be used for future comparisons to evaluate the effectiveness of implemented management practices. **A** total of seven surficial sediment grab samples were collected from the following areas: Northern McKay Bay adjacent to the 29th-50th Street outfalls, southeast McKay Bay adjacent to the SWFWMD parcel, northwest McKay Bay near DeSoto Park, and two Palm River stations near the US41 bridge and at the confluence with McKay Bay.

The following parameters were included in the sampling plan: oil and grease (O&G), total petroleum hydrocarbons (TPH), polyaromatic hydrocarbons (PAHs), pesticides (organochlorine/phosphate insecticides and herbicides), metals (cadmium, chromium, copper, lead, mercury, and zinc; total and dissolved), ammonia, nutrients (nitrate/nitrite, total nitrogen, ortho- and total phosphorous), total and dissolved solids, and BOD,. For the stormwater samples, TPH was analyzed instead of oil and grease; for sediment samples, individual **PAHs** were analyzed instead of O&G or TPH. Field measurements included temperature, specific conductance, pH, DO, and ORP at each sample location. One equipment blank was collected per sampling event. Water sample analyses were performed by several certified laboratories including the Southwest Florida Water Management District (SWFWMD), Environmental Quality Laboratory (EQL), and Southern Analytical Laboratories. Sediment sample analyses were performed by several certified contract laboratories including EQL and Savannah Laboratories.

## RESULTS

Analytes detected in the baseflow (6/97) and wet weather (12/97 and **1/98)** sampling are summarized in Table 1. Sediment sampling results are not tabulated in this paper, but are discussed briefly as they relate to stormwater discharges. Conventional parameters measured included: ammonia, and nutrients (nitrate/nitrite and phosphorous), solids (total and suspended), and BOD,. Toxic pollutants detected in baseflow and/or stormwater included: organic chemicals **(TPH,** phthalates, foaming agents, bis(2-chloroethyl-ether, malathion, and pentachlorophenol), and 11 metals (arsenic, cadmium, chromium, copper, iron, lead, mercury, nickel, selenium, silver, and zinc). Toxic pollutants detected in sediment samples included PAHs and metals. Several classes of organic chemicals (i.e., organochlorine/phosphate insecticides and herbicides) were included in the sampling plan due suspected discharges from land uses within the watershed and/or historical sediment and stormwater data, With the exception of malathion, pesticides were not detected in baseflow, stormwater, or sediment samples collected for this project. This may reflect elevated laboratory detection limits that did not allow detection of contaminants present at very low concentrations.

### Nutrients

No numeric water quality criteria exist for nutrients and BOD as impacts depend on the specific characteristics of each water system. However, the water quality in tributaries discharging to McKay Bay is above the $80^{th}$ percentile compared to other Florida streams based on total nitrogen concentration,

# Table 1.
## Baseflow and Stormwater Data
## McKay Bay Water Quality Management Plan

| Analyte | Units | 43rd St. Baseflow | | 29th St. Basin Baseflow | | 29th St. Basin Stormwater | | 50th St. Basin Baseflow | | 50th St. Basin Stormwater | | FL WQC[1] | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Sta3 | Dup (Sta7) | Sta1 | Sta2 | S11-12/97 | S11-1/98 | Sta4 | Sta5 | S12-12/97 | S12-1/98 | Fresh | Marine |
| **Conventional Parameters** | | | | | | | | | | | | | |
| Ammonia | mg/l as N | 0.02 | 0.03 | 0.08 | 0.06 | 0.66 | 0.02 | 0.41 | 0.16 | < 0.01 | 0.21 | ≤ 0.02 | |
| Nitrate + Nitrite | mg/l as N | 0.01 | 0.02 | 0.93 | 1.23 | 0.80 | 0.40 | 1.82 | 0.51 | 0.48 | 0.49 | No population imbalance. | |
| TKN ($NH_3$ + Org N) | mg/l as N | 1.28 | 1.61 | 0.60 | 0.42 | NA | NA | 1.64 | 1.29 | NA | NA | No population imbalance. | |
| Nitrogen, total | mg/l as N | | | | | 2.20 | 0.91 | | | 0.87 | 1.70 | | |
| Phosphorous, ortho | mg/l | 0.62 | 0.63 | 0.17 | 0.15 | 0.66 | 0.20 | 0.56 | 0.51 | 0.18 | 0.48 | No population imbalance. | |
| Phosphorous, total | mg/l as P | 0.75 | 0.76 | 0.18 | 0.15 | 0.72 | 0.28 | 0.58 | 0.59 | 0.25 | 0.61 | No population imbalance. | |
| Dissolved Solids, total | mg/l | 314 | 307 | 2055 | 492 | 281 | 12 | 709 | 2529 | 179 | 230 | | |
| Suspended Solids, total | mg/l | 2.24 | 2.24 | 0.43 | 0.35 | 32.06 | 12.04 | 0.52 | 6.03 | 31.24 | 18.45 | | |
| BOD | mg/l | 3.8 | 4.9 | < 1 | < 1 | 7.3 | 15.4 | 1.8 | 1.2 | 7.3 | 39.5 | No diss. oxygen impact. | |
| **Organic Chemicals** | | | | | | | | | | | | | |
| Oil & Grease | mg/l | < 1 | < 1.0 | < 1.00 | < 1.00 | NA | NA | < 1.00 | < 1.00 | NA | NA | ≤ 5.0 | ≤ 5 |
| TPH* | µg/l | NA | NA | NA | NA | 243 | 113 | NA | NA | 318 | < 100 | | |
| Foaming agents | mg/l LAS | 2.20 | 2.30 | 0.22 | 0.12 | NA | NA | 0.17 | 0.42 | NA | NA | ≤ 0.5 | ≤ 0.5 |
| Malathion* | µg/l | 8.5 | 6.2 | 11 | 5 | < 1.0 | < 1.0 | < 1 | < 1 | < 1.0 | < 1.0 | ≤ 0.1 | ≤ 0.1 |
| Butyl benzyl phthalate | µg/l | < 1 | < 1.0 | 1.1 | < 1 | < 1.0 | < 1.0 | < 1 | < 1 | < 1.0 | 1.4 | ≤ 3 | |
| Di(2-ethylhexyl)phthalate | µg/l | < 1.0 | < 1.0 | < 1.0 | < 1.0 | < 1.0 | 30.6 | | | < 1.0 | 1.2 | ≤ 3 | |
| Bis(2-chloroethyl) ether | µg/l | 1.6 | 140.0 | < 1 | < 1 | < 1.0 | < 1.0 | < 1 | < 1 | < 1.0 | < 1.0 | | |
| Pentachlorophenol | µg/l | < 1.0 | < 1.0 | < 1.0 | < 1.0 | < 1.0 | < 1.0 | | | 0.13 | < 1.0 | ≤ 30.0 | ≤ 7.9 |
| **Metals** | | | | | | | | | | | | | |
| Arsenic, total | µg/l | NR | NR | NR | NR | < 23 | < 23 | NR | NR | < 23 | < 23 | ≤ 50.0 | ≤ 50 |
| Arsenic dissolved | µg/l | 15.7 | 13.4 | 1.3 | 0.9 | 6.9 | 1.92 | 13.2 | 0 | 0.959 | 6.87 | | ≤ 36 |
| Cadmium, total | µg/l | < 0.3 | < 0.3 | < 0.3 | < 0.3 | 0.997 | 2.35 | < 0.3 | < 0.3 | 0.692 | 2.08 | ≤ 0.4 | ≤ 9.3 |
| Cadmium dissolved | µg/l | 0 | 0 | 0 | 0 | 0.41 | 0.312 | 0.1 | 0 | 0.24 | 0 | | |
| Chromium, total | µg/l | < 4.7 | < 4.7 | < 4.7 | < 4.7 | 10.8 | < 4.7 | < 4.7 | < 4.7 | 5.99 | < 4.7 | ≤ 67 | |
| Chromium dissolved | µg/l | 2.1 | 1.9 | 0.8 | 0 | 4.44 | 0.351 | 1.2 | 0.9 | 2.01 | 0.536 | ≤ 11.0 | ≤ 50 |
| Copper, total* | µg/l | 11.1 | 12.1 | 1.8 | < 1 | 55.1 | 10.2 | 11.6 | 6.1 | 23.3 | 34.3 | ≤ 3.6 | ≤ 2.9 |
| Copper, dissolved | µg/l | 6.6 | 6.6 | 1.3 | 0 | 21.5 | 9.75 | 7 | 1.9 | 7.75 | 8.99 | | |

# Table 1, cont.
## Baseflow and Stormwater Data
## McKay Bay Water Quality Management Plan

| | | 43rd St. | | 29th St. Basin | | | | 50th St. Basin | | | | FL WQC [1] | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Baseflow | | Baseflow | | Stormwater | | Baseflow | | Stormwater | | | |
| Analyte | Units | Sta3 | Dup (Sta7) | Sta1 | Sta2 | S11-12/97 | S11-1/98 | Sta4 | Sta5 | S12-12/97 | S12-1/98 | Fresh | Marine |
| **Metals, cont.** | | | | | | | | | | | | | |
| Iron, total | mg/l | 0.344 | 0.311 | 0.320 | 0.329 | 2.38 | 0.32 | 0.281 | 0.406 | 1.54 | 0.73 | ≤ 1.0 | ≤ 0.3 |
| Iron, dissolved | mg/l | 0.199 | 0.2 | 0.045 | 0.022 | 1.00 | 0.30 | 0.192 | 0.161 | 0.59 | 0.65 | | |
| Lead, total* | µg/l | 2.8 | < 2 | < 2 | 2.8 | 24.83 | 4.59 | 3.4 | 5.6 | 22.96 | 10.6 | ≤ 0.54 | ≤ 5.6 |
| Lead, dissolved | µg/l | 1.4 | 0 | 0.6 | 3 | 9.2 | 2.77 | 1.9 | 0.7 | 10.5 | 6.59 | | |
| Mercury, total* | µg/l | < 0.1 | < 0.1 | < 0.1 | < 0.1 | 0.3 | 0.17 | < 0.1 | < 0.1 | 0.42 | 0.115 | ≤ 0.012 | ≤ 0.025 |
| Mercury dissolved | | | | | | < 0.1 | 0.12 | | | 0.12 | 0.13 | | |
| Nickel, total | µg/l | 52.7 | 59.6 | < 4.3 | < 4.3 | 10.6 | < 4.3 | < 4.3 | < 4.3 | 5.22 | 4.59 | ≤ 48.8 | ≤ 8.3 |
| Nickel, dissolved | µg/l | 46.2 | 68.0 | 0 | 3.5 | 3.45 | 1.45 | 2.3 | 1.8 | 1.28 | 3.07 | | |
| Zinc, total* | µg/l | < 30 | < 30.0 | < 30 | < 30 | 181 | 46.3 | 30 | 42 | 160 | 83 | ≤ 32.7 | ≤ 86 |
| Zinc, dissolved | µg/l | 36 | 40.0 | 14 | 10 | 75.8 | 43.5 | 42 | 29 | 59.6 | 62.1 | | |

**Note:**

"*" - indicates COPC (see text).

NR - not reported; NA - not applicable, "0" - value as reported by SWFWMD analytical lab.

Baseflow grab samples collected 7/97 and stormwater samples collected 12/97 and 1/98.

1. Florida water quality criteria (WQC), Class III, from 62-302.530, 1/96. Blank indicates no criterion.
   Freshwater metals criteria for Cd, Cr, Cu, Pb, Ni, and Zn based on default hardness of 25 mg/l.
   Criterion for As based on As+3, total chromium based on Cr+3, dissolved Cr based on Cr+6.
   Criterion for foaming agents (LAS) based on detergents, butyl benzyl phthalate based on total phthalates.

As shown in Table 1, the concentration of total nitrogen was relatively constant. In five out of the six samples it varied between 1.3 and **1.8** mg/L. However, the concentration of the various forms of nitrogen varies widely among stations. For example, the ratio of NOx to total nitrogen varies between 0.01 and 0.75. The limited nature of the sampling program did not allow for an analysis of causes for this condition,

Stormwater samples show a larger variation in the concentration of total nitrogen than baseflow samples, with values ranging from 0.87 and **2.20** mg/L. In addition, the data do not show consistency among the two stations sampled. The sample from the first storm event showed that the total nitrogen concentration in the 29th St. basin was approximately twice as large as the concentration in the 50th St. basin sample. The opposite occurred for the samples taken during the second storm event. However, variation on the various forms of nitrogen in the stormwater runoff is not as drastic as that for baseflow samples.

Another nutrient of interest was phosphorus. Although this is not considered the limiting nutrient in the McKay watershed, total phosphorus concentrations at all stations also varied significantly, although not as dramatically as total nitrogen. Phosphorus concentrations seem to show similar pattern in both stormwater **and** baseflow samples. Total phosphorus concentration ranged between 0.15 and 0.76 mg/L. **As** expected, the majority of the phosphorus is in orthophosphate form.

Total nitrogen and total phosphorus were also measured in the sediment samples collected in 1997. Since a large portion of the nutrients in the sediment is associated with the organic matter present, sediment nutrient levels tended to follow the same basic trends exhibited by TOC. The highest levels of both nitrogen and phosphorus were measured in the Palm River with the lowest levels measured in northwest McKay Bay, Sediment nitrogen levels ranged from 122 to 3990 mg/kg and averaged 804 mg/kg while phosphorus concentrations ranging from 170 to 5080 mg/kg with an overall average of 1119 mg/kg.

The ratio of sediment nitrogen to phosphorus **can** often provide information of the source of the nitrogen **and** phosphorus found in the sediments as well as information on the amount of nutrient resulting from anthropogenic sources. Sediments at 1997 stations exhibited nitrogen-to-phosphorus (N\P) ratios ranging from 0.1 to 1.7 with an average of **0.8.** Generally, these N/P ratios suggest a combination of soil humus and terrestrial plant material as a primary source of nitrogen and phosphorus in the sediments (Meybeck, 1982). Higher N/P ratios were measured for Palm River and upper McKay Bay sediments reflecting increased potential nitrogen inputs from aquatic plants or anthropogenic nitrogen sources, such as fertilizers.

## Toxic Organic Chemicals

Several analytes classified as "toxic" pollutants were detected in baseflow and/or stormwater samples including: O&G/TPH, phthalates, foaming agents, bis(2-chloroethyl)ether, malathion, and pentachlorophenol.

The presence of total petroleum hydrocarbons (TPH) in stormwater samples indicates runoff from transportation land uses or industrial areas where fuels are released from vehicles, maintenance areas, leaking distribution pipes, or other spills. Petroleum releases are a primary anthropogenic source of PAHs identified as sediment COPCs in McKay Bay. Petroleum hydrocarbons were detected in stormwater from both the 29th **and** 50th St. basins. In 29th St. basin stormwater, TPH concentrations were **243** μg/l and 113μg/l for the 12/97 and 1/98 storm events, respectively, In 50th St. basin stormwater, TPH concentrations were 318 μg/l and <100 μg/l (below detection) for the **12/97** and **1/98** storm events, respectively.

Phthalates (butylbenzyl phthalate, di(2-ethylhexyl)phthalate [DEHP], and di-n-butyl phthalate) were detected in stormwater, but also in the equipment blank suggesting potential contamination of sampling equipment. However, DEHP was detected (30.6 μg/l) at approximately 10-times the WQC (£3 μg/l) in the **1/98** stormwater sample for the 29th Street basin.

Foaming agents, or surfactants, reported as linear alkylbenzenesulfonate **(LAS),** are widely used synthetic surfactants in domestic detergents. **LAS** was detected at low concentrations in baseflow for all three basins, with the maximum concentration in the 43rd St. basin (2.25 mg/l) above the Florida water quality criterion (0.5 mg/l). The presence of surfactants in the 43rd St. discharge may be attributable to **an** industrial release (spill or other illicit discharge).

Bis(2-chloroethyl) ether was not detected in stormwater or baseflow samples from the 29th or 50th St. basins, but was detected at significant concentrations in baseflow from the 43rd St. basin. This chemical is used as an industrial solvent, soil fumigant, or textile scouring agent; its presence in the 43rd St. discharge is likely due to an industrial release (spill or other illicit discharge). While no water quality standard has been promulgated for this chemical, it has been classified by U.S. **EPA** as a probable human carcinogen. Bis(2-chloroethyl) ether does not appear to be highly toxic to aquatic life) or persistent in the environment, or bioaccumulate significantly.

Malathion was detected in baseflow samples for the 29th and 43rd St. basins at one to two orders of magnitude above the water quality standard. The presence of malathion in baseflow samples is likely due to aerial spraying for med fly control in the Spring of 1997, a few weeks preceding the baseflow sampling event.

Pentachlorophenol (PCP) was detected at low levels in one stormwater sample from the 50th St. basin (0.13 μg/l). This concentration was well below the Florida freshwater and marine criteria and may result from natural sources. PCP was not detected in baseflow samples.

## Metals

Eleven trace metals or metalloids were detected in baseflow and/or stormwater samples including: arsenic, cadmium, chromium, copper, iron, lead, mercury, nickel, selenium, silver, and zinc. With the exception of cadmium, all of these metals are considered essential nutrients for biological organisms, but are toxic to sensitive organisms at elevated concentrations. Five metals were detected at concentrations significantly above Florida water quality criteria. In addition, the reported dissolved fraction for five of these metals including copper, lead, mercury, nickel, **and** zinc was **30-SO%,** or higher. Because potential risks increase with increasing dissolved fraction (increased bioavailability) and appropriate treatment methods differ for dissolved versus particulate-associated metals, consideration of pollutant form is an important consideration for reduction of risks associated with stormwater discharges. Sediment data collected for this study were used to identify pollutants depositing in particulate form. Because of the large differences in the sediment types found in the McKay Bay system, metal to aluminum ratios were also utilized to evaluate potential differences between background and site conditions (FDEP 1988).

**Copper.** Possible sources of copper include stormwater runoff containing copper-based algaecides, pesticides, and fertilizers, domestic wastes, and industrial discharge. For baseflow samples, average total copper concentrations ranged from 1.4 to 11.6 μg/l, with concentrations in the 43rd and 50th St. basins exceeding Florida water quality criteria. For stormwater samples, average total copper concentrations for the 43rd and 50th St. basins (32.7 and 28.8 μg/l, respectively) were well above Florida water quality criteria. Sediment copper concentrations ranged from 1.8 to 425 mgkg, with the highest copper concentration observed in upper McKay Bay. Copper to aluminum ratios for stations in upper McKay Bay were above the expected range; ratios for remaining stations were within or at the upper limit of the expected range. These results indicate that copper associated with suspended particulates is depositing near stormwater outfalls. The dissolved fractions for baseflow **(46-57%)** and stormwater (29-48%), however, suggest that appropriate treatment methods must also address copper in dissolved form in addition to particulate removal.

**Lead.** Possible sources of lead in the environment include runoff from transportation land uses (roadside runoff, automobile emissions, battery disposal) and discharges from industrial areas. For baseflow samples, average total lead concentrations ranged from **2** to **3** μg/l, with concentrations in all basins sampled exceeding Florida freshwater criteria. For stormwater samples, average total lead concentrations for the 43rd and 50th St. basins (6.0 and 8.5 μg/l, respectively) exceeded Florida freshwater and marine criteria. Lead in sediments averaged **47.5 mgkg** and **ranged** from **7.3** to 156 mgkg, with the highest lead concentrations observed in upper McKay Bay and East Bay. For seven of the eight sampling sites, lead to aluminum ratios exceeded the expected range. These results indicate that lead associated with suspended particulates is depositing near stormwater outfalls. The dissolved fractions for baseflow (29-41%) and stormwater *(5*1-75%), however, suggest that appropriate treatment methods must also address lead in dissolved form in addition to particulate removal.

**Mercury.** Possible anthropogenic sources of mercury in the environment include industrial discharges, stormwater runoff containing pesticides, and atmospheric deposition. For baseflow samples, average total mercury concentrations were below detection limits (0.1 μg/l) for all stations. For stormwater samples, average total mercury concentrations for the 43rd and 50th St. basins **exceeded** Florida freshwater and marine criteria (0.2 **and** 0.3 μg/l, respectively). Mercury concentrations at six of the eight sediment stations were near or below the analytical detection limits. Two stations in upper McKay Bay and the Palm River exhibited sediment concentrations of 0.21 **and** 0.27 mgkg, respectively, and were above Florida sediment screening criteria. Due to the poor relationship of sediment mercury concentrations with aluminum levels, no analysis of the mercury to aluminum ratios was conducted (FDEP 1988). The dissolved fractions for samples at both stormwater stations were approximately 47%, suggesting that removal of mercury not associated with (large) particulates is an important consideration for treatment options.

**Nickel.** For baseflow samples, average total nickel concentrations ranged from less than 2.9 μg/l to ***56.2*** μg/l, with concentrations in the 43rd St. basin exceeding freshwater and marine water quality criteria. For stormwater samples, average total nickel concentrations for the 43rd and 50th St. basins (7.5 and 4.9 μg/l, respectively) were below freshwater and marine water quality criteria. Nickel in sediments ranged from 1.82 to 14.7 mgkg with maximum concentrations in upper McKay Bay and the Palm River. Because all sediment nickel concentration were below screening criteria, nickel to aluminum ratios were not evaluated. The average dissolved fractions for baseflow (41-48%) and stormwater (33-44%), however, suggests that removal of nickel not associated with (large) particulates is an important consideration for treatment options.

**Zinc.** Possible anthropogenic sources of zinc include runoff from transportation land uses (roadside runoff, automobile emissions), stormwater containing fertilizers and pesticides, and industrial discharge. For baseflow samples, average total zinc concentrations were below **30** µg/l, with concentrations in all basins sampled below freshwater and marine criteria. For stormwater samples, average total zinc concentrations for the **43**$^{rd}$ and **50**$^{th}$ St basins (113.7 and 121.5 µg/l, respectively) were well above freshwater and marine criteria. Zinc in sediments averaged 124 mgkg and ranged from 14 to **361** mgkg with a maximum concentrations in upper McKay Bay and the Palm River. For most stations, zinc to aluminum ratios fell within or on the upper border of the expected range. For several stations in upper McKay Bay, zinc to aluminum ratios were well above the expected range. These results indicate that zinc associated with suspended particulates is depositing near stormwater outfalls. The dissolved fractions for baseflow samples **(40-99%)** and stormwater (50-52%), however, suggest that appropriate treatment methods for zinc must address dissolved forms in addition to particulate removal,

## DISCUSSION

Due to the variable and intermittent nature of stormwater discharges, pollutant characterization and estimation of loads to receiving waterbodies are often based on simple models or limited sampling efforts. Traditionally, stormwater management actions including selection of BMPs and treatment methods are based on the assumption that a large portion of the pollutant load is in particulate form. **As** shown by the results of this study, however, pollutants typically associated with solids loading may also be present at significant levels in dissolved form.

The results of this analysis have implications for the type of treatment that would be effective for pollution control. For example, it was determined during this study that the simple retrofit of existing flood control facilities as detention ponds for water quality would be probably effective for removal of the TKN associated with the particulate material. The TKN concentration represents, on the average, approximately 50% of the total nitrogen. However, the removal of the additional 50% of the total nitrogen is in NOx form, which is generally in dissolved form. Detention would not be effective as a treatment method. It was decided that a combination detention/created wetland would be more appropriate.

The results of this study also indicated that several toxic chemicals such as petroleum hydrocarbons and heavy metals were present above water quality criteria in both baseflow and stormwater discharges. For permitted wastewater discharges, metals and organic chemicals must be removed **prior** to discharge to minimize potential adverse (toxic) effects on biota in the receiving waterbody. For stormwater discharges, consideration of total and dissolved forms of toxic pollutants is also important to reduce impacts to receiving waterbodies as well as minimize potential exposures in treatment systems, particularly where constructed wetlands may be utilized as habitat.

For the organic chemicals and metals identified as pollutants of concern in stormwater discharges to the McKay Bay system, several BMPs were selected as recommended treatment options to reduce pollutants in both particulate and/or dissolved forms. These included projects incorporating both detention and wetland treatment. These BMPs will serve to reduce solids and particulate nutrient loadings as well as removal of toxic pollutants associated with particulates. They will also be effective for removing toxic pollutants identified as COPCs in baseflow and stormwater (PAHs, phthalates, LAS, copper, lead, mercury, nickel, and zinc).

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# MEGGINNIS ARM BASIN DIAGNOSIS --- A DISTRIBUTED WATERSHED MODEL USING XP-SWMM32™

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## ABSTRACT

Urban development in the 2,230-acre Megginnis *Arm* basin of the Lake Jackson watershed in the recent decades has resulted in serious water quality and aquatic habitat degradation. Increases in impervious land surface led to increased stormwater runoff volume and pollutant overloading which also contributed to water quality problems.

The study area mainly includes the Megginnis Creek sub-watershed that is the most intensively developed portion of the Lake Jackson Basin. Flows through the Megginnis Creek sub-basins are from south to north through a well-defined drainage system. **A** total of three facilities are discussed: the NWFWMD's Megginnis **Arm** Stormwater Treatment Facility, the City **of** Tallahassee's John **Knox** Road Facilities, and the City of Tallahassee's Boone Boulevard facility. From 1994 to 1996, a water quality monitoring program was undertaken to monitor the effectiveness of these facilities and data compiled from this program are expected to be used in our **SWMM** model development. The aim of this work is to develop a distributed watershed model to evaluate the effectiveness of the stormwater management facilities for the Megginnis Arm Basin based on the current conditions. This model is built via XP-SWMM32 (Version 6.02) which utilizes the mathematical engine of **EPA** SWMM 4.04. The RUNOFF layer is used to simulate the hydrologic responses of the basin and subsequently generates outflow hydrographs. These hydrographs are then routed by the TRANSPORT layer downstream through the drainage network. Our model is carefully calibrated and verified with the hydrologic data and concurrent rainfall data for 23 sub-catchments in the study area.

## INTRODUCTION

The Lake Jackson basin is situated in the Tallahassee Hills upland area in northern Tallahassee and west central Leon County, Florida. With an estimated drainage area of approximately 28,000 acres, the drainage basin consists of Lake Jackson, Lake **Carr,** Mallard Pond, Holley Pond, and land areas that drain into these lakes. Soils are predominately sandy loams to clay loams and most of the northern portions of the basin are heavily vegetated forests and pasturelands. The southern portion of the Lake Jackson have been the main receiving waters for most of the stormwater which runs off the more densely populated areas in the basin.

Lake Jackson is the largest lake in the Basin with a surface area of approximately 4,000 acres at a water elevation of **87** feet. Lake Jackson water level elevations have historically ranged from 75 feet NGVD with most of the lake bottom exposed to **96** feet NGVD at the highest flood stage observed. These water level fluctuations have been a critical factor in the management of the lake environment and flood control of the basin. It is believed that the lake stage fluctuations are primarily dominated by climatic conditions and sinkhole activity caused by a series of underlying geological processes and the interactions with groundwater at the bottom of the lake. Statistical analysis on the historical records of climatic data and lake level data reveals a strong positive correlation that can be used to derive future likelihood of lake response with respect to climatic conditions.

Lake Jackson has been considered a priority water body under the Northwest Florida Water Management District's SWIM Program and is classified by the State of Florida as an Outstanding Florida Waterbody (OFW) and an Aquatic Preserve. However, urban and suburban developments in the southern portions of the Lake Jackson watershed, including Megginnis Creek and its tributaries, have resulted in significant contributions of stormwater pollution. Poor water quality was frequently detected along with increased sedimentation, contamination of bottom sediments by heavy metals and other pollutants, and increased nitrification of the lake as a result of stream pollution. Since late 1970s, the lake has been given lake protection status under the Tallahassee-Leon County 2010 Comprehensive Plan and local environmental ordinances. Several stormwater management facilities were installed, including the City-owned Boone Boulevard pond and the John Knox Road facilities, and the 1-10 pond and the Stormwater Treatment Facility managed by the District. The District facility was further enlarged in the late 1980s to accommodate more runoff. It is expected that these facilities help attenuate peak runoff flow during large storms events while providing longer detention time to improve water quality.

Previous study on water quality and quantity in the Lake Jackson basin can be obtained from the Storm Water Management Plan for the Lake Jackson Basin developed by the NWFWMD (Bartel et al., 1991) for the City of Tallahassee and Leon County. The District further developed the Lake Jackson Regional Stormwater Retrofit Plan (Bartel et al., 1992) in 1992. Hydrologic models were also developed for the watershed using **EPA** SWMM in conjunction with HEC-2's backwater analysis. While these models provided calibrated hydrologic parameters for the basin, most of them were built from later 1970s to early 1980s, whose conditions differ drastically from the post-development conditions.

The goal of this work is to develop a comprehensive hydrologic-hydraulic model for the diagnosis of the Megginnis Arm sub-basin under current post-development conditions. Both field observations and previous modeling results indicated that the Megginnis Arm sub-basin was the flashiest sub-basin with the highest peak flow rates and storm volumes (Bartel et al. 1991). Because of the change of the urbanization characteristics of the basin, its hydrology as well as the drainage network in the area needs to be reexamined. The inclusion of John Knox ponds is expected to attenuate the peak runoff flow downstream and we hope to quantify the efficiency of these ponds statistically against the measured stage levels of the District's Storm Water Treatment Facility, and eventually be able to determine how often water tends to bypass the District pond. The model is expected to perform both flow and pollutant routing effectively while taking advantage of field measurements of stage/discharge and water quality in calibration and verification. Utilizing the XP-

SWMM32™ graphic interface, this model incorporates SWMM's RUNOFF, TRANSPORT, and EXTRAN layers interactively.

Reported in this paper is the model development in its early phase. **A** two-layer model, e.g., RUNOFF-TRANSPORT, has been developed and successfully applied in both storm-event and long-term simulations. Model hydrology is carefully calibrated and verified against measured data.

**Study Site**

Drainage in the 2,230-acre Megginnis Arm basin is controlled to various degrees by ditches, paved channels and detention ponds managed by the City of Tallahassee and the Northwest Florida Water Management District. The sub-basin has historically exhibited higher peak flow rates and relatively greater stormwater volumes than elsewhere in the Lake Jackson basin. It is also the site of a multimillion-dollar experimental water quality control facility consisting of detention ponds and an artificial marsh.

Land uses characters for Megginnis *Arm* Basin are delineated as Low-medium density residential to High density residential with a significant amount of land for commercial uses. Modern day land uses information is available in the form of digital maps based upon 1989 remote sensing data according to Level III of the Florida Land Use Cover and Forms Classification System (FLUCCS) developed by the Florida Department of Transportation. It is highly urbanized (almost 90 percent), consisting of residential areas, apartment complexes, office parks, commercial areas (including three large shopping malls), and two schools. Interstate 10 traverses the sub-basin of the tributaries, contributing additional amounts of stormwater runoff to the system.

The sub-basin water budget consists of a balance of the volumes of water associated with each of the components of the hydrologic cycle. Generally speaking, the total volume of surface water runoff from the catchment is a direct function of precipitation, evapotranspiration, land infiltration, and ground water inflow and groundwater outflow. Evapotranspiration and rainfall are temporal variables that are related to the climatic changes over time. Statistics derived from long-term precipitation data (1958-1999) collected at hourly intervals at the Tallahassee Municipal Airport indicate an annual average rainfall of 64.59 inches. On a monthly basis, July is the wettest month and has the most intense rainstorms on average, On the contrary, the month of October is the driest and December is the month with the lowest average rainfall intensities, Long-term daily pan evaporation data available from the Jim Woodruff **Dam,** northeast of Tallahassee, for the period from 1959 to 1976 indicates an annual average pan evaporation of 65.86 inches. June was the month with highest average **pan** evaporation of 7.8 inches, whereas December the lowest at 2.5 inches. Actual evapotranspiration (ET) is defined as the total amount of water removed from **an** area by transpiration and evaporation. Actual ET is commonly estimated from pan evaporation through a factor or factors that reflect(s) the general properties of the land and vegetative covers. For this area, long-term estimates of average annual evapotranspiration losses have been estimated to range from 35 to 45 inches per year depending upon soil types and vegetative covers,

Previous modeling studies indicate that only about 10 percent of the total volume of water entering the system became direct runoff into Lake Jackson. The remaining 90 percent are lost to evaporation and infiltration. For this study, a number of rainstorms in the late 1970s **and** early 1980s were used in model calibration and verification. The hydrologic model was also applied to a ten-year rainfall record from 1991 to 1999.

## Hydrologic Model Description

The model of the Megginnis *Arm* sub-basin is a quantitative description of the basin hydrology and drainage network hydraulics. It is an assemble of tremendous amount of data either gathered directly from existing database such as rainfall, flow measurements, and evaporation, or indirectly derived from analysis of the characteristics of the basin. The aim of the assemblage is intended for the model to be a principal tool for stormwater management in the area. The hydrologic/hydraulic model is developed and calibrated using the XP-SWMM32™ (version 6.02) that utilizes the **U.S.** EPA's Stormwater Management Model (SWMM) as its mathematical engine. Figure 1 shows the basin delineation as well as its location relative to the city. Figure 2 shows the station network for the Lake Jackson Stormwater Monitoring Project. Stage/discharge measurements obtained at some of the stations were utilized in model calibration and verification. In particular, station S64 provides stage measurements for the NWFWMD's facility in **5-** to 10-minute time intervals, The model starts at the weir (cross-section number **3680)** of the NWFWMD's Facility. The simulated hydrographs at this location are hence the inflow hydrographs to the facility and are hydraulically correlated to the stage fluctuations in the facility,

Figure **3** schematically depicts the runoff elements, major channels, and storage elements configured in the SWMM model. The basin is divided into **23** runoff sub-catchments. SWMM RUNOFF layer reads rainfall data and calculates runoff for each sub-catchments. The TRANSPORT layer subsequently routes flow down stream through the drainage network. Megginnis Arm Tributary 1 begins at the confluence with Megginnis Creek just south of Sharer Road. It crosses North Monroe Street and extends north through the Town and Country neighborhood. The second tributary to Megginnis Creek originates behind Northwood shopping center and runs along Boone Boulevard where the Boone Boulevard stormwater facility is located. Inflow to and outflow from the facility were measured along with the stage measurements for a number of years. These data were used to calibrate the hydrologic model at this location. The SWMM model reported in this paper is two-layer (RUNOFF- TRANSPORT). Inclusion of the EXTREN layer is currently under testing.

Table 1 lists the major parameters used in the hydrologic model. Most of these parameters were assembled from existing information such as physiographic, land use, and climatic data, and from regional regression equations for ungaged sub-basins (Bartel et al., 1991).

**Table 1.** Runoff layer parameters for Megginnis **Arm** Basin.

| PARAMETERS | VALUE |
|---|---|
| Area (total) | 2,230 acre |
| Percent imperviousness (average) | 29.77 |
| Slope (average) | 0.02936 |
| Manning's roughness (average) | |
| Impervious | 0.0175 |
| Pervious | 0.30 |
| Depression storage (average) | |
| Impervious | 0.02 in. |
| Pervious | 0.30 in. |
| Green-Ampt parameters (average) | |
| Suction | 6.1509 in. |
| Saturated hydraulic conductivity | 0.2259 in./hr |
| Initial moisture deficit | 0.35 |

The Green-Ampt infiltration model was assumed to simulate infiltration and the necessary parameters were estimated by identifying the soils in the basin (primarily sandy) from a county soil survey map. Hydraulic conductivity and capillary suction data for each soil were obtain from published data by Carlisle et al. (1981) and work by Bedient and Huber (1988). Manning's $n$ values were selected from charts based on average type of ground cover.

## RESULTS AND DISCUSSIONS

Rainfall-runoff data for calibration and verification were obtained from records gathered at NWFWMD's weather and rainfall stations. Eight major storms were selected in the late 1970s and early 80s. Figure 4 shows a verification run for a storm on 12/06/1979 at cross-section 3680 under pre-development conditions. Rainfall record for the storm is also shown in the figure. It is noted the cross-section 3680 is located at the very downstream of the entire simulated basin and is the weir of the NWFWMD's Stormwater Treatment Facility. Figure S compares the simulated hydrographs at this location for the same storm under pre- and post-development conditions. Statistics for these hydrographs are listed in Table 2. The hydrologic simulation reveals that a 24.4 percent attenuation in peak flow was resulted due to the addition of the stormwater management facilities in the basin.

**Table 2.** **Summary** of the statistics of the simulated hydrographs at cross-section **3680** for storm on 12/06/1979 under' pre- and post-development conditions.

| | Pre-development | Post-development |
|---|---|---|
| Average flow (cfs) | 100.187 | 99.945 |
| Flow standard deviation (cfs) | 8.327 | 7.516 |
| Maximum Flow (cfs) | 441.935 | 334.286 |
| Runoff Vol. (cubic feet) | 5.68 $10^6$ | **5.67** $10^6$ |

In Figure 6, a 12-month continuous simulation for the year of 1991 was performed and the simulated hydrograph at cross-section 3680 is plotted against rainfall. Also plotted is the measured stage record at station S64. The elevation of the top of the emergency spillway is at 101 feet NGVD. The figure indicates a strong correlation between rainfall, the inflow at section **3680** and the stage level. Frequency analysis on the hydrographs will be conducted and is believed to be of great importance to the management of the functioning of the facilities.

## CONCLUSIONS

**A** distributed stormwater management model has been developed for the Megginnis Arm Basin and the framework is demonstrated in this paper. Preliminary results presented here indicate that the hydrologic/hydraulic model functioned correctly in both event-based **and** long-term continuous simulations. The study quantitatively verified that the installation of the stormwater treatment facilities effectively attenuates the peak flow during major storm events. One of the future tasks is to perform additional statistical analysis on the simulated hydrologic time series. Such analysis is necessary to the discovery of important cross-correlation between the hydrologic response of the system and the climatic conditions, which is informatively useful to the stormwater management of the basin.

## ACKNOWLEDGMENTS

The authors wish to express their gratitude to Mr. Ronald Bartel for his constant guidance on this work. Nick Wooten provided field measurements in the Lake Jackson monitoring projects. Jim Dukes produced **GIS** figures at the Northwest Florida Water Management District. We also like to *thank* Anna Blalock for her valuable participation in this project.

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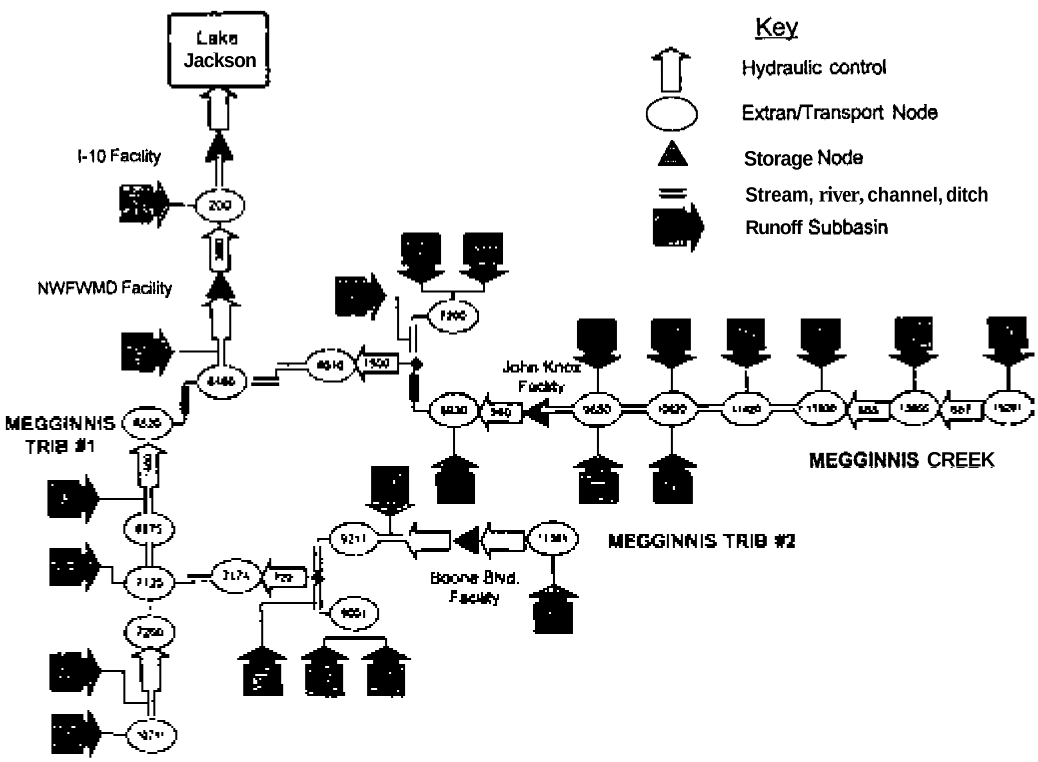


Figure 3. XP-SWMM32 model diagram for Megginnis Arm Basin

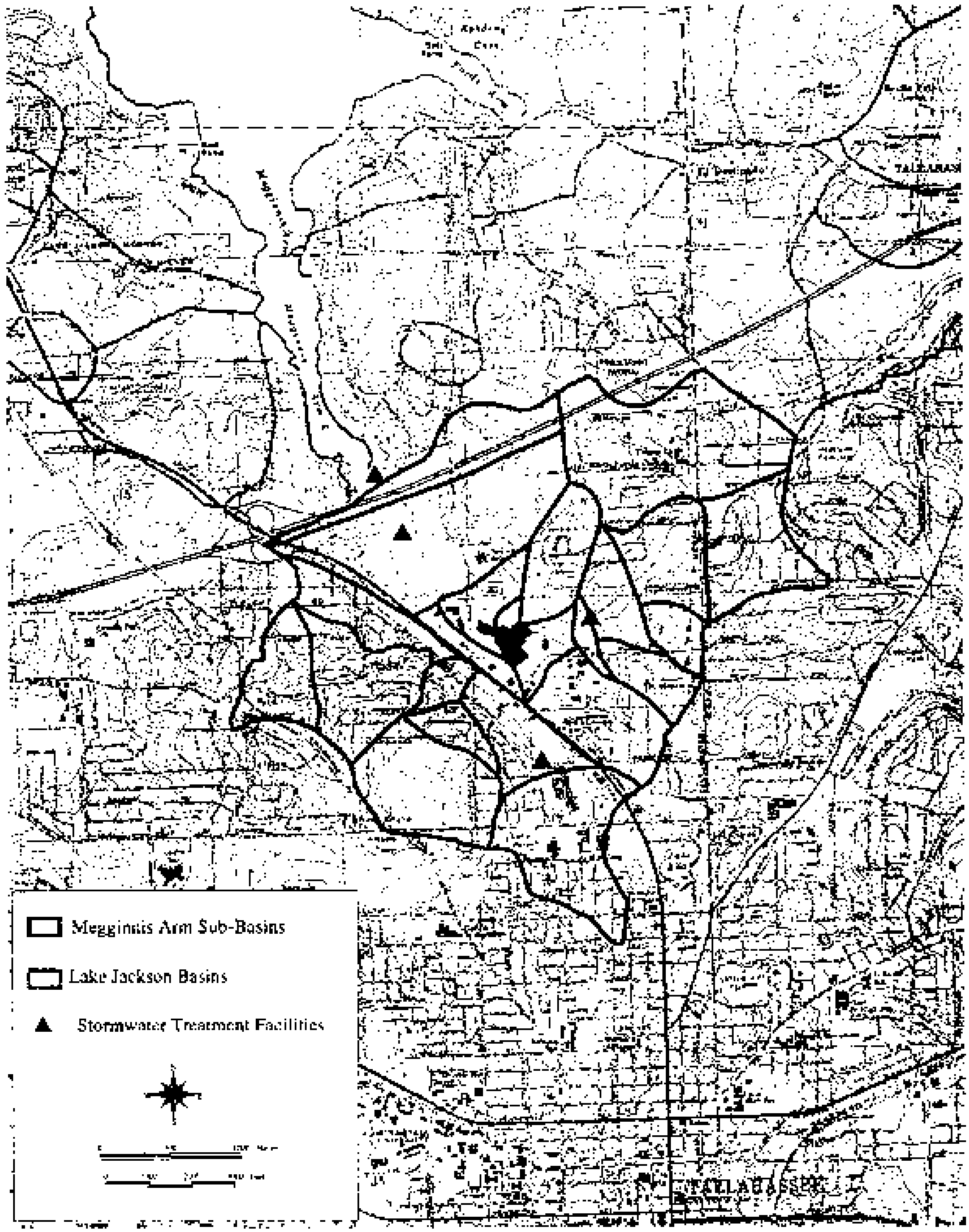


Figure 1. Lake Jackson stormwater monitoring project stormwater treatment facilities

08-01-002

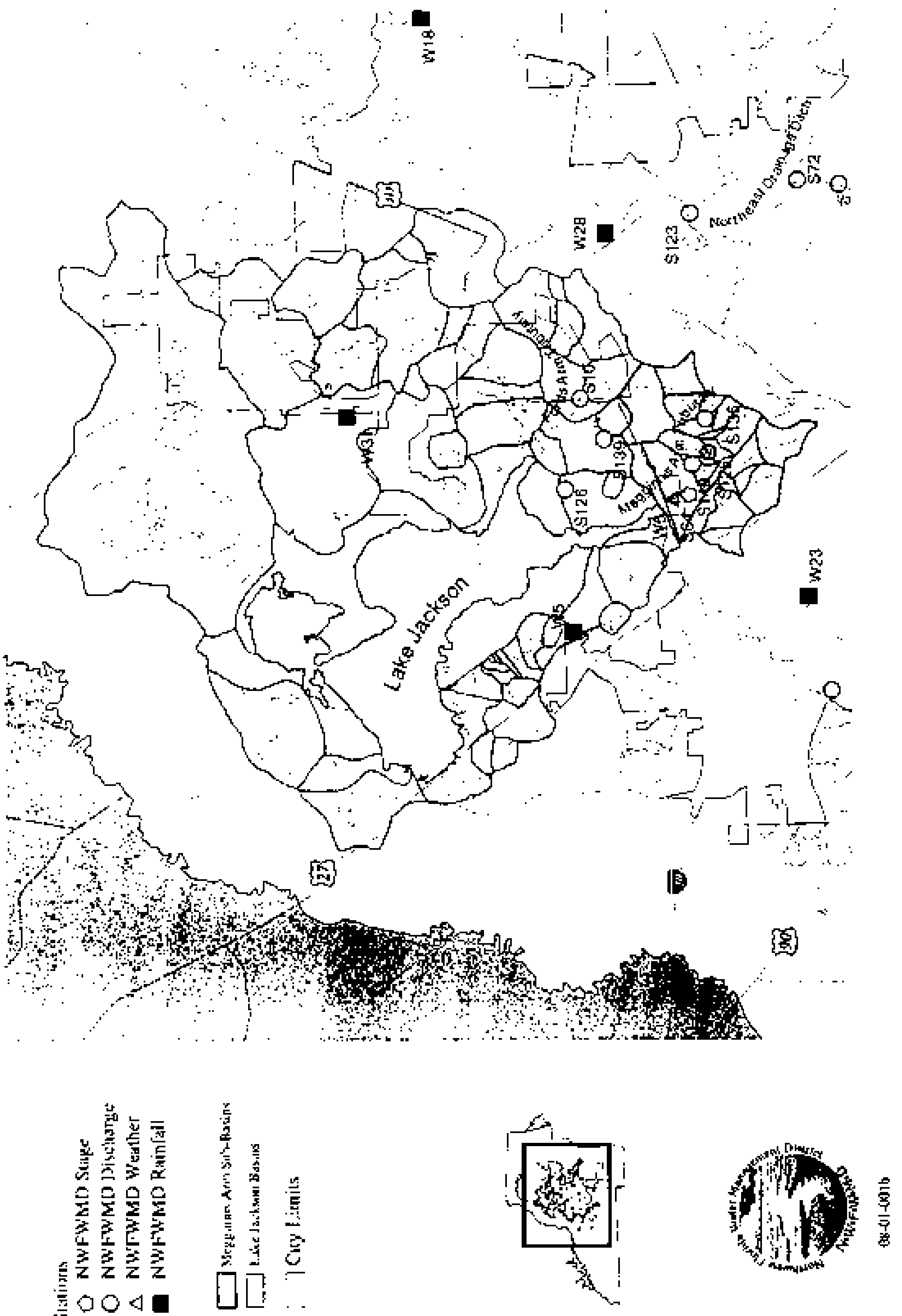


Figure 2. Lake Jackson stormwater monitoring project

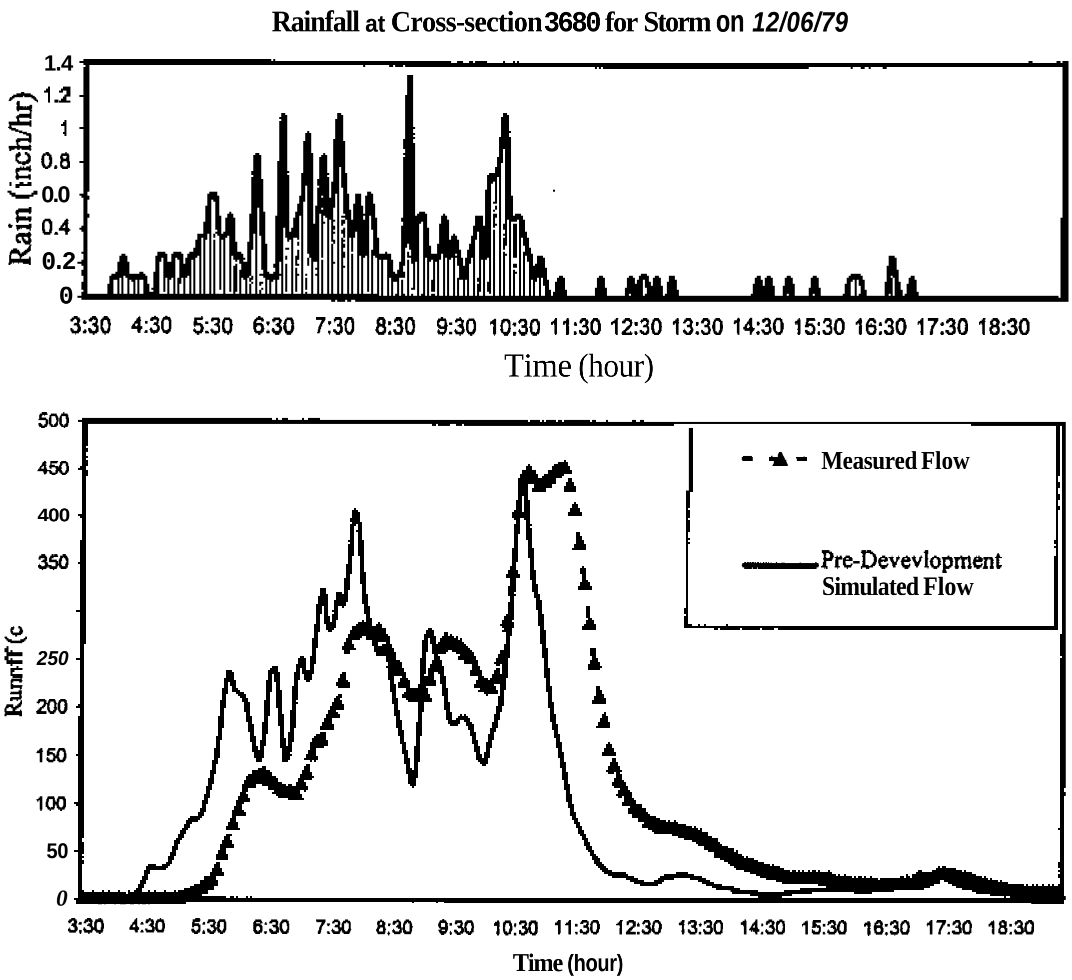


**Figure 4.** **Verification** run for storm **on** 12/06/79 **at cross-section 3680,** storm **Starts at** 03:25.

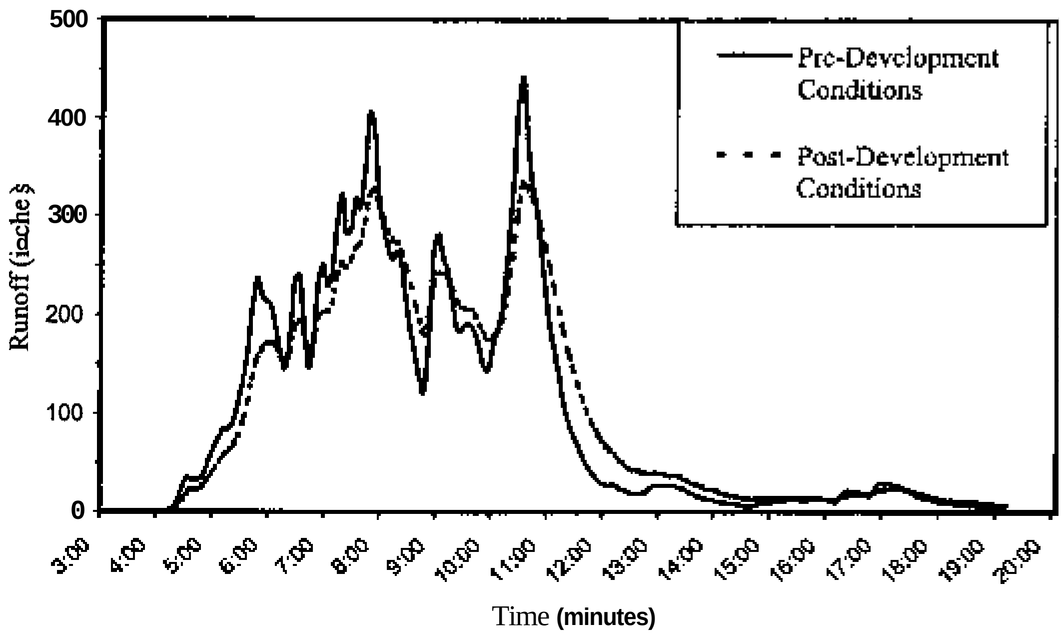


Figure 5. Comparison of **simulated hydrographs at cross-section 3680** for storm **on 12/06/79 under pre- and post-development conditions, storm starts at** 03:25.

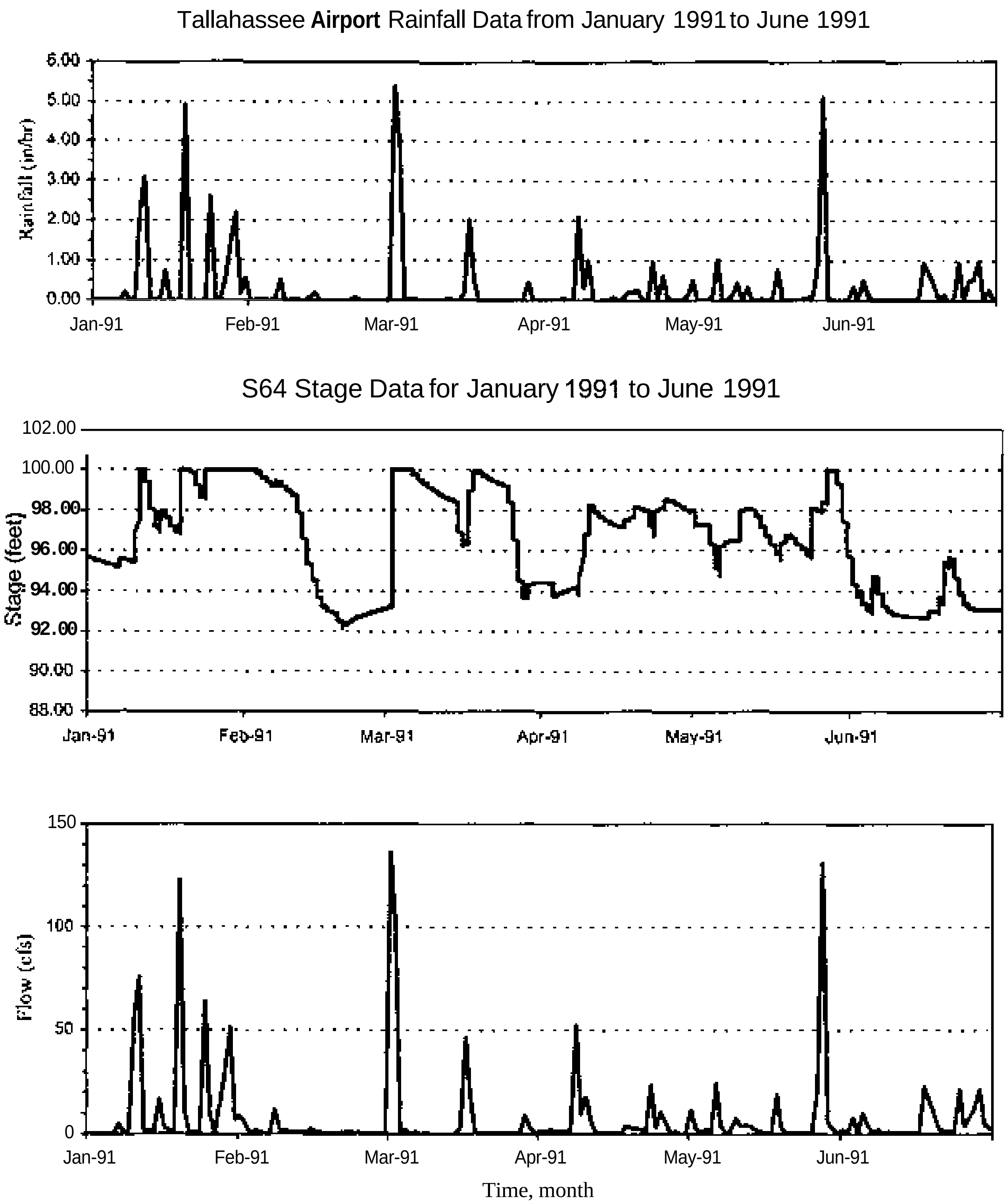


Figure 6. Simulated storm hydrograph for cross section 3680 from January 1991 to June 1991

# HYDROLOGIC MODELING OF RECLAIMED PHOSPHATIC CLAY SETTLING AREAS

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## ABSTRACT

Presently operating Central Florida Phosphate mines comprise about 57,000 acres of Clay Settling Areas (CSAs) and 20,000 acres designated for future CSAs. Current estimates indicate that 102,000 acres of the Peace and Alafia River watersheds are comprised of CSAs. This accounts for 10 to 15 percent of the combined Peace River watershed above Zolfo Springs, the North Prong of the Alafia River above Keysville, and the South Prong of the Alafia River above Lithia. Since hydrologic monitoring efforts indicate that CSA's function much differently than natural or urban areas, restoring the hydrologic function of reclaimed settling areas is critical to establishing a viable hydrologic regime in Central Florida.

To further evaluate the unique characteristics of these systems, the Florida Institute of Phosphate Research (FTPR) sponsored a three-year project to monitor hydrologic and meteorological conditions, calibrate hydrologic simulation programs, collect soils and topographic data, and run clay consolidation models. Results from the investigation were used to determine the impact of continued clay consolidation on **CSA** hydrology; recommend methods for integrating clay consolidation and surface water hydrologic analyses of CSAs; and develop guidelines for simulating the hydrology of **CSAs** such that post-reclamation designs provide low-flow and storm runoff characteristics that more closely mimic the pre-mining behavior of the CSAs. This report provides some results dealing with model representation of event storms on CSAs.

## INTRODUCTION

Approximately one-third of Central Florida's phosphate matrix consists of fine-grained clay-sized materials that **are** able to pass through a minus 150-mesh screen. During the phosphate ore beneficiation process, the fine-grained material is separated from the coarse-grained sand and phosphatic material. The fine-grained material (clay) is pumped to above grade impoundments as a dilute slurry. Upon completion of clay filling, quiescent consolidation, and mechanical dewatering, the **CSAs** are typically reclaimed by flattening the outside slopes of the embankments, minor interior grading and shaping, and revegetation. Typical post-reclamation land uses include pasture, silviculture, habitat areas, row crops, and wetlands. Final reclamation also includes breaching the embankment and constructing **an** outfall to enable controlled surface water discharge.

Restoring the hydrologic function of reclaimed settling areas is critical to establishing a viable hydrologic regime once mining and reclamation are complete. In addition, revegetation planning is crucial to the development and propagation of wildlife corridors and maintaining consistency with

Integrated Habitat Network (IHN) concepts.

Current regulatory emphasis regarding the discharge of water from clay settling areas centers primarily on protection of downstream properties from flooding. **As** such, the regulatory agencies customarily require that the hydrologic system response is evaluated only for large infrequent storm events (25 and 100 year return intervals). However, this emphasis results in conservative hydrologic analyses and often times minimal or no flow from the CSA. Furthermore, the regulatory agencies require that the starting water surface elevation for event modeling be equal to the weir crest elevation without providing justification that the system will actually achieve that elevation during normal rainfall conditions. Observations at some reclaimed CSA's indicate that normal water levels remain two feet or more below the weir crest with very limited surface water discharge.

The primary objective of the study was to develop procedures for predicting the hydrology of above ground clay settling areas that directly considers the short and long term effect of clay consolidation. This report provides some results dealing with representation of event storms on CSAs. **A** more detailed description of the investigation and its results can be found elsewhere (BCI Engineers and Scientists, Inc., In press).

## MATERIALS & METHODS

Three sites in Polk County were selected for use in this investigation:

- IMC-Agrico's Achan 10 (IMC),
- Estech General Chemical Company's SA-10 (Estech), and
- Williams Acquisition Holding Company's AC-OP-06 (Williams).

**Soil Sampling and Testing:** Soil samples were collected at each of the three study sites to identify the subsurface lithology and to characterize the sequences of clay deposition and reclamation. Using a hand auger, surficial clay and overburden samples were collected for laboratory analysis at several locations within each of the three CSAs. The soil samples were collected to depths of approximately six feet below ground surface (bgs), In addition, threaded, 3/8-inch diameter, steel probe rods were used to determine the thickness of the surficial clay deposits. Soil probing and sample collection was completed at the three CSAs in November 1995.

At the IMC, Estech and Williams sites, a hydraulic drill rig mounted on tracked or low ground pressure equipment was used to collect subsurface clay samples to depths of 40 feet bgs. The clay samples were collected for laboratory analysis and to delineate settling area bottom topography. Drill rig soil sampling was completed at the three Polk County CSAs in January 1996.

The soil samples were collected in jars and returned to the BCI soils laboratory to analyze the physical characteristics of the materials. Minus 140-mesh sieve and moisture content tests were conducted on the surficial clay and overburden samples to determine the coarse and fine fraction percentages and the solids contents of the materials. In addition, Atterberg Limits and Restricted Flow Consolidation tests were conducted on selected clay samples to determine consolidation characteristics. Moisture content tests were conducted on the clay samples collected during drilling operations to determine solids content profiles with depth.

**CSA Modifications:** Several improvements were made to the outfall configuration and interior of the sites to improve the ability to rate and monitor the volume and flow rate of water discharging from the project sites. These improvements maintained the overflow elevation while reducing the outfall cross section at the expected low flow stage condition. The resulting discharge at extreme low flow conditions was better defined and measured. All proposed improvements were reviewed and approved by South West Florida Water Management District, SWFWMD, prior to installation.

An existing discharge control structure at the IMC **CSA** was used without modification; however, some work was done immediately upstream of the outfall to improve the connection of the principal ponding area with the discharge control section.

The Williams CSA has two existing, concrete, drop inlet structures that did not require modifications for use in this project. However, the site was modified to create two distinct drainage basins. The intended design of the Williams site called for two outfall points with only one drainage basin, To define the contributing basin area reporting to each outfall, a basin divide was created by bisecting the site with a berm constructed prior to initiating monitoring activities. The location of the divide was carefully selected and followed existing exposed spoil rows where possible.

The existing outfall at the Estech CSA consisted of a partially eroded open channel conveyance. To improve the stability of this outfall, a weir was constructed in August 1995. The weir consisted of a v-notch, concrete overflow and was constructed within the confines of the existing outfall ditch conveyance.

**Aerial Photography and Topographic Elevations:** A reconnaissance of each **CSA** was conducted to determine the nature of vegetation and the impact vegetation would have on topographic mapping. Based on the evaluations, mowing was conducted at the Estech site prior to completing aerial photography, The purpose of the mowing was to remove standing vegetative cover that would interfere with interpretations of the stereoscopic images obtained during subsequent aerial photography. Aerial photographs of each of the three sites were obtained at the beginning and at the completion of field investigations.

Topographic maps of each site were constructed from photogrammetric interpretations of the stereoscopic images obtained during aerial photography. Each of the maps was prepared using one-foot contour intervals. The initial topographic maps were used to establish the baseline depressional storage and runoff characteristics for each of the sites. Comparisons of the initial topographic maps with those prepared at the end of field monitoring activities allowed quantification of continued clay consolidation effects on the CSA topography. Digital terrain mapping software was used to compare the digital files generated from photogrammetric mapping and define the changes in surface topography and associated storage volumes.

**USGS Data Collection: As part** of this cooperative investigation, the **USGS** collected hydrologic and climatic data for a 2-year period at each **CSA.** Data collected during the period from September 1996 through September **1997** were common to all study areas. The data collection network at each site included: stream flow, pond stage, periodic and continuous recording water levels in wells, rainfall, wind speed and direction, water temperature, relative humidity, air temperature, and pan evaporation. Results of the USGS monitoring data are summarized elsewhere (USGS 1999).

Climatologic instrumentation installed at each of the **CSAs** included: a tipping-bucket rainfall gage, wind speed and direction sensor, air temperature sensor, and a relative humidity sensor. Pan evaporation gages were also installed at the IMC **CSA.** Automatically recorded hydrologic and climatological data were digitally collected in 15 or 60-minute intervals. The data collection period at the IMC and Estech sites was from August 1995 to November 1997, and at the Williams sites from April 1996 to September **1998.**

In addition, the USGS collected rainfall, pan evaporation, and other climatic data at an off-site station near the Bartow Airport, located in central **Polk** County. This station is referred to throughout the remainder of this report as the Polk County Weather Station (PC Weather Station). This station was used to compare climatological data collected at the three Polk County CSAs with natural background conditions.

At each of the sites, streamflow and pond-stage data were collected by electronic data-loggers that recorded water-level elevations in a stilling well from the rising and falling of a float. Discharge at each basin was monitored by a streamflow gage at the outfall of each CSA. One to three water level stage gages were installed in each basin to monitor pond fluctuations. The relationship between stage and discharge at the outfall(s) of each **CSA** was determined in the field.

Surficial aquifer ground water levels were collected continuously in each basin from a monitor well drilled within the interior of each **CSA** basin, and from a second monitor well drilled within one of the perimeter dams. Groundwater levels were also collected at approximate monthly intervals from a network of nine to 16 wells set within the perimeter dams. The wells were constructed of two-inch diameter polyvinyl chloride (PVC) casing and screen, and set at depths ranging from 10 to **33** feet bgs. These wells were installed by auguring through the upper clay crust, then hand-driving the PVC tubes to a depth below the water.

**Event Model Simulations:** BRN, the Basin Runoff Networking program, was used in this investigation and is a proprietary program developed by James Boyd and Russ Ferlita (Boyd, J. J., 1993). In general, the program transforms each one-hour rainfall into several hours of basin runoff, then the runoff within an hour contributed from each hour of rainfall is summed to get a discharge hydrograph. Several common steps were followed in setting up and calibrating the models.

- Sub-basins were delineated from topographic maps.
- Stage, discharge, area, volume, and cross sectional data were tabulated for each channel and reservoir reach utilizing topographic information and field reconnaissance observations.
- Measured rainfall data were digitized and formatted for use in model simulation.
- Observed discharges, flows, and stages were digitized in time series format to enable comparison to model output.
- Model parameters needed to describe sub-basin configurations, runoff, and flow routing were calculated and entered into program input files.
- Models were executed and the results analyzed and compared to observed data.
- Select parameters were adjusted and the simulation was rerun.

In an effort to define the effect of altering the basin configuration and limiting parameter adjustments the Estech **CSA** was used to evaluate the sensitivity of subdividing a CSA into smaller numerous basins.

Table 1 summarizes the physical based characteristics used in the initial setup of the hydrologic models for each CSA. These parameter values were estimated using thematic maps including topographic, soils and land use. Table 2 provides the values used to calculate Soil Conservation Service, **SCS** runoff curve numbers for each **CSA** based on calculated soil and land use complexes and published curve number tables. Using this method, the estimated curve number ranges between **82** and 96.

**Table 1. Clay Settling Area Characteristics Used in Hydrologic Modeling**

| | **IMC** | **Estech** | **Williams** |
|---|---|---|---|
| Basin Area (acres) | **370.8** | 69.5 | 448.0 |
| Reach Area (acres)' | **78.5** | **0.3** | 19.4 |
| Average Hydraulic Length (feet) | **1870** | **988** | 4580 |
| Average Hydraulic Slope (percent) | 2.1 | 4.9 | 0.8 |
| Time of Concentration (hours) | 0.81 | **0.32** | 2.65 |

'The basin area does not include the area of the reach

Rainfall events used for calibration range from 3.55 inches to **5.37** inches. The peak hourly rainfall ranged from 0.37 to 2.47 inches, and this peak hour occurred between 29 and 54 hours after the start of the period used in the simulations. The peak discharge ranged from about **2 to** 5 cubic feet per second (cfs) for rain events used in calibration. Discharge from the **CSA** continued from between **97** and **408** hours or from 60 to **230** hours after the peak hour of rainfall. Table **3** lists characteristics of the rainfall events used in calibration, and Table 4 lists characteristics of the discharge from the **CSA** during these events.

Results are provided in this report using BRN for calculating the runoff response from the CSAs based on the **SCS** method of calculating excess rainfall, the lag method of calculating the time of concentration, and the unit hydrograph method of calculating the runoff hydrograph.

The curve number method of estimating runoff is most appropriate for mid-sized basins, for which the discharge is not dominated by channel storage processes (Ponce 1989). For BRN, separate representation of the channel storage component overcomes limitations using the curve number method alone. To meet the Florida Department of Environmental Protection guidelines, the discharge volumes and peaks are usually adjusted during reclamation design by including a pond **and** control structure at the outfall for the CSA. So, it is important to incorporate channel storage processes in the model representation of the CSA system hydrology.

**Table 2. Estimated Curve Numbers For Clay Settling Areas (CSA) Based On Landuse and Soils**

| CSA | FLUCCS and Landuse | Soil | Hydro. Group | Approximate Description | Curve No. | Area (acres) |
|---|---|---|---|---|---|---|
| IMC | 310 Herbaceous | >6 ft of Sand/Overburden | B | Meadows, Good | 58 | 36.8 |
| | 310 Herbaceous | 0-6 ft of Sand/Overburden | C | Meadows, Good | 71 | 79.6 |
| | 640 Vegetated Non-forested Wetland | Clay at Surface | D | Lakes & Ponds | 100 | 254.4 |
| | | | | Average | 89.6 | |
| | | | | Pasture, Good | 61 | 18 |
| Estech | 211 Improved Pasture | >6 ft of Sand/Overburden | B | | | |
| | 211 Improved Pasture | 0-6 ft of Sand/Overburden | C | Pasture, Good | 74 | 42 |
| | 640 Vegetated Non-forested Wetland | 0-6 ft of Sand/Overburden | C | Lakes & Ponds | 100 | 20.6 |
| | 640 Vegetated Non-forested Wetland | Clay at Surface | D | Lakes & Ponds | 100 | 19.4 |
| | | | | Average | 82.1 | |
| Williams | 310 Herbaceous | >6 ft of Sand/Overburden | B | Meadows, Good | 58 | 54.6 |
| | 310 Herbaceous | Clay at Surface | D | Meadows, Good | 78 | 24.1 |
| | 534 Reservoirs | 0-6 ft of Sand/Overburden | C | Lakes & Ponds | 100 | 0.5 |
| | 621 Cypress | 0-6 ft of Sand/Overburden | C | Lakes & Ponds | 100 | 0.4 |
| | 640 Vegetated Non-forested Wetland | 0-6 ft of Sand/Overburden | C | Lakes & Ponds | 100 | 46.7 |
| | 640 Vegetated Non-forested Wetland | Clay at Surface | D | Lakes & Ponds | 100 | 341.1 |
| | | | | Average | 94.0 | |

**Table 3. Characteristics of the Rainfall Event Used For Calibration**

| Basin Name | Starting Date | Peak Hour (hours) | Peak Rate (inches/hour) | Total Rainfall (inches) |
|---|---|---|---|---|
| Achan | 12/31/95 | 32 | 1.21 | **5.18** |
| Estech | 12/31/95 | 29 | 0.37 | 3.55 |
| Williams | 06/09/97 | 54 | 2.47 | 5.37 |

**Table 4. Characteristics of the Discharge for the Event Selected for Calibration**

| Basin | Peak Hour (hours) | Peak Rate (cfs) | Total Discharge (inches) | Total Hours[1] |
|---|---|---|---|---|
| Achan | 37 | 3.96 | 0.79 | 168 |
| | | | | 408 |
| Williams | 101 | 4.90 | 2.47 | |

'Hours after the rain event begins included in simulation

## RESULTS

**Single Basin Representation of CSAs:** In BRN, the discharge rate must increase with each row of the table. For most observed rainfall events, there is some storage within the pond to fill prior to discharge, This cannot be represented in BRN very well, since two stages cannot be specified in adjacent rows with a discharge of zero. In some cases, a very small discharge rate was listed in the table for pond elevations below the actual outfall elevation.

Table ***5*** lists the BRN calibrated parameter set using the ***curve*** number method of estimating excess runoff. The estimated curve numbers range from about 70 to 72. Figures 1 shows the observed and BRN simulated stages and discharges at the Estech **CSA.**

**Table 5. Calibrated Model Parameters for the CSAs using BRN.**

| Parameter | Estech | Achan | Williams. |
|---|---|---|---|
| Curve Number | 70 | 72 | 70 |
| Initial Abstraction | 0.2 | 0.2 | 0.0 |
| Peak Rate Factor | **382** | 125 | 484 |

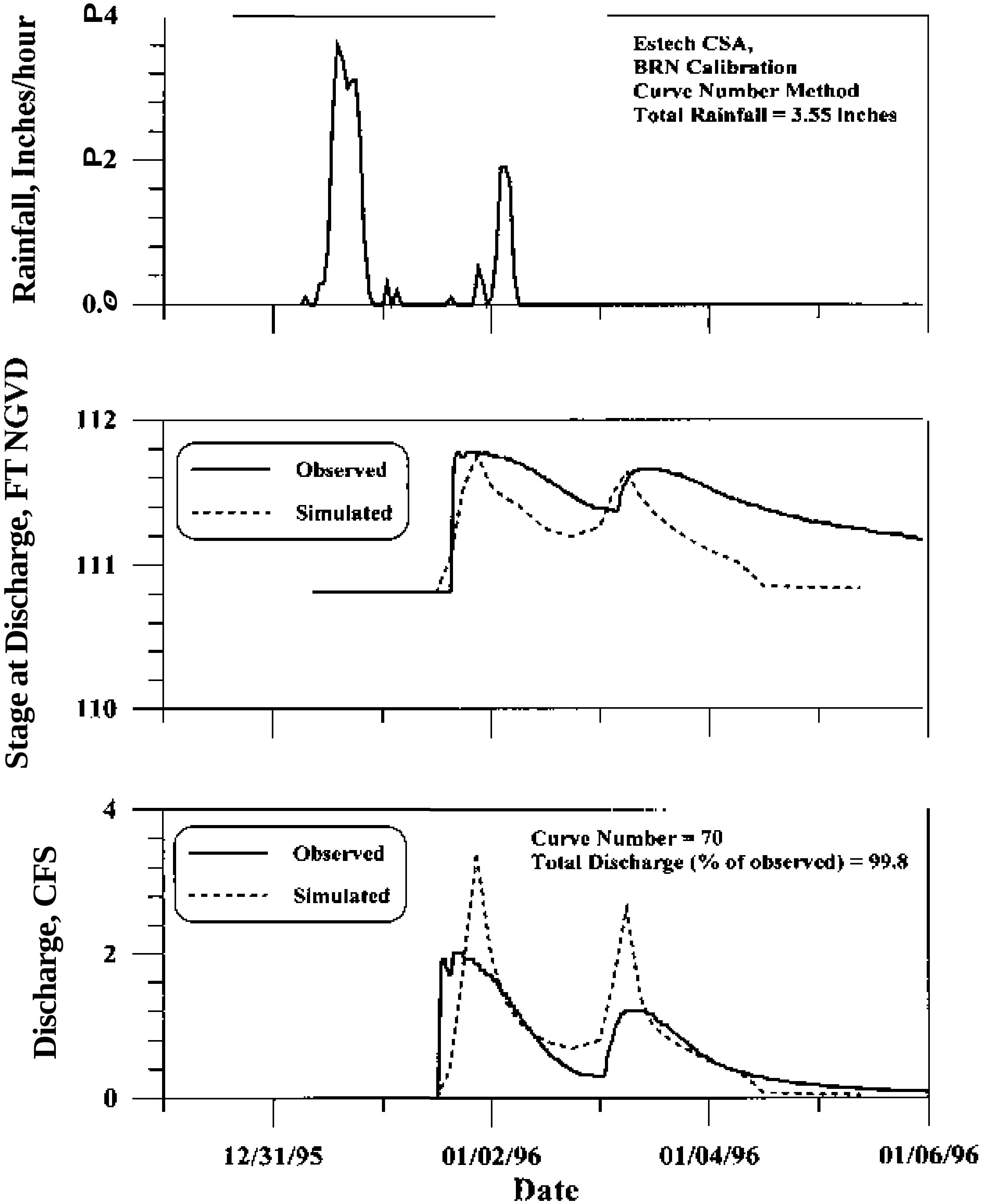


**Figure 1. BRN event storm calibration, simulated stage and discharge at Estech clay settling area**

**Multiple Basin Representation of CSA:** Initially, each **CSA** was represented simply as a basin contributing runoff to a pond that discharges through the outfall. To estimate the importance of the model routing complexity on estimating model parameters, the Estech CSA was divided into five subbasins, each having a minor storage component and calibrated by varying the model parameters. The only observed/measured discharge was that collected for the CSA final outfall. The hydraulic length, hydraulic slope, percent impervious area, available storage, and flow constructions (controlling flow section) for each subbasin were calculated using topographic maps and field reconnaissance observations.

Table **6** lists physically based and calibrated model parameters for each of the subbasins used in the representation of the Estech CSA with multiple basins. The calibrated estimate of the curve number was 75, indicating that there is some delay in the discharge provided by the more complex representation of the **CSA.** That is, the lower curve number estimated for the single basin representation of the CSA is in part to compensate for the detention of water upstream of the principle pond located at the final outfall. Frequently in the CSAs the outfall control is the dominant factor (as compared to upstream routing) affecting the discharge from the CSA diminishing the importance of the multi-basin representation of the CSA. In addition, the difference in estimated discharge using a curve number of **70** and 75 may not be significant when compared to the affects of clay crack formation and clay consolidation,

## DISCUSSION AND/OR CONCLUSIONS

Calibration of the models representing CSAs to an observed rainfall event resulted in an estimated curve number less than expected. The calibrated estimates of curve number ranged between 70 and 72, while those estimated based on soils and landuse ranged between **82** and 94 (Table 2). Though the use of estimated curve numbers based on soils and landuse may be appropriate for designing detention storage preventing downstream flooding, a design based on these curve numbers may result in significant reductions in total annual discharges

By subdividing a **CSA** into smaller units with pond storage and channel routing between the subbasins, the calibrated estimate of curve number increased to about **75. So,** the detail used in representing the CSA can have a significant impact on the estimated peak and volume discharges from the basin. Alternatively, these systems are characterized by changing topography and depressional storage (changes in pond storage and channel routing) which could have a significant impact on discharges out of the **CSA.**

Since clay settling areas make up a large part of the reclaimed phosphate mining area, the volume and character of discharges over the long term are important. This uncertainty can be partly attributed to topography changes during clay consolidation, and the crack formation clays as the clays desicate. This uncertainty in the hydrologic response of these systems as they change through time can be compensated by long term hydrologic monitoring and possible adjustment of discharge controls as a part of reclamation.

**Table 6. Calibrated Model Parameters for the Estech Clay Settling Area Using BRN, Multiple Basins**

| | Physically Based | | | | Calibrated | |
|---|---|---|---|---|---|---|
| Subbasin Number | Area (acres) | Hydraulic Length (ft) | Hydraulic Slope | Percent Impervious | Initial Abstraction Coefficient. | Peaking Coefficient. |
| 1 | 18.4 | 458 | 2.5 | 2 | 0.2 | 484 |
| 2 | 5.6 | 335 | 3.4 | 1 | 0.2 | 484 |
| 3 | 17.9 | 457 | 2.8 | 10 | 0.2 | 484 |
| 4 | 13.3 | 451 | 1.2 | 1 | 0.2 | 484 |
| 5 | 14.4 | 261 | 2.3 | 2 | 0.2 | 484 |

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# STORMWATER GEOGRAPHIC INFORMATION SYSTEM APPLICATIONS IN CENTRAL FLOFUDA

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## ABSTRACT

Recent National Permit Discharge Elimination System (NPDES) permits issued by the United States Environmental Protection Agency (**U.S.** EPA) for municipal separate storm sewer systems (MS4) require some level of routine inspection and maintenance for stormwater systems. Typically, a municipality is required to maintain an internal record keeping system to track inspection and maintenance activities. In order to accomplish this, a municipality needs to have a good inventory of the maintained stormwater system including structure geometry and condition information. Historically, municipalities have relied on primary system inventories completed during a stormwater master planning process or subdivision record drawings. If a municipality relies solely on master plan inventories, the development of a municipal-wide inventory may take many years and is unlikely to be completed in time to help with NPDES permit compliance (i.e. structure ownership, structure maintenance, dry weather field screening of outfalls, etc.).

For these reasons, several municipalities in central Florida have initiated programs to inventory the stormwater structures using a Geographic Information System (GIS) tailored to their specific needs. These municipalities will select appropriate stormwater structure maintenance levels of service (LOS) based on their stormwater structure inventories. The associated GIS will be used in the development and implementation of maintenance programs to facilitate meeting NPDES permit requirements as well as to increase the effectiveness and operable life of stormwater facilities, This paper discusses the development of a stormwater GIS for Brevard County, Florida and how this GIS is being used to enhance their maintenance program and help prepare for their expected NPDES permit requirements.

## INTRODUCTION

EPA recently issued draft amendments to the Clean Water Act in Section 40 Code of Federal Regulations (40 CFR) Subsection 122.26 to include small municipal separate storm sewer systems (MS4's) into the NPDES permit program. The state of Florida will administer the stormwater NPDES permit program in the near future and will likely issue a state-wide general permit for small MS4's. Small municipalities will be able to gain coverage through the general permit via a Notice of Intent. The fundamental goal of the permit will be that they develop a stormwater management program which controls stormwater pollution to the maximum extent practical. **As** a minimum, the management program must have the following aspects:

- Public education and outreach;
- Public involvement and participation;
- Illicit discharge detection and elimination;
- Construction discharge controls;
- Runoff retention for development and significant redevelopment; and,
- Municipal operations and pollution prevention.

Although the permitting requirements will not be as stringent as those for more populous counties, it is clear from review of the permits being issued around the state that many municipalities will be required to perform additional maintenance on their stormwater systems, Another expected requirement will be documentation of maintenance activities **and** a demonstration that the maintenance program is effectively working to reduce pollutant loads to waters of the United States.

For most small municipalities, a routine maintenance program for their entire stormwater system would be cost prohibitive and probably unnecessary to meet **EPA** requirements. However, it will be necessary to craft a maintenance program that will meet local demands and future NPDES permit conditions for the **MS4.** For these reasons, a stormwater Operations and Maintenance (O&M) program structured to provide inspections of all facilities according to a fixed schedule and to provide maintenance as needed may be more appropriate. **A** routine inspection program will function as a routine maintenance program but will cost significantly less than a routine maintenance program. Routine maintenance schedules can still be used as guidelines for the overall program. This is **an** efficient way to manage staff time and work efforts while still meeting the intent of the EPA NPDES program.

The remainder of this paper discusses O&M levels of service **and** how municipalities in Central Florida are using Geographic Information Systems **(GIS)** to improve O&M programs including documentation of maintenance completed.

## Levels of Service

Although the operable life of a stormwater facility is generally expected to last several decades or more, lack of maintenance resulting in overgrown vegetation, accumulated sediment and debris, and deteriorated structures can greatly reduce effectiveness. Without regular operation **and** maintenance programs, these facilities may not store, treat, or convey stormwater according to their design, and may require frequent repair or even replacement. Regular maintenance will allow facilities to operate as designed for their maximum lifetime, enabling optimum flood control and water quality treatment as well as demonstrating to the public that stormwater capital investments are being protected in a systematic, responsible and cost-effective manner. However, fiscal constraints often limit the LOS that can be appropriately provided in a specific area.

The highest LOS is a routine O&M program including a scheduled inspection and maintenance program for all stormwater facilities including ponds, culverts, inlets, ditches, and primary channels.

**A** routine O&M program requires a complete inventory of stormwater structures for which a municipality has maintenance responsibility. Additionally, a municipality needs to have access to these structures and have defined maintenance protocol based upon structure type. This LOS requires a municipality to be proactive in addressing potential problems such as cracks in headwalls or box culverts which could lead to deterioration of rebar and the ultimate failure of the structure. In Florida, such routine O&M programs are rare at best and in most cases unnecessary. It has been our experience that most programs are somewhere between purely reactionary, addressing problems only when they become critical, and the routine O&M program described above. The advantage of a more rigorous O&M program is that stormwater facilities are more likely to operate as designed.

Before a LOS can be defined, a municipality must have a good inventory of the stormwater structures it is responsible for maintaining. Historically, stormwater maintenance has been reactionary (complaint driven) in nature with no effort on identification and mapping of stormwater structures under the maintenance responsibility of a municipality. Therefore, decisions regarding the balance between maintenance costs and **LOS** provided could not be effectively addressed. To address this issue, several Central Florida municipalities have initiated stormwater structure inventory programs using **GIS** tools. Once the inventories are completed and LOS objectives defined, the stormwater GIS can be used for maintenance planning and reporting.

## Stormwater Geographic Information System Development

Brevard County is striving to develop a county-wide stormwater GIS to assist with stormwater planning and maintenance activities and making LOS decisions. **As** part of the Brevard County Stormwater Needs Assessment, Camp Dresser & McKee Inc. (CDM) compiled stormwater structure data **and** provided geographic-based information to the Brevard County Surface Water Improvement Division in a GIS, using Microsoft Access970 and ArcView Version 3.10. Brevard County intends to use this GIS stormwater inventory to help develop and implement an ongoing stormwater facility maintenance program to their desired LOS and meet the requirements of their NPDES **MS4** permit.

CDM initially developed drainage basin and primary stormwater management system **GIS** data layers (coverages) of the County using existing data sources. These coverages provided the foundation for the subsequent development of a stormwater structure **GIS** coverage. Discussions of the development of these coverages and the linked database are summarized below.

### Base Map

The digital base map was based on existing coverages obtained from the Brevard County GIS Department and St. John's River Water Management District (SJRWMD). The digital map data obtained included roadway center lines, municipal boundaries, county commission districts, parcel boundaries, rivers, streams, and the edge of water bodies. CDM added basin boundaries and primary stormwater management system coverages to the County base map. The SJRWMD-defined major basin boundaries were used as the starting point for refining basin boundaries to a level of detail suitable for the County's O&M program. Specifically, the original SJRWMD basin delineations do not reflect impacts to the natural stormwater conveyance systems east of Interstate 95 from development. Consequently, CDM revised or subdivided major basin boundaries using available topographic information, completed stormwater master plans, and aerial photographs, The basin coverage was further refined as needed using data gathered under the stormwater structure field inventory.

## Primary Stormwater Management Systems (PSWMS)

The **PS**WMS GIS coverage provides the stormwater network that conveys runoff to receiving water bodies (including waters of the United States). The PSWMS is generally defined as structures with equivalent diameters greater than 24-inches and/or facilities the County has clear maintenance responsibility for. The existing **USGS** hydrology coverage was the starting point for defining the PSWMS because it showed streams, ditches, canals, and shorelines. CDM used the major and minor attributes of the digital hydrology coverage to create a subset of the streams, canals, and ditches estimated to be part of the County's PSWMS. Adjustments to this coverage were also made using aerial photographs and completed stormwater structure inventories.

## S Inventory GIS Design

In **a** parallel effort to the development of the digital map coverages, CDM worked with the County to define the types of structures and associated attribute data to be included in the Stormwater Inventory GIS. The types of structures included are summarized in **Table 1.**

For each structure type, associated attribute data were defined including location information (commission district, section-township-range, basin, state plane coordinates), geometric characteristics (diameters, lengths, elevations, etc.), physical condition descriptors, construction date, planned inspection frequency, last inspection date, and next scheduled inspection date. Once defined, a database was developed using Microsoft Access97©. The database was then linked to the **GIS** coverages developed in ArcView©.

In order to link the **GIS** coverages with the associated database tables, each structure inventoried was assigned a unique identifier. The unique identifier developed for the County included a three-digit numeric value representing each map tile (grid) defined for the project, followed by **a** two-digit alphanumeric value representing the structure type (see Table 1), **and** finally followed by a four-digit numeric value assigned **by** the County based upon the number of each structure type identified on a map tile (ascending order, 0001,0002, etc.).

| Structure Type | Structure Code | Coverage Type |
|---|---|---|
| Bridge | BR | Point |
| Curb Inlet | CI | Point |
| Control Structure | CS | Point |
| Culvert | CU | Line |
| End Structure | ES | Point |
| Grated Inlet | GI | Point |
| Manhole | MH | Point |
| Open Channel | OC | Line |
| Outfall | OF | Point |
| Pond | PO | Point |
| Pump Station | PS | Point |
| Storm Sewer | SS | Line |

**Field Inventory and Database Population**

Two representatives from the Brevard County Surface Water Improvement Division provided the field inventory work. To accomplish this task, the field crews inventoried the PSWMS on a section by section basis, Structures were hand drawn on existing parcel maps of the County plotted out by section. For each structure, the field crews took appropriate photographs and completed the data form shown in **Figure 1.** The data form was developed from the data dictionary previously described. Information from the forms were manually transferred to the Microsoft Access970 database and linked to ArcView©.

**As an** alternative to the "paper data form", CDM has designed digital data forms that can be used with a palm top computer. This type of system was implemented for the City of Nashville, Tennessee, as part of its stormwater inventory GIS. This type of system requires more of an up-front capital investment but eliminates completing paper forms in the field and then manually entering information from the paper forms into the master database. Information is entered into the palm top computer in the field and then electronically transferred to the master database. For large inventory efforts, the digital forms may be more cost effective.

The final **GIS** coverages were linked with the database using SQL Connect, which is **a** standard Arcview tool. **A** representative view of the linked GIS is presented on **Figure 2.** The three main

# Brevard County Structure Inventory Field Sheet

Plan view

N

Date

Field crew

ocation:

'ated Inlets

| Map ID | Sump | Sediment Device | Grate Length (in) | Grate Width (in) | Elevation (FT, NGM) ToP | Elevation (FT, NGM) Bottom | Condition | GPS (X/Y) |
|---|---|---|---|---|---|---|---|---|
| | Yes No | Yes No | | | | | Good Fair Poor | |
| | Yes No | Yes No | | | | | Good Fair Poor | |
| | Yes No | Yes No | | | | | Good Fair Poor | |
| | Yes No | Yes No | | | | | Good Fair Poor | |
| | Yes No | Yes No | | | | | Good Fair Poor | |
| | Yes No | Yes No | | | | | Good Fair Poor | |
| | Yes No | Yes No | | | | | Good Fair Poor | |

urb Inlets

| Map ID | Access* | Sump | Sediment Device | Curb Inlet Length (in) | Curb Inlet Width (in) | Elevation (FT, NGVD) Inlet | Elevation (FT, NGVD) Bottom | Condition | GPS (X/Y) |
|---|---|---|---|---|---|---|---|---|---|
| | | Yes No | Yes No | | | | | Good Fair Poor | |
| | | Yes No | Yes No | | | | | Good Fair Poor | |
| | | Yes No | Yes No | | | | | Good Fair Poor | |
| | | Yes No | Yes No | | | | | Good Fair Poor | |
| | | Yes No | Yes No | | | | | Good Fair Poor | |
| | | Yes No | Yes No | | | | | Good Fair Poor | |

Manhole, Grate (no access), Grate (Access), None

anholes

| Map ID | Baffle Box | # of Chambers | Shape | Material | Elevation (FT, NGVD) Top | Elevation (FT, NGVD) Bottom | Condition | GPS (X/Y) |
|---|---|---|---|---|---|---|---|---|
| | Yes No | | Circ Rect Irreg | Conc Brick Lined | | | Good Fair Poor | |
| | Yes No | | Circ Rect Irreg | Conc Brick Lined | | | Good Fair Poor | |
| | Yes No | | Circ Rect Irreg | Conc Brick Lined | | | Good Fair Poor | |
| | Yes No | | Circ Rect Irreg | Conc Brick Lined | | | Good Fair Poor | |
| | Yes No | | Circ Rect Irreg | Conc Brick Lined | | | Good Fair Poor | |
| | Yes No | | Circ Rect Irreg | Conc Brick Lined | | | Good Fair Poor | |
| | Yes No | | Circ Rect Irreg | Conc Brick Lined | | | Good Fair Poor | |



Field data sheet (front)

End Structures/Outfalls

| Map ID | Type* | Headwall Type | Material** | Condition | Receiving Water** |
|---|---|---|---|---|---|
| | ES OF | | | Good Fair Poor | |
| | ES OF | | | Good Fair Poor | |
| | ES OF | | | Good Fair Poor | |
| | ES OF | | | Good Fair Poor | |
| | ES OF | | | Good Fair Poor | |
| | ES OF | | | Good Fair Poor | |

* Type choices: Headwall; Headwall/Grate; Mitered; Mitered/Grate; Grate; Interconnect; None; and Other
** Material choices; Sand-Cement (Rip-Rap); Concrete; Brick; Coquina; and Other
*** Receiving water choices (applicable for outfalls): Indian River (IR); Banana River (ER): Eau Gallie River **(EER)**: Sebastian River **(SR)**; St. Johns River (SJR); Sykes Creek (SC); Crane Creek (CC), Turkey Creek (TC); M-1 Canal (M-1); South Lake (SL); Washington Lake (WL); Atlantic Ocean (AO); Unnamed Creek (creek); Unnamed Canal (canal), Unnamed Pond (pond).

Storm Sewers/Culverts

| Map ID | Type | Shape* | Dimensions Diam or H/W | Material** | Inverts Upstream | Inverts Downstream | Condition | Length (ft) |
|---|---|---|---|---|---|---|---|---|
| | SS CU | | | | | | Good Fair Poor | |
| | SS CU | | | | | | Good Fair Poor | |
| | SS CU | | | | | | Good Fair Poor | |
| | SS CU | | | | | | Good Fair Poor | |
| | SS CU | | | | | | Good Fair Poor | |
| | SS CU | | | | | | Good Fair Poor | |
| | SS CU | | | | | | Good Fair Poor | |

* Shape choices. Circular, Arch; Oval; Rectangular; and Irregular.
Also specify if French Drain
** Material choices: Reinforced Concrete Pipe **(RCP)**: Corrugated Metal Pipe (CMP); Asphalt Coated CMP (ACCMP): Corrugated Aluminum Pipe (CAP); ADS plastic pipe (ADS); PVC; Iron; Asbestos Concrete Pips (ACP): Vitrified Clay Pipe (VCP); and Concrete-lined **(CL)**.

Open Channels

| Map ID | Type* | Shape** | Material*** Sides | Material*** Bottom | Width (ft) Bank | Width (ft) Bottom | Elevation (NGVD) US | Elevation (NGVD) DS | Condition | Length (ft) |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | Good Fair Poor | |
| | | | | | | | | | Good Fair Poor | |
| | | | | | | | | | Good Fair Poor | |
| | | | | | | | | | Good Fair Poor | |
| | | | | | | | | | Good Fair Poor | |
| | | | | | | | | | Good Fair Poor | |
| | | | | | | | | | Good Fair Poor | |

* Type choices: natural or man-made.
** Shape choices: Trapezoid: V-shaped: Irregular: or natural.
*** Material choices: Grass; Sand-Cement; Bars Geosynthetic; Brush-lined; Concrete; or other.

Control structures

| Map ID | Type* | Material** | Length | Dimensions Diam or H/W | Skimmer | Grate | Weir type | Condition |
|---|---|---|---|---|---|---|---|---|
| | | | | | Y N | Y N | | Good Fair Poor |
| | | | | | Y N | Y N | | Good Fair Poor |

* Diversion box. Riser/standpipe, Mod inlet inline, Manhole w/weir, Pond outfall, Flashboard, Unknown
** concrete, iron. brick, concrete-lined, *other*
*** circular, rectangular, oval. V. arch. trapezoid, Irregular. nmo

Bridge

| Map ID | Road | Width | Length | Deck | Material* Pier | Wingwall | Pier Number | Pier Spacing | Scour | Condition |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | Y N | Good Fair Poor |

* Concrete, asphalt, brick. imn. other

Pond

| Map ID | Top of bank Elevation | Area | Fence |
|---|---|---|---|
| | | | Y N |



**Figure No. 1b**
**Field data sheet (back)**

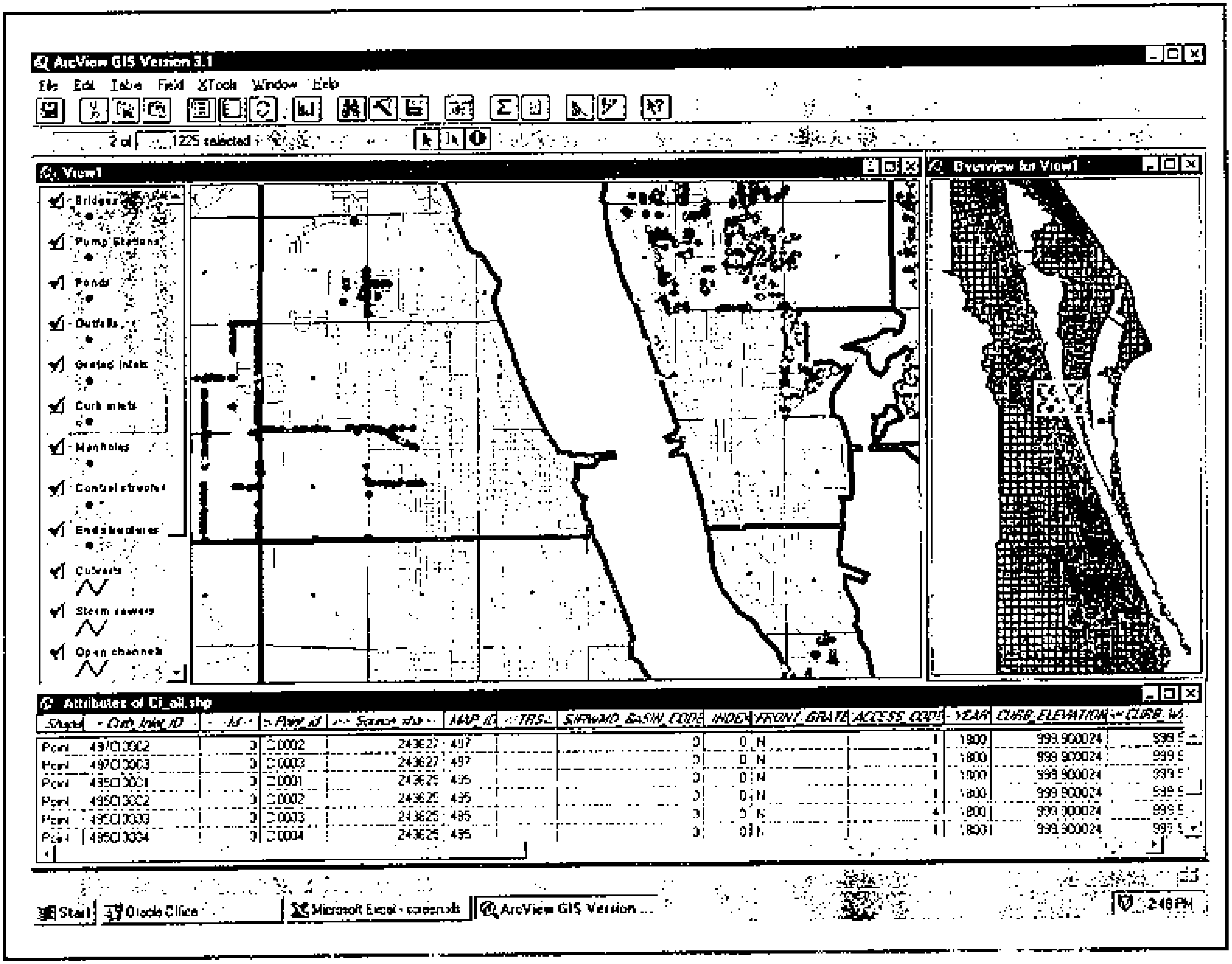


Figure No. 2

Stormwater structure GIS screen example
Mack, Dean, Kurz and McClelland

components of the display are the main view, the overview to the right, and attributes tables at the bottom of the screen (database). The main view can be used to zoom in on areas of interest and select specific structures, The overview window highlights the relative location of the main view on the base map. Information stored in the database for a selected structure can be displayed on the attribute tables shown at the bottom of the figure.

## CONCLUSION

Once a stormwater GIS is defined and populated, a municipality can use the information to define a desired LOS and costs. CDM has defined the LOS criteria for O&M activities shown in **Table 2** to classify existing maintenance on stormwater facilities and for setting O&M goals. These goals can differ between drainage basins and structure types based on the characteristics of each system. In general, the LOS goal for a rural watershed may be less than the **LOS** goal for an urban watershed without having a significant negative impact on flooding or water quality. Achieving desired water quantity and quality goals in an urban system may require LOS **A.**

**Table 2**
**Brevard County Stormwater Needs Assessment**
**Operation and Maintenance Level of Service**

| Level of Service (LOS) | Operation & Maintenance (O&M) |
|---|---|
| A | Routine Inspection and Maintenance |
| B | Routine Inspection with Specific Routine Maintenance |
| C | Routine Inspection with Inspection-Based O&M |
| D | Reaction/Complaint-Based O&M |
| E | No Service |

LOS A designates a system receiving a routine maintenance program of the stormwater facilities based on the typical maintenance schedule for each facility type.

LOS B designates a system with specific facilities receiving routine maintenance and the remaining facilities receiving inspection-based maintenance.

LOS C designates a system receiving routine inspections with maintenance performed based on the results of the inspections.

LOS D designates a system which receives maintenance strictly as a result of complaints.

LOS E designates a system receiving no maintenance.

Using the stormwater inventory GIS, a municipality can assign a LOS to each structure inventoried. Each structure LOS can be assigned a unit maintenance cost based upon **an** assumed inspection and maintenance frequency. The GIS reporting features can then be used to **query** this information to determine an overall planning level program cost for a selected LOS. Assigned LOS criteria can then be adjusted until an affordable and implementable O&M program is developed.

**As** previously discussed, a stormwater inventory GIS can be a useful tool in managing O&M activities and adjusting these activities to meet a desired LOS. The system described in this paper provides Brevard County with desktop access to its PSWMS by showing drainage patterns, structure locations and attribute data, and showing stormwater structure inspection and completed maintenance dates. The system can be used to assist with O&M activity planning, documentation of stormwater structure inspection and maintenance activities necessary for NPDES permit requirements. Also, annual evaluations of the accomplishments of the program can be performed using the GIS (i.e., structure condition versus inspection/maintenance frequency) and ineffective programs can modified. Similar systems are currently being developed for Seminole County and Volusia County to assist with their O&M and NPDES programs.