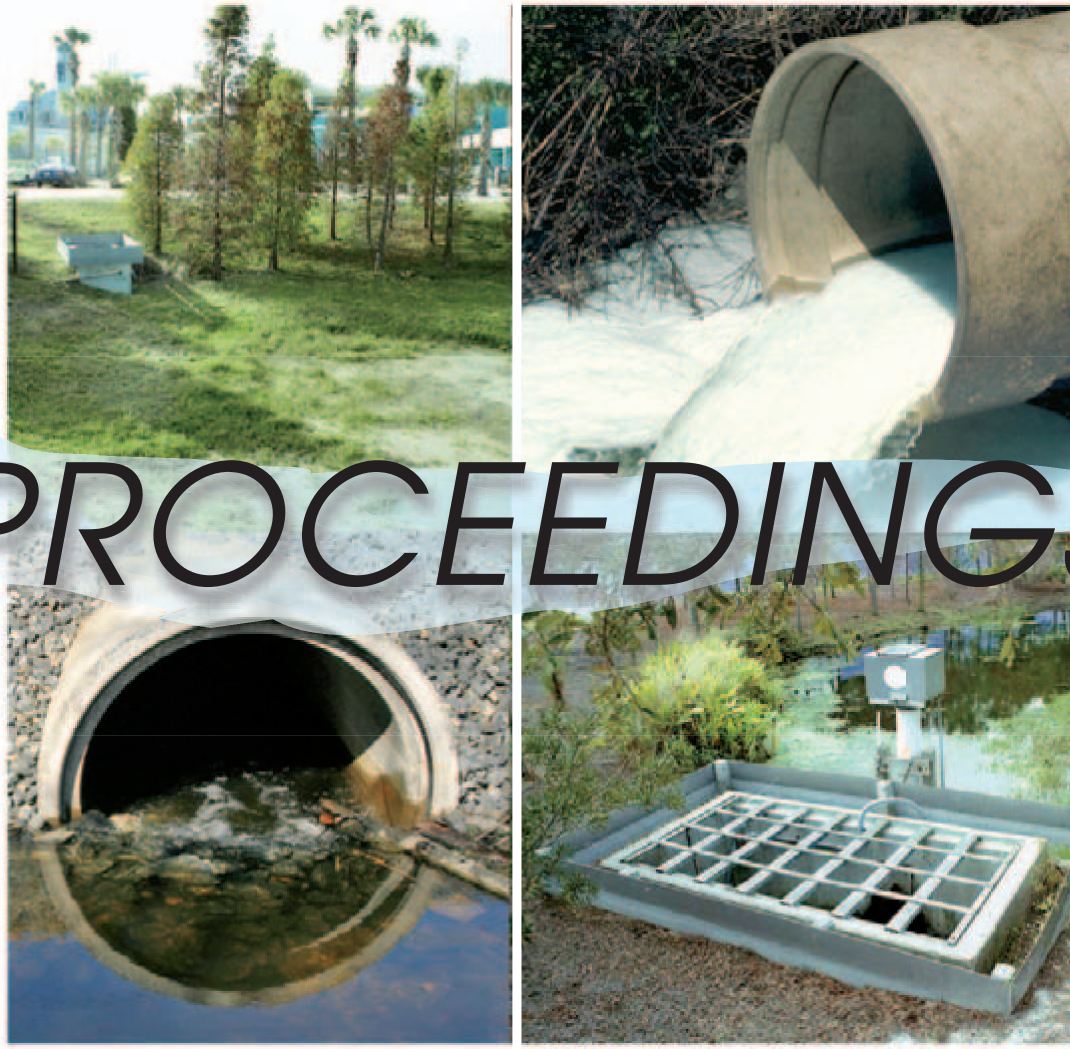
APRIL 27-28, 2005

8TH BIENNIAL CONFERENCE ON

Stormwater Research & Watershed Management

TAMPA MARRIOTT WESTSHORE HOTEL • TAMPA, FLORIDA



PROCEEDINGS

Southwest Florida Water Management District

DEPARTMENT OF ENVIRONMENTAL PROTECTION FLORIDA

Hosted by the Southwest Florida Water Management District and Florida Department of Environmental Protection

PROCEEDINGS

EIGHTH BIENNIAL STORMWATER RESEARCH AND WATERSHED MANAGEMENT CONFERENCE

MARRIOTT TAMPA WESTSHORE, TAMPA, FLORIDA

APRIL 27-28, 2005

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FOREWORD

This conference is the eighth in a continuing series of symposia sponsored by the Southwest Florida Water Management District and the Florida Department of Environmental Protection. It is designed to disseminate the findings of current stormwater research, as well as present the latest developments in watershed management. The conference is organized to provide a forum for the dissemination of a wide range of ideas, where issues can be debated and research results can receive initial peer review. The ultimate goal of the conference is to present the engineers, scientists, and regulators working in the field of stormwater management with the most current ideas and data available so that more efficient and cost-effective treatment of storm runoff can be realized. It is our hope that this conference and these proceedings will not only contribute to improved stormwater management in Florida, but that once the information is available on the world wide web (http://www.swfwmd.state.fl.us/documents/), it will reach a wider audience and help maintain or improve the conditions in our nation's rivers, lakes and estuaries.

This year's conference includes papers emphasizing the present realities of stormwater management on improving receiving waters, the importance of storm volume control in stormwater treatment, the effectiveness of specific stormwater Best Management Practices, the result of pond design on pollution removal, and the use of modeling for watershed management. Twenty-four papers and three poster presentations documenting a wide range of current practices are presented.

Betty Rushton
Eric Livingston

ACKNOWLEDGMENTS

JoAnne Rehor, the Sr. Administrative Assistant for the Environmental Section deserves special recognition, not only for planning and administering the Eighth Biennial Stormwater Research and Watershed Management Conference, but also for her careful attention to detail and for keeping the project on schedule. **Gwen Brown**, Administrative Supervisor for the Resource Management Department and **Josie Guillen** of the Resource Management Department provided additional valuable administrative support. **Pati Twardosky**, Senior Graphic Illustrator, Visual Communications Section, designed the conference logo and provided graphic support. **McRae & Company, Inc.**, helped coordinate many aspects of the conference. **Tammy Smith**, of McRae & Company, Inc., was the District's key contact and was also responsible for compiling and assembling the Proceedings. **Christine Clark, Dwayne Williams and Glenn Lane** of the District's Office Support Section handled the printing of all conference materials, including the Proceedings. **Scott Harbison** of Network Systems coordinated the audio-visual aspects of the conference and kept the presentations running smoothly and on time.

TABLE OF CONTENTS

KEYNOTE

TMDLs AND BMPs – MYTHS & REALITIES
Eric W. Strecker, P.E., Principal, GeoSyntec Consultants, Portland, Oregon....................1

BMPs, IMPERVIOUS COVER, AND BIOLOGICAL INTEGRITY OF SMALL STREAMS
Eric H. Livingston, Florida Department of Environmental Protection, Tallahassee, Florida; Richard R. Horner and Christpoher May, University of Washington, Seattle, Washington; and John Maxted, ARC, Auckland, NZ…………………………………...15

SESSION 1 – BMPs

LOW IMPACT DEVELOPMENT AT BIG BOX STORES
Neil Weinstein, PH.D., P.E., Low Impact Development Center, Rockville, Maryland…39

MANAGING IMPACTS OF DEVELOPMENT ON STORMWATER RUNOFF: BENEFITS OF LOW IMPACT DEVELOPMENT APPROACHES
Moris Cabezas, Ph.D., P.E., PBS&J, Tampa, Florida; and Paul Jensen, Ph.D., P.E., PBS&J, Austin, Texas; and Jack Merriam, Hillsborough County Public Works, Tampa, Florida……………………………………….…………………………......40

CASE STUDIES ON THE PERFORMANCE OF BIORETENTION AREAS IN NORTH CAROLINA
Lucas J. Sharkey and William F. Hunt, Ph.D., P.E., N.C. State University, Raleigh, North Carolina…………………………………………………………….……..44

A MONITORING FIELD STUDY OF PERMEABLE PAVEMENT SITES IN NORTH CAROLINA
E. Z. Bean, EI, Graduate Research Assistant, W.F. Hunt, Ph.D., PE, Assistant Professor and Extension Specialist and D.A. Bidelspach, EI, Extension Associate, Department of Biological & Agricultural Engineering, North Carolina State University, Raleigh, North Carolina………………………………57

SESSION 2 – MODELING

DESCRIPTION OF A CONTINUOUS SIMULATION MODEL (WEANES) FOR ESTIMATING LONG-TERM EFFICIENCIES OF WET RETENTION/DETENTION BASINS
Ashok Pandit, Ph.D., P.E., Professor and Head and Chi Hyueon Youn, Ph.D., Assistant Research Professor Civil Engineering, Florida Institute of Technology, Melbourne, Florida……………………………………………………………………67

HILLSBOROUGH RIVER BASIN ASSESSMENT USING WAM
A. B. Bottcher, and B. M. Jacobson, Soil and Water Engineering Technology, Inc., Gainesville, Florida; J. G. Hiscock, JGH Engineering, Palm Beach Gardens, Florida….77

SESSION 3 – MODELING/WATERSHED MANAGEMENT

VOLUME CONTROL USING INTER-EVENT DRY PERIODS
Marty Wanielista, Ph.D., P.E., Josh Spence, and Ewoud Hulstein, Stormwater Management Academy, University of Central Florida, Orlando, Florida….89

RESTORATION OF LAKE SEMINOLE IN PINELLAS COUNTY FLORIDA: THE SCIENTIFIC PROGRAMMATIC AND PUBLIC POLICY ASPECTS OF URBAN LAKE RESTORATION
Jim Griffin, Southwest Florida Water Management District, Brooksville, Florida; and Jeff Herr, Environmental Research and Design, Orlando, Florida; and Kelli Hammer Levy, Pinellas County Department of Environmental Management……99

SOUTHWEST FLORIDA WATER MANAGEMENT DISTRICT WATERSHED MANAGEMENT PROGRAM – THE DATA PIECE AND TOOLS TO MANAGE
Gordon McClung, P.E., Southwest Florida Water Management District, Brooksville, Florida……………………………………………………………………109

WATER QUALITY FRIENDLY FIELD OPERATIONS YARD
Christopher Bowles, P.E. and Craig W. Dye, Clean Water Services, Hillsboro, OR…110

SESSION 4 – BMP SESSION II

BMP RESEARCH PROJECTS IN SOUTHWEST FLORIDA
Michael L. Lohr, PSM, Johnson Engineering, Inc., Ft. Myers, Florida; and Kim Fikoski, Senior Biologist, Bonita Bay Group, Bonita Springs, Florida…………..118

A STORMWATER RETROFIT INCORPORATING A CDS UNIT
Betty Rushton, Ph.D., Southwest Florida Water Management District, Resource Management Department, Brooksville, Florida……………………………………..…130

CHARACTERIZATION OF STORMWATER SYSTEM SEDIMENTS FOR APPROPRIATE DISPOSAL
Timothy G. Townsend and Pradeep Jain, Department of Environmental Engineering Sciences, University of Florida, Gainesville, Florida…….…………………..………..140

SESSION 5 – WATERSHED MANAGEMENT

URBAN SOIL COMPACTION AND ITS EFFECT ON STORMWATER RUNOFF
Justin H. Gregory, Engineer Intern, Jones Edmunds & Associates, Gainesville, Florida; and Michael D. Dukes, P.E., Assistant Professor, Agricultural and Biological Engineering Department, University of Florida, Gainesville, Florida; and Pierce H. Jones, Professor and Director of the Program for Resource Efficient Communities, University of Florida, Gainesville, Florida; and Grady L. Miller, Associate Professor, Environmental Horticulture Department, University of Florida, Gainesville, Florida……………………………………………………………………..146

ENHANCED CREP & VEGBACC: LANDOWNER INCENTIVE PROGRAMS FOR TEMPERATURE MANAGEMENT
Bruce Cordon and Craig W. Dye, Clean Water Services, Hillsboro, Oregon………….156

QUANTIFYING THE EFFECT OF A VEGETATED LITTORAL ZONE ON WET DETENTION POND PHOSPHORUS LOAD REDUCTION
D. M. Kent, Community Watershed Fund, Rockledge, Florida; and T. Auter, M. Kharbanda, T.A. DeBusk, K. Grace and F.E. Dierberg, DB Environmental, Rockledge, Florida……………………………………………………………………...165

EVALUATION OF A FLOATING WETLAND FOR IMPROVING WATER QUALITY IN AN URBAN LAKE
Thomas A. DeBusk, DB Environmental, Inc., Rockledge, Florida; and Rick Baird, Orange County Environmental Protection Division, Orlando, Florida; and David Haselow and Tom Goffinet, Azurea, Inc., Rockledge, Florida…………………175

SESSION 6 – BMP SESSION III

EFFECTS OF RESIDENCE TIME AND DEPTH ON WET DETENTION SYSTEM PERFORMANCE
Harvey H. Harper, Ph.D., P.E., Environmental Research & Design, Inc., Orlando, Florida………………………………………………………………….......185

FINDING SPACE FOR STORMWATER IN THE URBAN ENVIRONMENT
Michele C. Adams, Wesley Horner, Andrew Potts, Cahill Associates, West Chester, Pennsylvania……………………………………………………….........197

AN EVALUATION OF AN EFFLUENT FILTRATION STORMWATER POND SYSTEM
Betty Rushton, Ph.D., and Kara Teague, Environmental Section, Stormwater Program, Southwest Florida Water Management District, Brooksville, Florida………………....206

SESSION 7 – BMP SESSION III (CONT.)

CONVERSION OF AN URBAN POND TO A WATER QUALITY TREATMENT POND
Jonathan T. Smith, P.E., CPSWQ, Extension Engineer and William F. Hunt, Ph.D., P.E., Assistant Professor and Extension Specialist, Department of Biological and Agricultural Engineering, North Carolina State University, Raleigh, North Carolina; and Steve Jadlocki, Water Quality Program Administrator, Engineering and Property Management, City of Charlotte, Charlotte, North Carolina……………………………………………………………………….216

THE DYNAMICS OF EFFECTIVE STORMWATER MANAGEMENT
Candie Pedersen, BCI Engineers & Scientists, Inc., Lakeland, Florida; and Jim Price, Environmental Consultant, Lakeland, Florida……………………………………….224

ADDITIONAL PAPERS AND POSTER PRESENTATIONS

ARC HYDRO ENHANCED DATABASE FOR WATER RESOURCES MANAGEMENT
Jack Hampson, CMS, Program Manager, PBS&J, Tampa, Florida……………………229

ASSESSMENT OF GAGE-ADJUSTED RADAR RAINFALL DATA FOR USE IN A WATERSHED MANAGEMENT PROGRAM AT THE SOUTH FLORIDA WATER MANAGEMENT DISTRICT
D. Michael Parrish, BEM Systems, Inc., West Palm Beach, Florida…………………..239

STORMWATER POND TREATMENT EFFICIENCY EVALUATION
Chris Zajac and Randy Smith, Southwest Florida Water Management District, Brooksville, Florida………………………………………………………..…………………248

TMDLs and BMPs – Myths & Realities

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Introduction

Much has recently been learned about the performance of BMPs and yet this information is only slowly making its way into improving how we manage stormwater as well as how we implement regulatory programs, such as TMDLs and local design standards. Recently, there has been a growing trend of providing "more sustainable" and "lower-impact" approaches to development that is encouraging in its ability to improve stormwater quality as well as protect downstream habitat. This paper and talk were developed to challenge some of the traditional thinking about stormwater management and provide some recommendations and guidance to practitioners. The author encourages everyone to critically think through the concepts presented in this paper.

Best Management Practices – What Have We Learned About Their Performance

The US EPA (Environmental Protection Agency)/ASCE (American Society of Civil Engineers) National Stormwater BMP (Best Management Practice) Database has been under development since 1994 under a US EPA grant project with the Urban Water Resources Research Council (UWRRC) of ASCE (Urbonas, 1994). The project was initiated due to the:

- Recognized inconsistent data collection and reporting methods that limit scientific comparison/evaluation of studies,
- Resulting wide range of reported "effectiveness" (e.g. – to + percent removals), and
- Widespread use of BMPs and faulty BMP performance information without sufficient understanding of performance and factors leading to performance

The project has included the development of recommended protocols for BMP performance (Urbonas, 1994 and Strecker 1994), a compilation of existing BMP information and "loading" of suitable data into a specially designed database (www.bmpdatabase.org), and an initial assessment of the results of the analyses of the database (Strecker et. al., 2001). A detailed guidance document on BMP monitoring has been developed, titled: Urban Stormwater BMP Performance Monitoring: A Guidance Manual for Meeting the National Stormwater BMP Database Requirements (download at: www.bmpdatabase.org). This paper includes a summary of more recent analyses.

Municipal separate storm sewer system owners and operators, industries, and transportation agencies need to identify effective BMPs for improving stormwater runoff water quality that directly target their "pollutants of concern," especially given the increasing inclusion of TMDLs into Stormwater permits. The protocols developed under this project and the Urban Stormwater BMP Performance Monitoring guidance addresses the need for improved information by helping to establish a standard basis for collecting water quality, flow, and precipitation data as part of a BMP monitoring program. The collection, storage, and analysis of this data will ultimately improve BMP selection and design.

One of the major findings of the EPA/ASCE BMP Database efforts to date has been that BMP pollutant removal performance for most pollutants is believed best assessed by (Strecker et. al., 2001):

- How much stormwater runoff is prevented? (via evapotranspiration and/or infiltration; *Hydrological Source Control*)
- How much of the runoff that occurs is treated by the BMP or not? (amount of flow by-passed or exceeding BMP effective treatment rates; *Amount of Runoff Treated*)
- Of the runoff treated, what is the effluent quality? (Statistical characterization of effluent quality; *Quality of Treated Runoff).*

For some pollutants, the amount of material captured may also be important (e.g., for TMDL compliance), as well as how the BMP mitigates temperature and/or flow changes. The most common performance measure used today is percent removal of pollutants. The database team has determined that percent removal is a highly problematic method for assessing performance and has resulted in some significant errors in BMP performance reporting (Strecker, et. al., 2001). Percent removals are not recommended as performance descriptors for stormwater BMPs as they can result in significant errors and mistaken BMP performance characterizations.

An Updated Re-Evaluation of the National BMP Database

The project team has completed an assessment of the recently expanded database. Table 1 presents an overview of the structural BMPs currently in the database, including the number of data records for each structural BMP type. These are studies that meet the protocols established for BMP monitoring and reporting. The almost 200 studies now in the database compares with the total of just over 60 BMP studies in the database during the initial evaluation. New BMP information is being provided to the database team at about a rate of 15 to 30 studies per year. There are currently about 50 studies awaiting entry into the database that are now being entered with renewed funding.

Each study has again been analyzed in a consistent manner as described in Strecker, et. al. (2001) and on the project web site. The data being produced includes lognormal distribution based summary statistics, comparisons of influent and effluent water quality through parametric and non-parametric hypothesis tests, and a large number of other summary statistics. The project team has been investigating the effects of BMPs on hydrology and effluent quality.

Hydrology Evaluation

One of the goals of the database was to provide better information on the effects of BMPs on hydrology and whether some BMPs may have some benefits over others in terms of reducing volume of runoff (Hydrological Source Control-HSC). For example, one would expect that a wet pond might not significantly decrease the volume of runoff, but a biofilter might, given the contact with drier soils and resulting evapotranspiration and/or infiltration. Much of the premise of Low Impact Development (LID) is based upon reducing runoff volumes. Accurately measuring flow during storm conditions is very difficult (EPA, 2002). In a field test of over 20 different flow measurement technologies and approaches, FHWA (2001) found that flow measurements of volume of runoff over a storm can be upwards of 50 percent or more off of the expected true flow. Therefore any assessments of the database will likely show some variability in flow changes. However, some trends are evident in that BMPs with soil soaking and drying are showing a decrease in runoff volumes likely due to a combination of evapotranspriation and deeper infiltration.

Table 1. Structural BMPs in the International BMP Database

BMP TOTALS BY CATEGORY

BMP CATEGORY	NUMBER OF BMPS
Structural	
Biofilter (Grass Swales)	32
Detention Basin	24
Hydrodynamic Device	16
Media Filter	30
Percolation Trench/Well	1
Porous Pavement	5
Retention Pond	33
Wetland Basin	15
Wetland Channel	14
Total	**170**
Non-Structural	
Maintenance Practice	28
Total	**28**
Grand Total	**198**

BMP TOTALS BY STATE/COUNTRY

STATE	NUMBER OF BMPS
Domestic	
AL	13
CA	**41**
CO	4
FL	24
GA	2
IL	5
MD	4
MI	5
MN	7
NC	6
NJ	3
OH	1
OR	3
TX	19
VA	29
WA	20
WI	10
International	
Sweden	1
Canada	1

Figure 1 presents plots of inflow vs. outflow for Biofilters (Swales and filter strips), Detention Basins (dry ponds), Retention Ponds (wet ponds) and Wetland Basins. Hydrodymic devise and filters were not included as they do not reduce runoff volumes. Biofilters showed an average of about 40 percent less and dry-extended detention systems about30 percent less volume of outflows as compared to inflows. The other BMPs showed a large scatter, but generally showed an increase in runoff volumes.

Table 2, presents the results of removing the smaller more insignificant storms from the analyses (storms less that 0.2 watershed inches removed). From these analyses, it is apparent that detention basins (dry-ponds) and biofilters (vegetated swales, overland flow, etc.) appear to contribute significantly to volume reductions, even though they were likely not specifically designed to do so. Based upon the recommended criteria above for assessing BMP performance, it appears that there is a basis for factoring in volume and resulting pollutant load reductions into BMP performance. This has significant implications for Total Maximum Daily Loads (TMDLs) implementation planning and other stormwater management planning. It is also expected that as BMPs that are specifically designed to reduce runoff volumes (e.g., lower impact development, etc.) are tested and information added into the database, that these results will improve.

Water Quality Performance

The analysis of water quality performance data of the BMPs that we conducted is comprised of three levels:

1) a comprehensive evaluation of effluent vs. influent water quality for each BMP study;

2) comparisons of effluent quality amongst BMP types; and

3) comparisons of performance vs. design attributes for BMP types and individual BMPs.

Even with the increase in data in the database since the last evaluation, the total number of BMPs in any one category is still relatively small as compared to the number of design parameters and other regional factors that can be potentially investigated (Table 1).

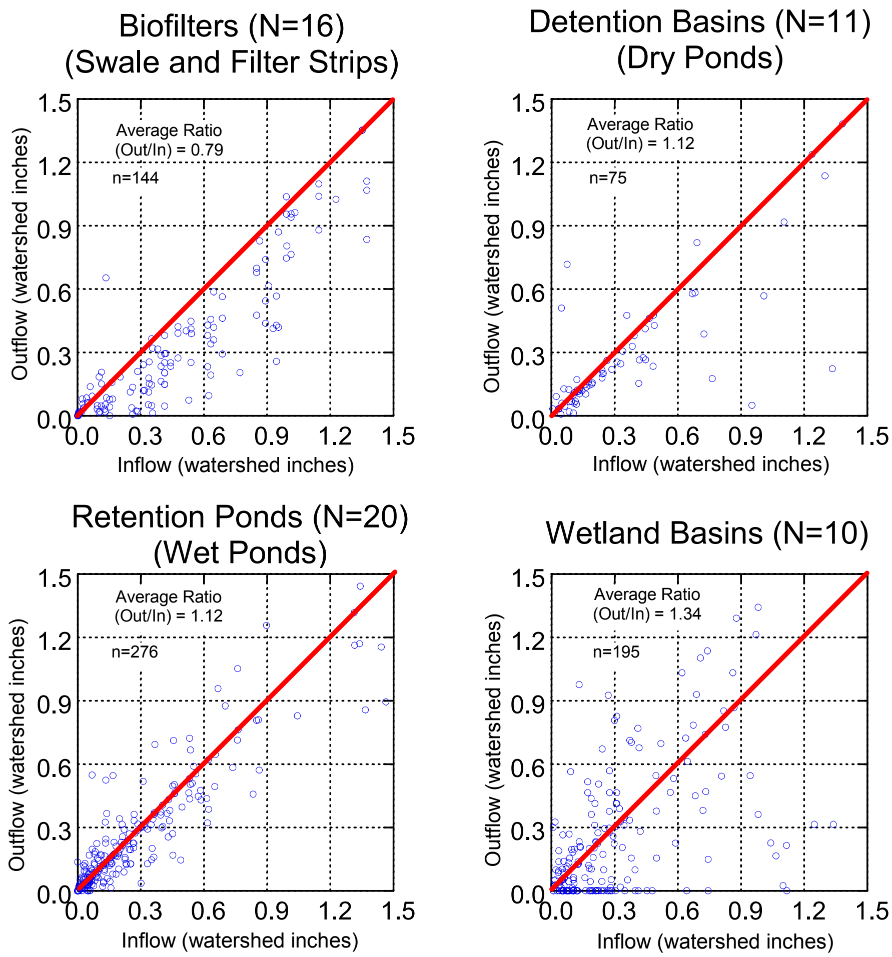

Figure 1. Comparison of Individual Storm Inflow and Outflow Volumes for Indicated BMPs (N= number of BMPs included; n= number of storm events)

<u>Effluent Quality.</u> Effluent quality is much less variable than fraction removed (or percent removed) for BMP studies as shown in Figure 2, which shows box plots by BMP types of the fractions of total suspended solids (TSS) removed and a box plots of TSS effluent quality. The box plots present the median, the upper and lower 95 percent confidence intervals of the median, and the 25^{th} and 75^{th} percentiles.

As has been found previously (Strecker et. al., 2001), it appears that percent removal is more or less a function of how "dirty" the inflow is. What is new from the analyses of the expanded database is that effluent quality can now be assumed to be different amongst different BMP types for some parameters. It appears that Retention Ponds (wet ponds) and Wetlands can achieve lower concentrations of TSS (and other parameters) than other BMPs, while hydrodynamic devices were the lowest performers (higher effluent concentrations) on average for TSS. As a comparison, the 95% confidence interval for the median wet pond removal is between about 50 and 90 percent (a little better than 0 to 100), while the median effluent quality 95% confidence range is between about 11 to 18 mg/l.

Table 2. Ratio of Mean Monitored Storm Event Outflow to Inflow for inflow Storms Greater than 0.2 watershed inches.

BMP Type	Mean Monitored Outflow/Mean Monitored Inflow for Events Greater Than or Equal to 0.2 Watershed Inches
Detention Ponds	0.70
Biofilters	0.62
Media Filters	1.0
Hydrodynamic Devices	1.0
Wetland Basins	0.95
Retention Ponds (wet)	0.93
Wetland Channels	1.0

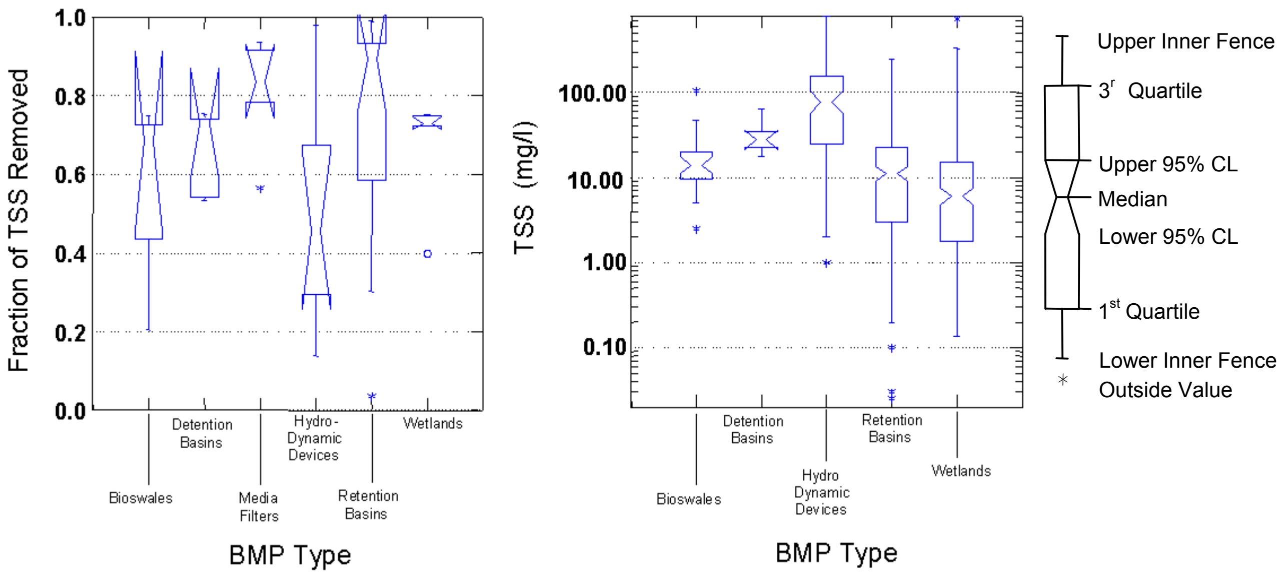

Figure 2. Box plots of the fractions of total suspended solids (TSS) removed and of effluent quality of selected BMP types, by BMP Study.

Figure 3 shows the influent and effluent box total and dissolved copper box plots for event data (each event considered separately). For all BMP types, total copper influent and effluent can be assumed to be different for all BMP Types. However, for dissolved Copper concentrations only bioswales and wet ponds appear to have effected concentrations. Note that incoming dissolved concentrations are quite low and therefore this effects “efficiency.”

Figure 4, shows the effluent quality results for comparing total and dissolved zinc and phosphorus for the same BMP categories weighted by BMP study (each BMP Study is a single data point). For dissolved constituents, data is still somewhat sparse. In these plots, the effluent quality of hydrodynamic devices is somewhat more consistent with other BMP types; this may be a confirmation of the work by Sansalone et. al. (1998)

which showed that a sizeable proportion of some pollutants are associated with fractions that may be removable via limited detention time devices. Some of his current work is demonstrating this in more detail (Sansalone, 2004). It is interesting to note that the lowest effluent quality achieved for phosphorus is about 50 to 60 ug/l. This contrasts with TMDLs or other water quality programs where the ultimate phosphorus goal has been set to 10 to 20 ug/l and then showing achievement of such goals by misapplication of percent removal approaches. Some programs have "allowed" implementers to daisy chain BMPs based upon percent removals. This approach is not supported by this data. However, an effluent quality of 50 to 60 ug/l is a significant reduction as compared to typical urban runoff concentrations.

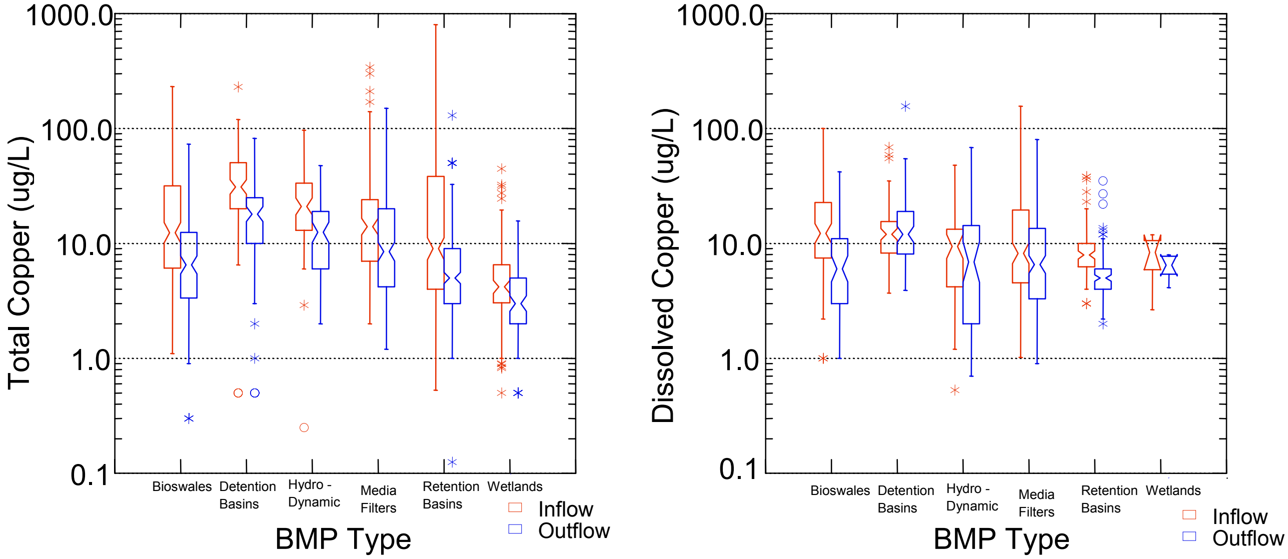

Figure 3. Box plots of influent and effluent quality of selected BMP types for total and dissolved Copper by event.

Human pathogens are increasingly of concern in stormwater discharges. There is still much debate over the usefulness of the fecal coliform test (or other bacteria tests) as an indicator of human pathogen levels in urban stormwater. Figure 5 shows a comparison of influent and effluent fecal coliform box plots for the indicated BMP types and a more detailed look at wet ponds. It should be noted that this is grab sample data. From the plot, it is apparent that some BMPs appear to be able to reduce fecal coliform concentrations, including media filters and retention ponds, while others are not. The second plot for retention ponds demonstrates the influent and effluent quality observed for wet ponds. It should be noted that in cases where there is heavy wildlife use, increases have been found.

Some of the other assessments that are being performed are the potential reductions in toxicity of heavy metals by BMPs. More recent BMP studies have been collecting data on water hardness and therefore there is an ability to assess potential toxicity issues via comparisons of effluent quality with EPA acute and chronic criteria values (as benchmarks as the criteria apply in receiving waters). One trend that we have noticed in the data is that for many BMPs, hardness levels are increased in effluent vs. the influent and therefore this could contribute along with concentration reductions to reduce toxicity (as defined by EPA's Acute Criteria for Aquatic Life). We will also be looking at the effects of BMPs on load reductions considering both hydrological source control performance as well as effluent quality.

<u>Design vs. Performance.</u> During the initial evaluation no statistically relationships between design parameters and performance were found (Strecker, et. al., 2001). This included retention ponds and wetlands and their treatment volume relative to measured storm events. Figure 6 shows a scatter plot of Retention Ponds (wet ponds with a permanent pool) effluent quality vs. the ratio of the treatment volume to mean monitored storm event volume, and a box plot of Retention Pond mean effluent quality for sites with ratio less than one and greater than one ratio of the treatment volume to mean monitored storm event volume. The plots clearly demonstrate that at those sites where the wet pool treatment volume was greater than the average size storm event inflows monitored, the effluent quality was significantly lower. In addition, the variability of effluent quality for the larger retention ponds was lower. These results are expected, but it is one of the first times that they have been demonstrated statistically.

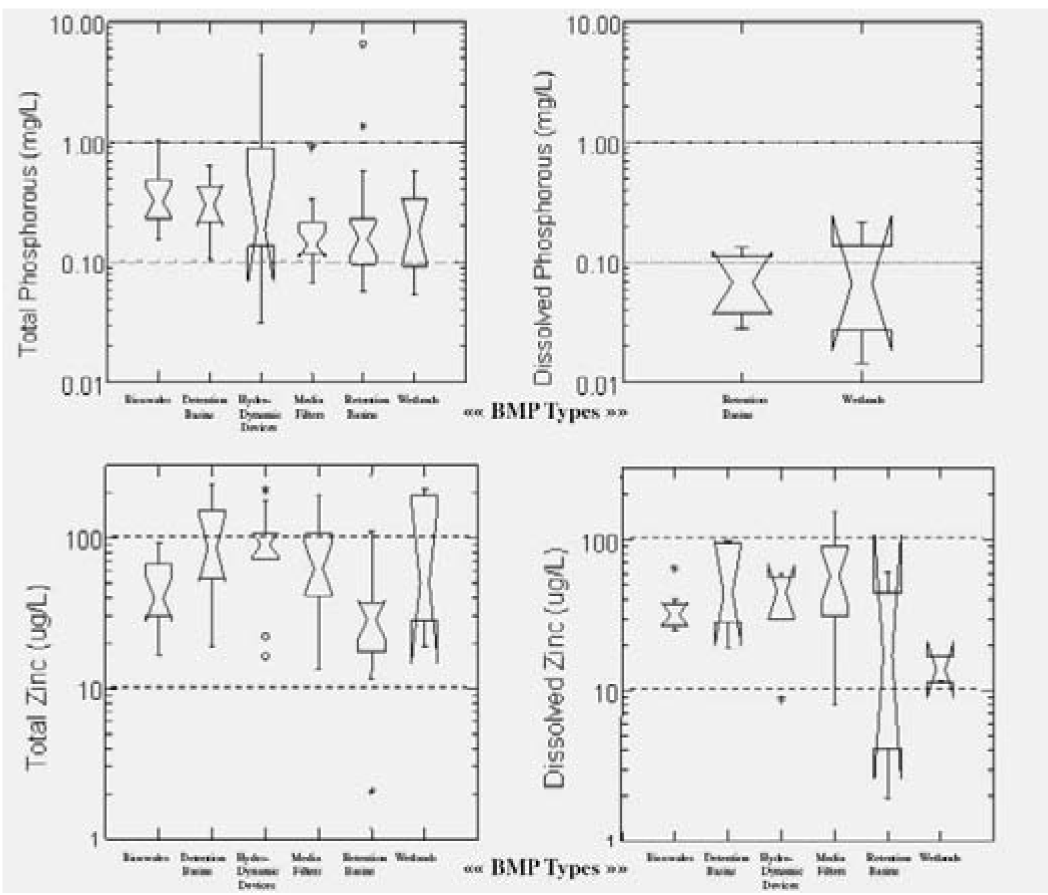

Figure 4. Box plots of effluent quality of selected BMP types for total and dissolved phosphorus and zinc, by BMP type.

Figure 7 shows effluent comparisons for the same ratio for total phosphorus and total zinc. Note that for phosphorus, for the sites with a ratio less than 1, one cannot conclude that the BMP had an effect. For sites that are of the average size inflow, performance is better. It should be noted that this ratio is based upon the average size inflow volume and not the average sized rain event. One should not use the average size event at a rain gage as a basis for asserting BMP sizing and this average rain event would include many events that did not produce runoff or very little runoff.

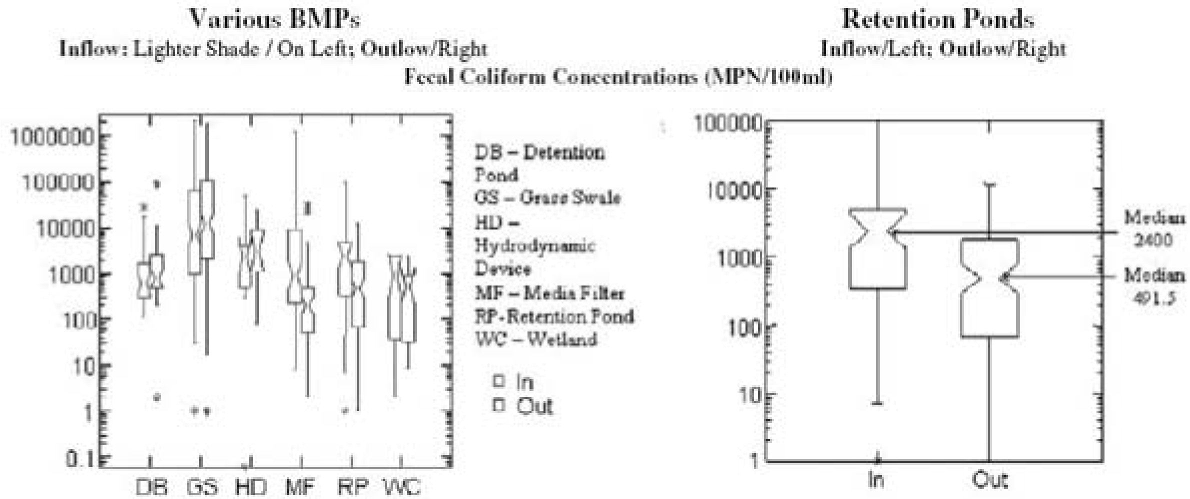

Figure 5. Box plots of effluent quality of selected BMP types for Fecal Coliform and Fecal Coliform inflow and outflow highlighted by event.

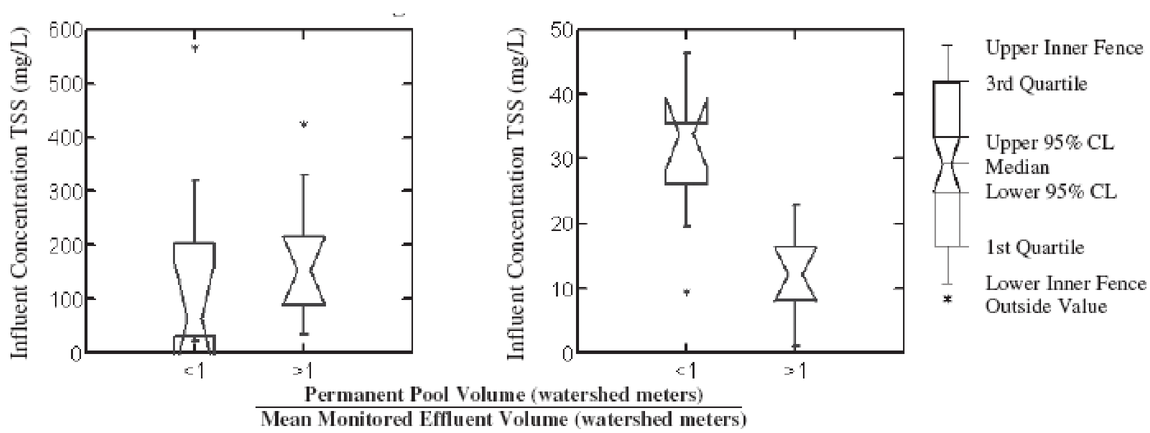

Figure 6. Scatter plot of 1) Retention Pond (with permanent wet pool) TSS effluent quality vs. the ratio of the permanent pool volume to mean monitored effluent volume and 2) Box plots of the TSS effluent quality of sites grouped by a ratio of less than or greater than 1 for the ratio of the permanent pool volume to mean monitored effluent volume by BMP study.

Implications for Setting of BMP Design Requirements and TMDLs

The analysis of water quantity and water quality performance of BMPs is very useful in consideration of setting of stormwater design standards and development of TMDL implementation plans. Some recommendations include:

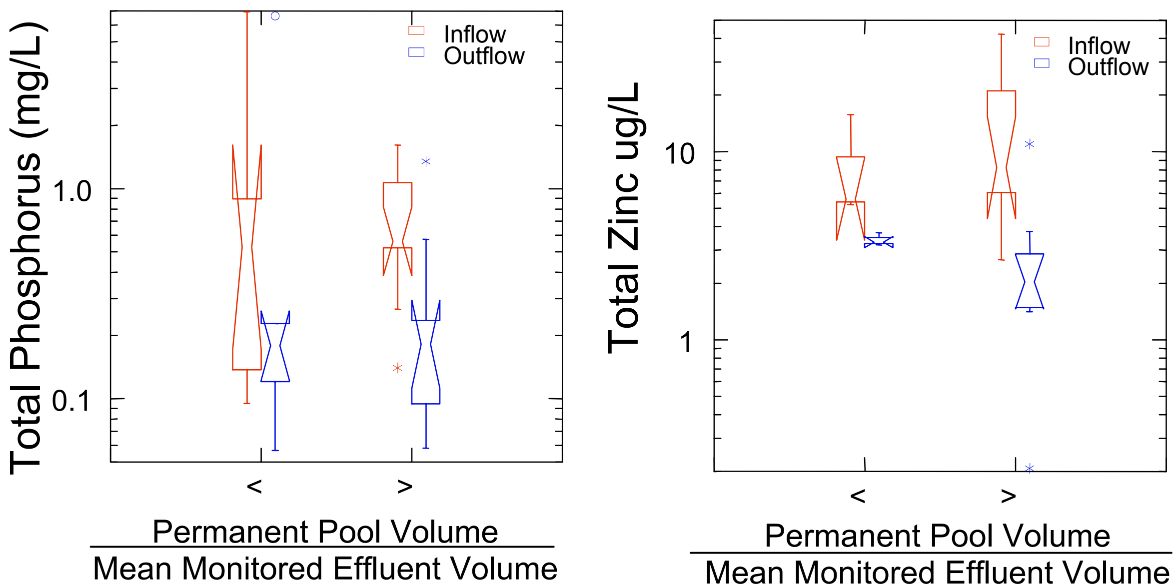

Figure 7. Box plots of the total phosphorus and total zinc effluent quality of sites grouped by a ratio of less than or greater than 1 for the ratio of the permanent pool volume to mean monitored effluent volume by BMP study.

- That design standards should account for the hydrologic losses (HSC) that can occur with some BMP types to encourage their use. Both biofiltration systems and dry extended detention ponds appear to show significant reductions in runoff that is routed through them.

- Continuous simulation techniques should be employed to assess potential BMP design sizing vs. "percent capture" to ascertain what the potential hydraulic performance of BMPs will be over long-time periods. Given the expenditures of resources by the private and public sector on BMPs, it is imperative that those setting standards should conduct these more detailed assessments with more local rain gages to assess the hydrologic and hydraulic performance of BMPs. Using a 24-hour rainfall analysis to set standards is problematic and often results in under-sizing of BMPs.

- BMPs should be targeted based upon expected performance of BMPs to "Pollutants-of-concern". For example, if TSS and dissolved copper are the constituents of concern, than a hydrodynamic device alone, will likely not address the issues. Several efforts are under way to develop unit processes descriptions of BMP performance. These efforts together with BMP performance information should be used to evaluate the potential results of employment of various BMP types. It is likely that given a wide mixture of pollutants of concern, that more "treatment train" approaches will be needed.

- BMP "Acceptance" is becoming a larger issue for communities. Are all "BMPs" acceptable regardless of performance? One of the problems that BMP Vendors face is regulatory requirements that appear to state that one selected treatment BMP for any area must "do it all." When in fact, in most cases a treatment train is acceptable and in fact more desirable from a water quality perspective. Vendors to stay in business have to make claims to be all encompassing. Developing acceptance standards that are defensible as well as result in well performing BMPs will become an increasing goal of BMP requirement programs. An example of the problems of BMP acceptance is presented in Figure 8. By almost all BMP acceptance criteria, this BMP would be accepted for its greater than 80 percent removal. One has to consider though whether an average effluent quality of over 100 mg/l is acceptable. Compared to other BMPs effluent quality, it is not. That is not to say that this BMP type might not serve a valuable role as initial treatment to a stormwater wetlands.

Table 3. George Field Study Evaluation of a Vortechs model 11000

Runoff	TSSin (mg/L)		TSSout (mg/L)		% Reduction	
Event #	Interpolated	Arithmetic	Interpolated	Arithmetic	Interpolated	Arithmetic
1	987.48	693.52	263.18	205.98	73%	70%
2	128.73	88.57	59.23	59.18	54%	33%
3	1040.04	882.42	337.87	486.75	68%	45%
4	213.73	225.42	359.14	388.08	-68%	-72%
5	1673.57	1217.53	71.39	102.84	96%	92%
6	535.16	603.54	70.14	85.23	87%	86%
7	180.81	132.22	29.76	34.88	84%	74%
8	2491.55	2202.78	35.41	35.47	99%	98%
9	89.99	76.60	31.98	33.14	64%	57%
10	1047.02	2257.46	37.08	31.22	96%	99%
11	439.45	344.86	16.57	13.83	96%	96%
12	445.19	291.58	17.36	14.91	96%	95%
13	1156.16	674.94	44.72	37.91	96%	94%
Averages	802.2215	745.4954	105.6792	117.6477	87%	84%

(Winkler and Guswa 2002)

Conclusions

An evolving tool is available to practitioners who are assessing the performance of BMPs via the International Stormwater Best Management Practices Database Project. Practitioners can perform their own evaluations via downloading of information from the web site.

Results of the analyses of the now expanded database have reinforced the initial findings that BMPs are best described via their ability to reduce runoff volumes, how much of the runoff record is treated (and not), and of that treated, what does the effluent quality and characteristics (potential toxicity) look like. The results are showing that the effluent quality of various BMP types can be statistically characterized as being different from each other. BMPs design factors, including sizing are becoming more statistically discernable in the BMP type data sets with larger number of studies. Continued population of the BMP database with additional studies will improve the ability to discern performance vs. BMP selection and design. The BMP database provides a useful tool to develop more accurate design requirements for stormwater BMPs as well as implementation plans for TMDLs that will be more targeted at achieving desired outcomes. These Basic BMP performance description elements can be utilized to:

- assess the concentrations that BMPs are able to achieve (concentration TMDLs),
- more accurately assess effects on total loadings (TMDLs) (how much runoff is prevented, treated and more realistic estimates of resulting loads)
- frequency of potential exceedances of water quality criteria or other targets, and
- other desired water quality performance measures.
-

For now designers are urged to utilize a treatment train approach for BMPs wherever possible that considers the pollutants of concern and their form, the unit processes that are needed to remove those pollutants, and the unit processes that occur in significance in various BMP types. For example as Figure 8 shows, if one is interested in removing multiple pollutant types, then a treatment train has many advantages. Using a treatment train will help to account for the inherent variability and uncertainties that are associated with BMP performance. Designers should employ conservative criteria, including sizing and focusing on longer residence times for volume based BMPs as well as larger sizing of filters and other flow-through BMPs (see ASCE/WEF 1998 Water Quality Manual of Practice).

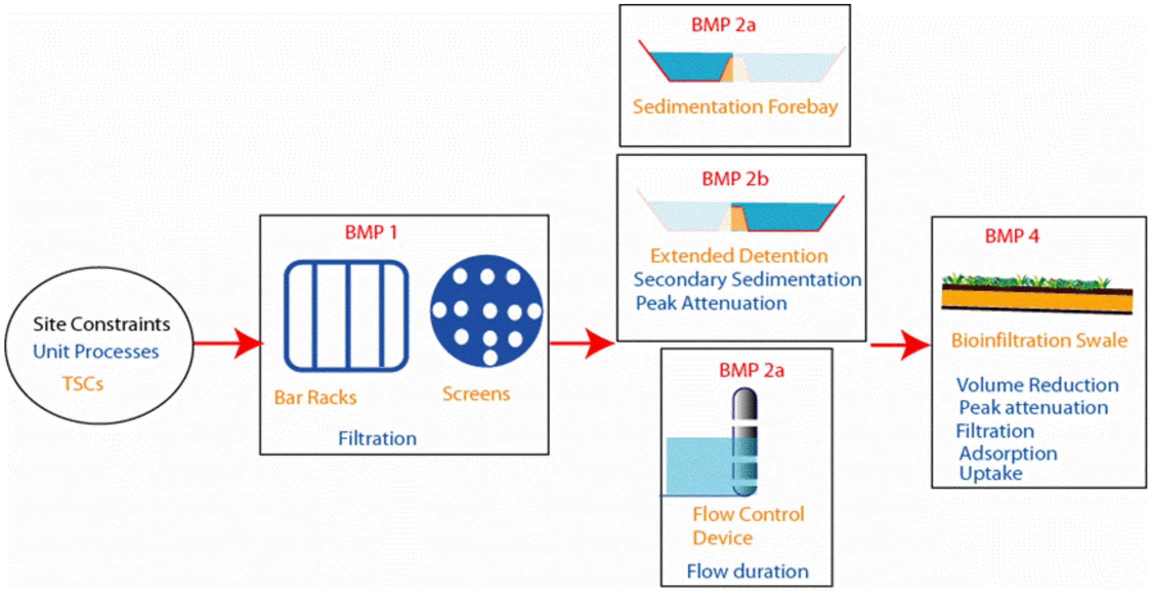

Figure 8. A Treatment Train Designed to Remove Trash/Debris, TSS and Dissolved Copper

Finally, it is important that we also attempt to minimize the increase in runoff. Typical urban development has severely reduced the evapotranspiration (ET) and infiltration. Too often, we think infiltration could be the answer in areas where pre-development infiltration was minimal, but is eliminated due to soils and/or slope conditions concerns. We need to look at ways of mimicking pre-development evapotranspiration rates as the first step in stormwater management. It is often the case that pre-development evapotranspiration may be as high as 80+ percent of rainfall. If we infiltrate all of that water, then we will have increased infiltration greatly over pre-development. To increase ET, the "sponge" should be restored which includes:

- Trees, Shrubs and Grasses
- Shallow soils (non compacted)
- EcoRoofs

Stormwater Management is a difficult task, but we need to keep applying new knowledge that is carefully evaluated for ones own situation.

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BMPs, Impervious Cover, and Biological Integrity of Small Streams

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Abstract

Stream ecosystems in three different locations in the United States were found to benefit in a similar fashion from retention of watershed forest and wetland cover and wide, continuous riparian buffers with mature, native vegetation. The findings can help guide comprehensive watershed management and application of these non-structural practices in low-impact urban design. Intensive study of structural best management practices (BMPs) in one location found that, even with a relatively high level of attention, a minority of the developed area is served by these BMPs. Those BMPs installed are capable of mitigating an even smaller share of urban impacts, primarily because of inadequacies in design standards. Even with these shortcomings, though, results showed that structural BMPs help to sustain aquatic biological communities, especially at moderately high urbanization levels, where space limits non-structural options.

Introduction

By the mid-point of the 1990s the effects of watershed urbanization on streams were well documented. They include extensive changes in basin hydrologic regime, channel morphology, and physicochemical water quality associated with modified rainfall-runoff patterns and anthropogenic sources of water pollutants. The cumulative effects of these alterations produce an in-stream habitat considerably different from that in which native fauna evolved. In addition, development pressure has a negative impact on riparian forests and wetlands, which are intimately involved in stream ecosystem functioning. Much evidence of these effects exists from studies of urban streams around the United States (e.g., Klein 1979; Richey 1982; Pedersen and Perkins 1986; Scott, Steward, and Stober 1986; Garie and McIntosh 1986; Booth 1990, 1991; Limburg and Schmidt 1990; Booth and Reinelt 1993; Weaver and Garmen 1994).

What was missing at that point in time, though, was definition of the linkages tying together landscapes and aquatic habitats and their inhabitants strong enough to support management decision-making that avoids or minimizes resource losses. Lacking this systematic picture, urban watershed and stormwater management efforts have not been broadly successful in fulfilling the federal Clean Water Act's stipulation to protect the biological integrity of the nation's waters. Effective management needs:

- well-conceived goals of what biological organisms and communities are to be sustained and at what levels;
- understanding the foundation for judging what habitat conditions they need for sustenance; and, in turn

- understanding the watershed attributes consistent and inconsistent with these habitat conditions.

To date, drainage programs have focused on amelioration of stormwater peak flow rate increases following development to reduce stream erosive shear stress and its damage to stream habitats. Since 1980, a few stormwater management programs have required treatment by passive structural BMPs such as infiltration systems including swales and basins or wet detention ponds. Numerous projects around the country have monitored the effectiveness of these BMPs in reducing stormwater pollutant loadings or concentrations. However, there has been little tie between these prescriptions and ecological considerations, or even how well they work to sustain biological communities that they ostensibly exist to protect.

What little study had been done was far too limited to draw firm conclusions but was not promising. Maxted and Shaver (1997) were not able to distinguish a biological advantage associated with the presence of structural BMPs serving eight Delaware stream reaches versus their absence in 33 cases. Jones, Via-Norton, and Morgan (1997) studied biological and habitat response in streams receiving discharges from several types of water quality and quantity control BMPs relative to reference locations. They concluded that appropriately sited and designed BMPs provided some mitigation of stormwater impacts, but that the resulting communities were still greatly altered from those in undeveloped watersheds.

Project Background and Methods

With this background of insufficient understanding of relationships among watershed and aquatic ecosystem elements, and the capabilities of prevailing management strategies to influence these relationships, the U.S. Environmental Protection Agency (USEPA) commissioned the Watershed Management Institute (WMI) to investigate "The Ecological Effects on Small Streams of Stormwater and Stormwater Controls." This project would study stream habitats and biology across gradients of urbanization and BMP application in four regions of the nation (Austin, TX; Montgomery County, MD; Puget Sound, WA; and Vail, CO). The hypothesis being tested was that the implementation of structural and nonstructural BMPs to reduce pollutant loadings and peak discharge rates would allow higher levels of biological integrity to occur at higher levels of watershed imperviousness.

Phase 1 of the project was conducted in Montgomery County and Austin. Phase 2 of the project was conducted in Vail, where the focus was primarily on nonstructural BMPs, and in the Puget Sound region. Phase 3 of the project led to collecting additional watershed characteristic information in Montgomery County, Austin, and Vail, especially on stream riparian zones, wetland retention, and forest retention. Phase 3 also focused on two Puget Sound subwatersheds where detailed information on stormwater BMPs was collected. Finally, Phase 4 of the project built on some of the insights gained from the earlier phases to refine the relationships between structural and nonstructural BMP implementation and stream ecosystem health.

This study followed an earlier effort along similar lines in the Puget Sound region funded by the Washington Department of Ecology Centennial Clean Water Fund. Together these studies built a database now totaling more than 650 reaches on low-order streams in watersheds ranging from no urbanization and relatively little human influence (the reference state, representing "best attainable" conditions) to highly urban (>60 percent total impervious area, TIA).

Biological health was assessed according to the benthic index of biotic integrity (B-IBI; Fore, Karr, and Wisseman 1996) and, in Puget Sound, the ratio of young-of-the-year coho salmon (a relatively stress-intolerant fish) to cutthroat trout (a more stress-tolerant species). A description of sampling sites, methods, and preliminary results from the initial Puget Sound research and Phases 1 and 2 of this project have been previously reported, including at the two previous Biennial Research Conferences (Livingston, et. al, 1999, 2002).

Phases One and Two: Influence of Structural BMPs Results and Discussion

Maxted (1999) gave a preliminary report on the overall results available at that time of Phases one and two of the WMI study. Differences in expressions of macroinvertebrate community integrity appropriate for the various locations were reconciled by scoring each relative to the best attainable measure for the region. The patterns of association between these biological expressions and TIA were similar for the Maryland, Texas, and Washington sites, and also similar to the Delaware watersheds studied earlier (Maxted and Shaver 1997). Namely, none exhibited a threshold level of urbanization where biological decline began. Additionally, as the Delaware results had indicated, stream reaches studied by WMI with and without structural BMPs could not be distinguished in biological quality (Figure 1). This preliminary analysis pointed out two instances of general unity among differing ecoregions in landscape-aquatic ecosystem relationships – the importance of both riparian and forest retention within a watershed.

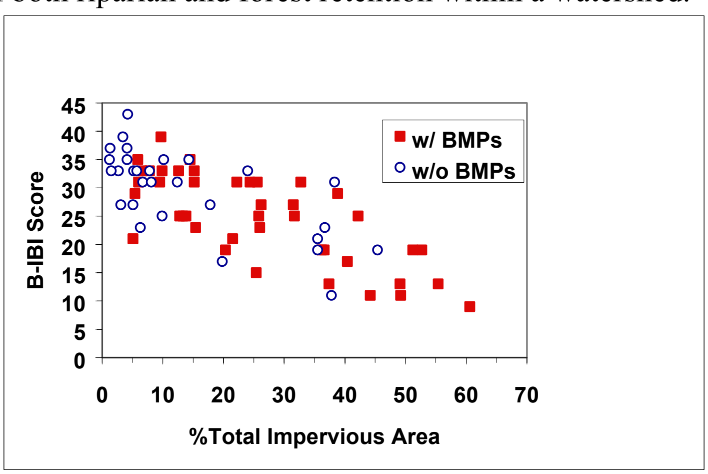


Figure 1. Puget Sound Benthic Index of Biotic Integrity (B-IBI) Over a Gradient of Watershed Total Impervious Area (% TIA) With and Without Structural BMPs

In Puget Sound, both biological measures declined with TIA increase without exhibiting a threshold of effect; i.e., declines accompanied even small levels of urbanization (May 1996; Horner et al. 1997; May et al. 1997). However, stream reaches with relatively intact, wide riparian zones in wetland or forest cover exhibited higher B-IBI values than reaches equivalent in TIA but with less riparian buffering. Until TIA exceeded 40 percent, biological decline was more strongly associated with hydrologic fluctuation than with chemical water and sediment quality decreases. Accompanying hydrologic alteration was a loss of habitat features, like large woody debris and pool cover, and deposition of fine sediments that reduce dissolved oxygen in the bed substrata where salmonid fish deposit their eggs.

Phase 3: Ecological Benefits of Riparian & Forest Retention Results and Discussion

Observation in the Puget Sound study area of the role played by riparian and upland forest retention in maintaining stream ecology suggested that their benefits might be found in other regions having different aquatic ecosystems. If similarity were demonstrated, the finding would not only serve the pragmatic need for targeting management attention, but would also continue to develop the picture of general unity among ecoregions. In Phase 3 of the project, the hypothesis was tested in the Montgomery County, Austin, and Vail study areas using the data collection and analysis methods developed in the Puget Sound study. Invertebrate data from each program were used to develop multi-metric community indices appropriate for prevailing ecological attributes but similar in complexity.

An Index of Riparian Integrity (IRI) was developed in a manner similar to the B-IBI formulation (Fore, Karr, and Wisseman 1996) to express with one number the key attributes of riparian zones (Table 1). Scores of 1 to 4, representing poor to excellent ratings or riparian buffering, were assigned to six attributes according to two measures of the lateral extent of the buffer, human encroachment into the buffer, corridor continuity, and two measures of the riparian vegetative cover. The six scores were summed and divided by the total possible score to express the IRI as a percentage of maximum value. The IRI should include metrics that measure each of the main components of natural riparian ecosystem integrity. These measures will vary depending on the ecoregion and the unique structural and functional elements of regional riparian integrity.

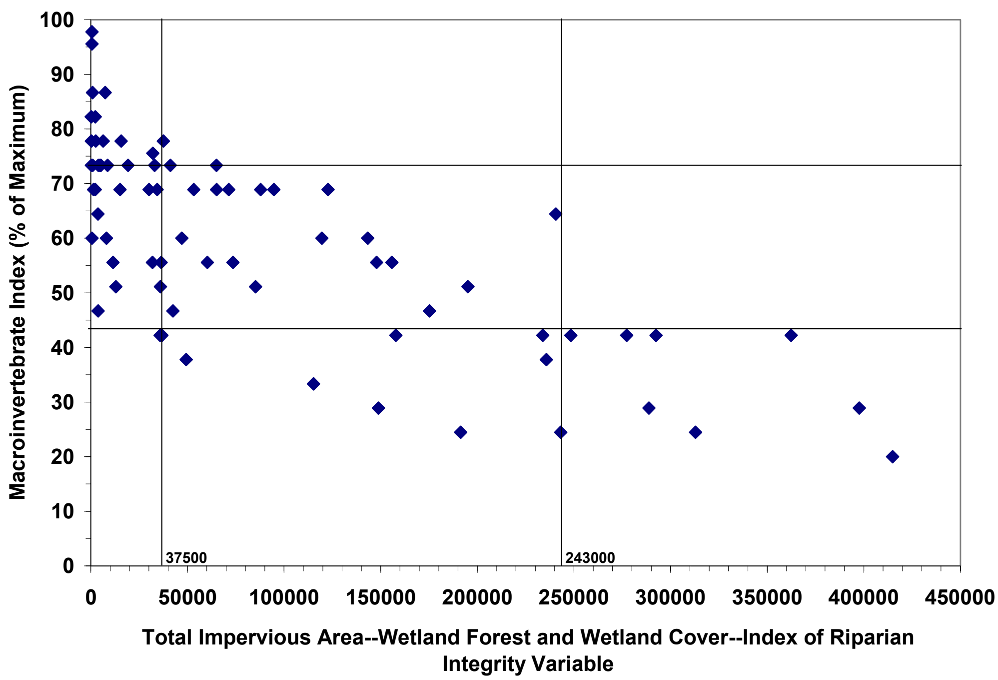
Table 1. Puget Sound Index of Riparian Integrity Metrics and Scoring Criteria

Index of Riparian Integrity Metric	Excellent (4)	Good (3)	Fair (2)	Poor (1)
Width (lateral extent > 30 m, %)	> 80%	70-80%	60-70%	< 60%
Width (lateral extent > 100 m, %)	> 50%	40-50%	30-40%	< 30%
Encroachment (% < 10 m wide)	< 10%	10-20%	20-30%	> 30%
Corridor continuity (crossings/km)	< 1	1-2	2-3	> 3
Natural cover (% forest or wetland)	> 90%	75-90%	50-75%	< 50%
Mature native vegetation or wetland (%)	> 90%	75-90%	50-75%	< 50%

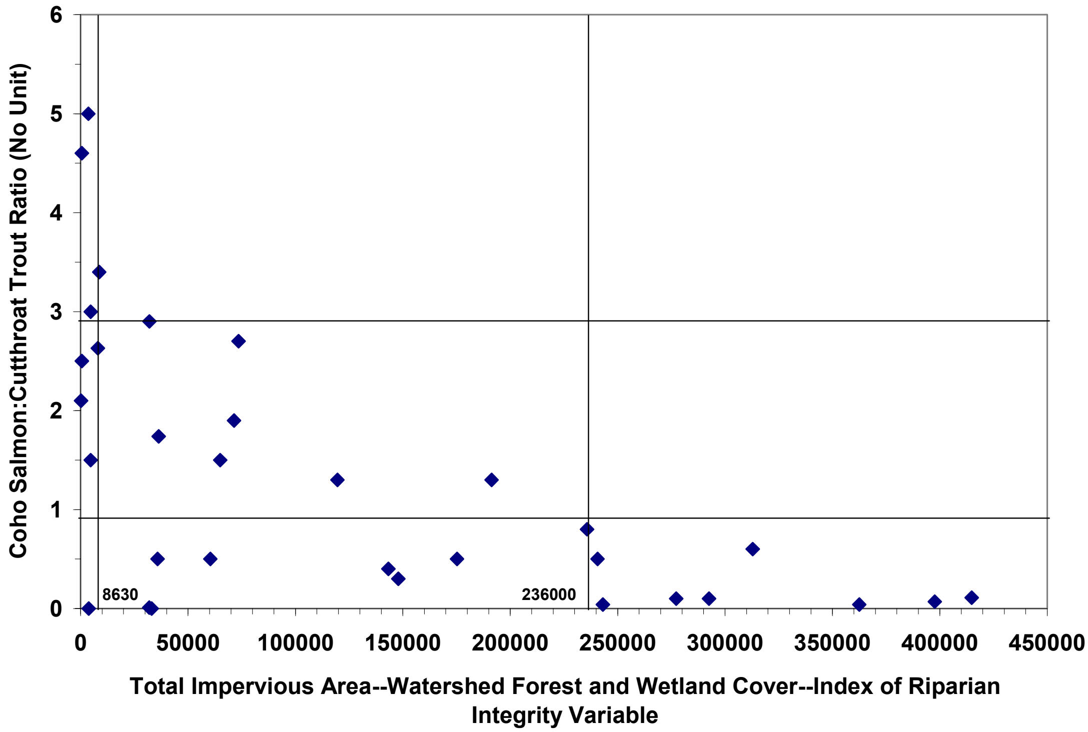
The principal objective of the analysis was to compare patterns among the study locations of aquatic biological response to urbanization and the retention of watershed forest and wetland cover and stream riparian buffers. To permit comparison among study regions, invertebrate indices in each case were converted to percentage of the maximum possible score for the location. The coho salmon:cutthroat trout ratio (CS/CT) was an additional biological variable employed in Puget Sound data analysis. It was revealing in making these comparisons to plot biological measures against independent variables representing combinations of urbanization and the *de facto* non-structural BMPs. These variables were constructed to combine the hypothesized negative effects of urbanization (expressed as TIA) and loss of the non-structural elements (% watershed forest and wetland cover, Index of Riparian Integrity).

Figures 2a and 2b present plots of biological measures versus one of the combination variables constructed to represent the watershed attributes, in this case multiplying the effects of impervious area, forest and wetland cover, and riparian integrity for the Puget Sound study sites. Plots of similar data from the Montgomery County and Austin study sites were similar. Vail data did not exhibit these trends.

Figure 2. Puget Sound Biological Community Indices Versus (% Total Impervious Area, TIA)*(100 - Forest and Wetland Cover)*(100 - Index of Riparian Integrity, IRI) Variable [*Note: Upper and lower horizontal lines represent indices considered to define relatively high and low levels of biological integrity, respectively. Left and right vertical lines indicate maximum TIA associated with high biological integrity and minimum TIA associated with low biological integrity, respectively. Numbers near the vertical lines are horizontal axis-intercepts.*]



(a) Macroinvertebrate Index



(b) Coho Salmon:Cutthroat Trout Ratio

Figures 2a and 2b, along with the graphs for other combination independent variables not shown, exhibit several trends consistent among regions and ways of viewing the data:

1. The very highest biological indices in all cases are at extremely low values of the combination independent variables, meaning that in three different regions of the nation the best biological health is impossible unless human presence is very low <u>and</u> the natural vegetation and soil systems are well preserved near streams and throughout watersheds. These most productive, "last best" places can only be kept by very broadly safeguarding them through mechanisms like outright purchase, conservation easements, transfer of development rights, etc.
2. Biological responses to urbanization in combination with loss of natural cover do not indicate thresholds of watershed change that can be absorbed with little decline in health, the same as seen in the plots of biological measures versus TIA alone in earlier reports on this work (Horner and May 1999; Maxted 1999).
3. Regardless of location or variables considered, relatively high levels of biological integrity cannot occur without comparatively low urbanization and intact natural cover. However, these conditions do not guarantee fairly high integrity and should be regarded as necessary but not sufficient conditions for its occurrence.
4. In contrast, comparatively high urbanization and natural cover loss make relatively poor biological health inevitable.
5. In all cases the rates of change in biology are more rapid to about the points representing crossover to relatively low integrity (the intersections of the lower horizontal and right-hand vertical line), and then further decline becomes somewhat

less rapid. This pattern is probably a reflection of communities with organisms reduced in variety but more tolerant of additional stress.

6. The points at which landscape condition takes away the opportunity for good biological health, or alternatively assures poor health, are similar among the study locations but deviate somewhat numerically. While these results might be put to general use in managing streams elsewhere, quantitative aspects should not be borrowed.
7. Comparing Puget Sound fish and macroinvertebrates, coho salmon exhibit more rapid rates of decline with landscape stress, lower points at which the quite healthy communities can exist, and also lower points of poor health.

In viewing these data, a reasonable question is whether or not protecting more forest and wetland, riparian buffer, or both can confidently be expected to mitigate increased urbanization. This question has considerable significance for the ultimate success of clustering development within low-impact designs to sustain aquatic ecosystems. In beginning to think about this issue, it must first be reiterated that if the goal is to maintain an ecological system functioning at or very close to the maximum levels seen, the answer is no. If the goal is to keep some lower but still good level of health, or to prevent degradation to a poor condition, though, the findings suggest that there is probably some latitude.

Phase 3 Detailed Structural BMP Assessment

Introduction and Methods

Specific, direct evidence of the effectiveness of stormwater structural BMPs in protecting aquatic biota and receiving water beneficial uses is extremely sparse. To add to this minimal information base, the Puget Sound component of the study conducted an intensive BMP assessment in the watersheds of four of its stream reaches, two in Big Bear Creek and one in its tributary Cottage Lake Creek (King County, WA), plus one in Little Bear Creek (Snohomish County, WA). Having received extensive management attention because of its rich salmonid fauna, the Big Bear Creek system has relatively large numbers of structural BMPs for its development level. The Little Bear Creek reach has relatively few structural devices for the urbanization level. Sites were divided in this way because of the observation in earlier work that BMP service level (density of coverage) varied widely among the urban catchments in the study and, as seems logical, is a factor in effectiveness. These five catchments contain a total of 165 individual BMPs, about 6.5 percent of the more than 2500 found in the entire regional survey.

All BMPs were located and visited in the field, maintenance condition was noted in both quantity and quality control facilities. The assessment went beyond service level to encompass quality of implementation as well. Implementation quality was rated according to a BMP Performance Index developed for this purpose. The indexing system encompasses structural BMPs designed to control the quantity of stormwater runoff (generally, peak flow rates) or its water quality, as well as those intended to serve both purposes. Quantity control BMPs (mostly dry detention ponds and below-ground tanks and vaults, plus a few infiltration facilities) were rated in terms of their estimated replacement of natural soil and vegetation storage lost in development. For runoff treatment BMPs, implementation quality was gauged according to recognized design and maintenance standards for maximizing performance, which were expressed as condition scores. King and Snohomish County stormwater management agency files had information on almost all of the BMPs, which supplemented the field data collection and observations.

Table 2 summarizes the characteristics of the catchments and BMPs given detailed attention. The Big Bear and Cottage Lake Creek watersheds have the greatest coverage with structural BMPs among the 38 studied in the regional project. However, only about one-sixth to one-third of the developed area even has quantity control BMPs, the primary management objective in these salmonid streams subject to habitat destruction by more frequent elevated flows after urbanization. The average facility was built before the mid-1980s in the Cottage Lake Creek watershed, where many BMPs are below ground. Those serving Big Bear Creek average 5 years younger and tend more to be wet ponds.

Table 2. Characteristics of Watersheds in Detailed Structural BMP Assessment

Characteristic[a]	Cott-2[b]	BiBe-1[b]	BiBe-4[b]	LiBe-2[b]
Catchment:				
Catchment area (km^2)	17.5	9.5	29.5	16.9
% developed	66.8	44.0	50.0	67.8
% impervious	11.1	6.6	8.3	9.9
Quantity Control BMPs:				
No. Qn BMPs	56	22	59	17
% Qn BMPs below ground	41.1	9.1	32.2	11.8
% developed area with Qn BMPs	30.9	24.2	15.9	11.5
Average age Qn BMPs (y)	13	8	8	9
Quality Control BMPs:				
No. Ql BMPs	11	22	49	5
No. infiltration devices	4	3	3	0
No. wet ponds	5	11	25	5
No. wet ponds that are also Qn BMPs	4	9	24	4
No. biofilters (swales, filter strips)	2	8	21	0
% developed area with Ql BMPs	4.6	15.4	13.5	3.4
Average age Ql BMPs (y)	11	8	7	9
Quantity and Quality Control BMPs:				
Total no. BMPs	63	35	84	18
Stream Biology:				
Benthic Index of Biotic Integrity	33	29	33	25
Coho Salmon:Cutthroat Trout Ratio	2.9	5.0	3.4	1.7

[a] Qn—quantity control; Ql—quality control; average ages are at time of stream ecology work; infiltration devices considered to be both quantity and quality controls;

individual BMPs total 165, but table numbers do not sum to that total because some have combined functions and upstream BMPs also serve downstream stations.
[b] Cott-2—Cottage Lake Creek site 2; BiBe-1,4—Big Bear Creek sites 1 (upstream) and 4 (downstream); LiBe-2—Little Bear Creek site 2.

The quality control service levels are even lower, especially in the older Cottage Lake Creek developments (<5 percent of developed area). The much higher numbers in the Big Bear Creek catchments indicate the turn to quality control along with quantity control in the heavy development period there around 1990. The wet pond is the most prominent BMP type, somewhat exceeding biofilters in numbers. Most wet ponds perform double service as quantity control ponds with live storage too. Many installations are wet pond-biofiltration swale treatment trains, with ponds usually but not always draining into swales. Facilities expressly designed to be infiltration devices are relatively uncommon in these glacial till catchments.

Results and Discussion

The BMP analysis shows that < 4 percent of soil and vegetation storage lost to development was recovered by BMPs in the Cottage Lake and Big Bear Creek catchments, and approximately 1 percent in the Little Bear Creek cases. These very low percentages are in strong contrast to the proportions of developed areas having quantity control BMP storage, which are about an order of magnitude greater, although still far from complete. This dichotomy signifies inadequate standards for designing these BMPs. Achieving the full potential of water quality treatment was similarly low. The Cottage Lake Creek catchment scored near the Big Bear ones despite a much lower service level because of substantially more infiltration there, a factor also reflected in its quantity control score.

This investigation started out to examine if the highest BMP service levels make a demonstrable difference in stream biological integrity. However, the mitigation potential provided by even these service levels proved to be so small that this question still cannot be conclusively answered. Biological measures are indeed lower in the relatively less served Little Bear Creek catchment, but factors other than structural BMPs could be responsible. Table 3 summarizes these potential factors for the five intensively studied catchments and two others with similar development but no structural BMPs at all. All of these streams are still producing salmon (generally, several species) and are thus resources to which strong management attention should be directed.

Table 3. Watershed and BMP Conditions and Stream Biological Integrity in Eight Cases with Total Impervious Area in the Approximate Range of 5 to 10 Percent

Condition[a]	Cott-2	BiBe-1	BiBe-4	LiBe-2	GrCo-2	LiSo-1
TIA (%)	11.1	6.6	8.3	9.9	7.8	6.3
B-IBI	33	29	33	25	33	23
CS/CT	2.9	3.4	5.0	1.7		
% forest & wetlands	33.2	56.0	50.0	32.2	76.5	69.3
IRI	55.5	87.5	79.2	45.8	79.2	33.3
Qn score	2.0-3.9	1.5-3.0	1.2-2.4	0.8-1.6	0	0
Ql score	4.1	5.4	4.2	0.7	0	0

[a] TIA—total impervious area; B-IBI—benthic index of biotic integrity; CS/CT—coho salmon:cutthroat trout ratio; IRI—index of riparian integrity; Qn—quantity control; Ql—quality control.

Table 3 does not present an entirely consistent picture. The Green Cove Creek reach equals the highest B-IBI among these sites without structural BMPs but has high levels of forest, wetlands, and riparian buffer preservation. The LiBe-2 and LiSo-1 sites exhibit the lowest B-IBI values and also substantially lower riparian indices than the other locations. Still, Cott-2 also equals the highest B-IBI with the highest and oldest development, nearly the least forest and wetlands, and only moderate IRI. It cannot be dismissed that this system is holding its level of health with the contribution of structural BMPs, even with their overall low service level and quality of implementation. Big Bear Creek has been the beneficiary of a King County program of fee-simple and conservation easement purchases that has encompassed 10.4 and 3.6 percent of the BiBe-1 and 4 catchments, respectively. These efforts are undoubtedly contributing to the thorough riparian buffering and moderate forest and wetlands retention seen there. Still, in biological measures these sites do not rise above the nearby Cottage Lake Creek catchment, which has very little (0.2 percent of the catchment) of these protected lands.

The analysis determined that, even in the watersheds around Puget Sound best served by structural BMPs, a distinct minority of the development has any coverage at all. The existing BMPs mitigate very small percentages of the hydrologic and water quality changes accompanying urbanization. What is probably the safest observation is that many sources of natural variation in these ecosystems make clear-cut definition of cause and effect elusive. However, the general conclusion of the primacy of riparian buffering drawn in the preceding section appears to be upheld by these observations, and structural BMPs cannot be dismissed as contributing. Verification of that premise and delineation of how much protection they can actually afford requires their thorough and high quality implementation and then follow-up ecological study.

Phase 3 – Relationship of Structural and Nonstructural BMPs

Stormwater and urban water resources management first developed around the concept of structural BMPs but recently broadened to encompass principles often given names like conservation design and low-impact development. Most fundamentally, these principles guide where to place development and how to build it to minimize negative consequences for aquatic ecosystems. There are many specific tools to implement them, but they fit generally into the broad categories of separating development from water bodies (i.e., retaining riparian buffers); limiting impervious area in favor of natural vegetation and soil, especially forest cover; and strategic and opportunistic use of structural BMPs. The Puget Sound database offers some opportunity to examine how these structural and non-structural strategies might fit together and what they can accomplish in different urbanization scenarios.

Figure 3 encompasses the various general elements of conservation design and how they relate to stream biology in terms of macroinvertebrates. Structural BMPs are expressed as the density of BMP coverage per unit area of impervious surface (sites with TIA <5 percent do not have structural BMPs and are excluded). Non-structural practices are represented as the product of watershed forest and wetland cover (percent) times index of riparian integrity (percent of maximum) and graphed for the highest, intermediate, and lowest one-third of the resulting numerical values.

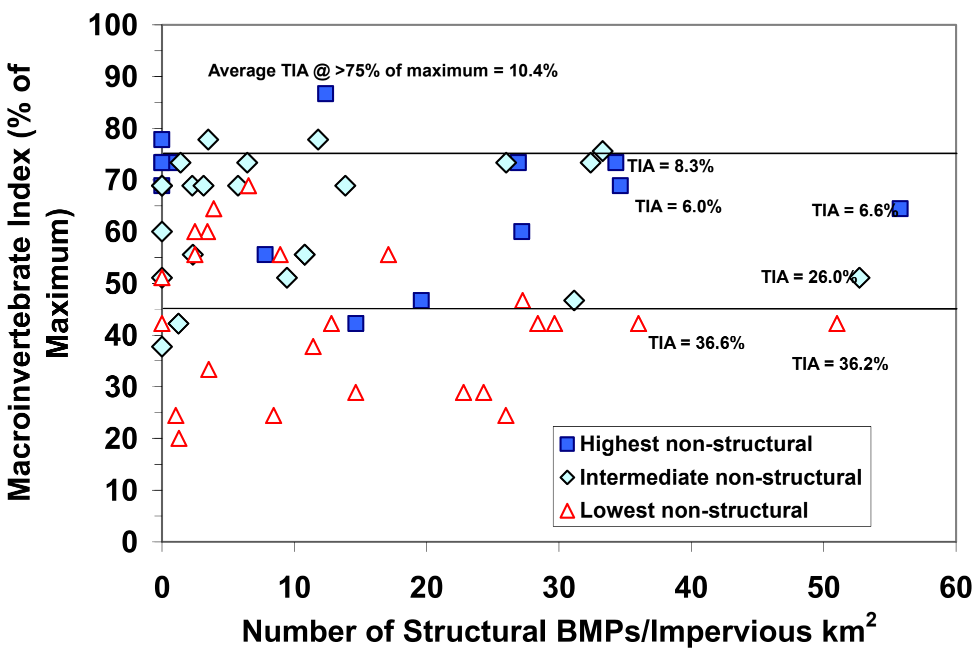


Figure 3. Macroinvertebrate Community Index Versus Structural BMP Density with the Highest, Intermediate, and Lowest One-Third of Natural Watershed and Riparian Cover [*Note: Upper and lower horizontal lines represent indices considered to define relatively high and low levels of biological integrity, respectively.*]

The first observation that should be made about Figure 3 is that the five highest macroinvertebrate indices are not represented, because they are from sites with <5 percent TIA. It is apparent that neither structural nor non-structural measures, at least at the levels represented in this database, can provide for the highest benthic macroinvertebrate integrity if any but the most minimal development occurs.

It can further be observed in Figure 3 that points at the left (relatively few BMPs) disperse widely over the macroinvertebrate index range. Some sites with little forest, wetland, and riparian retention rise into the intermediate biological integrity zone (45 to 75 percent of maximum index value), while a few locations with higher non-structural measures fall close to or into the region of relatively low ecological health. This observation is an expression of what is also apparent in Figures 2a and 2b, namely that a certain ecological status is not assured by any condition, or even combination of conditions, but is only more likely with those conditions.

The Figure 3 points converge with increasing structural BMP density, overall and in each non-structural category. Sites with the lowest macroinvertebrate indices (and also highest urbanization and lowest non-structural measures) appear to benefit from structural BMP application. Those with higher biological and natural cover measures and lower urbanization do not, with the result that points tend toward the intermediate biological level. If ecological losses are to be stemmed at high urbanization, structural BMPs appear to have a substantial role. In this situation development has taken forests and wetlands and intruded into riparian zones, reducing the ability to apply non-structural options.

Any conclusions from this analysis must be tempered according to the scope of the underlying data. Probably the leading factor giving caution is that no instances exist of structural BMPs being exceptionally widely applied and designed to mitigate a large share of the known impacts of urbanization. Therefore, the fullest potential of these practices has not been examined, and it is possible that extremely thorough applications would demonstrate additional benefits not suggested in these data.

Phase 4: Watershed Assessment Tools and Analysis

Analyses during Phases 1-3 yielded substantial insights on the functioning of stream ecosystems in relation to watershed conditions and information that can be applied to improve watershed management. These conclusions were built largely on general independent variables that aggregate watershed attributes: (1) total impervious area, representing development; and (2) proportion of the watershed in forest and wetland cover, representing pervious land best preserved in a natural state. The earlier analysis was able to get more specific with riparian zone definition, having developed an index of riparian integrity that assimilates six characteristics representing the lateral and longitudinal extent of vegetated riparian land, the quality of the vegetation cover, and its continuity (freedom from crossings and intrusions by human works and activity).

The goal of the first task in Phase 4 was to develop a watershed assessment protocol to provide guidance for performing a quantitative assessment of the ecological health of a watershed based on landscape-level characteristics. The resulting protocol also provides a road map for developing a relative risk model for a watershed using inputs from stakeholders. GIS analytical techniques are used to relate multiple parameters that potentially impact the ecological health of the watershed. The assessment uses the principles of landscape ecology, which takes into account the spatial arrangement of the components or elements that make up the environment.

We then used these techniques to further analyze the data from Austin, Montgomery County, and Puget Sound databases to see if they reduce dispersion seen in the data, especially at relatively low urbanization, and give a more incisive functional portrait and interpretations that can further improve management guidance. These analyses employed various graphical, statistical, multivariate, and indexing techniques.

All three regions represented in the study developed benthic macroinvertebrate indices based on community metrics appropriate for the regional ecology. In each case the index consisted of a summation of scores for the metrics. With differing numbers of metrics, the respective regional indices had different maximum and minimum values. To place all indices on a common base for comparison, each was expressed as "percent of best integrity" according to:

$$\text{\% of Best Integrity} = \left(\frac{Score - Minimum}{Maximum - Minimum}\right)x100$$

The same procedure was applied to the watershed condition index developed as described below, as well as to the Montgomery County fish index of biotic integrity.

The classifications of pervious and impervious land cover made possible by the complete GIS databases produced in the first Phase 4 task provided much more information about conditions at the watershed, riparian, and local scales than previously available. Regional data analysis began with routine statistical explorations of possible associations between stream ecological variables and these various environmental attributes at the several scales. These examinations were performed with Statistical Program for the Social Sciences (SPSS) 10.1 for Windows software.

The first investigation was bivariate correlation analyses on the full matrix of landscape and ecological variables. The intent was to identify the landscape variables exhibiting the highest correlations with measures of habitat quality and the macroinvertebrate and fish communities to inform subsequent analyses. The next exploration employed multiple linear regression techniques. The third exploratory analysis involved the development of logistic regression equations. A logistic regression equation allows predicting the probability that an ecological measure is in a certain group (e.g., >75 percent of the maximum possible value) based on one or more independent variables (here, landscape measures).

The utility of a numerical index incorporating land cover variables was explored for each region as an alternative to the multiple linear and logistic regressions equations developed as described above. Development of the indices followed a procedure analogous to that used by Fore, Karr, and Wisseman (1996) for the B-IBI and Horner et al. (2002) for the IRI. The Montgomery County data set consists of more than 460 stream reaches and was randomly divided into two subsets for initial WCI development and later independent verification of trends and models generated using the index. However, the full Austin and Puget Sound databases were used in the development process.

Development of a WCI for Puget Sound began with the selection of nine possible metrics. The majority were chosen because of their relatively high correlation with B-IBI and their representation of urbanization (TIA) and buffering (forest cover) in the watershed as a whole and the riparian zones relatively near (within 50 meters) and distant (up to 300 meters) from the stream. For initial trials three additional variables were selected to represent transportation land use (road density, km/km^2), riparian fragmentation (breaks/km), and urbanization in a 300-meter diameter local zone. upstream of the sampling location (as paved plus urban grass-shrub cover). Exploratory analyses indicated the potential utility of these metrics.

Austin and Montgomery County WCI development started with, respectively, nine and five metrics chosen using similar considerations as applied to the Puget Sound data. For Austin these trial selections were: TIA in the overall watershed and 10- and 100-meter riparian zones; transportation land use in the watershed and the 100-meter riparian corridor; TIA and natural land cover in a local zone 100 meters on each side of the stream extending 1 km upstream from the sampling point; commercial land use in the watershed; and stream road crossings. The initial Montgomery County choices were watershed TIA and land cover by roads, roofs, parking, and native forest.

The utility of the trial regional WCIs was examined by first plotting available biological variables against the index. Various models were investigated using Excel software to explain the relationship between biology and land cover as represented by the trial WCI (e.g., linear regression and variable transformations to assess logarithmic, power, and exponential regression fits). The adjusted R^2 was employed as the first screen of the models. With these evaluations of the regional WCIs complete, some trial adjustments were then made to see if the addition of one or more land cover variables would improve the model. Also, the deletion of one or more variables was attempted to see if improvement would occur, or if the model would be just as acceptable with a smaller number of metrics, and thus be less demanding of input data. The evaluations of these alternative indices were according to the same basis as outlined for the initial trial WCI. Once the most appropriate models linking biological variables to the regional WCIs were identified, confidence limits for the estimates of the biological variables were computed (95, 90, and/or 80 percent limits, depending on the regional data set) using SPSS software.

As the final step in evaluation of Watershed Condition Indices, discriminant function analyses were performed using SPSS to see if using the WCI and its component variables independently would yield similar outcomes. If so, the similarity would be a sign of relative robustness in the WCI to place sites in their proper groups. Discriminant function analysis is a technique for combining independent variables (in this case, the land cover variables comprising the WCI) into a single new variable, the discriminant function, that best discriminates among values of the dependent variable according to a criterion based on the statistic Wilks' lambda (Everitt and Dunn 2001).

Table 4 lists the final composition of the watershed condition indices for the three regions. There are some common elements in the WCI metrics for the three regions. Total impervious area and components making up impervious land use (e.g., automotive-related land covers, roofs) predominated in the selections watershed-wide and over a range of buffer scales. This dominance points out the importance of obtaining good measures of impervious land cover in performing watershed analyses. Forest cover was also prominent. A local-scale metric was less instrumental but was useful to improve the representation of watershed conditions in the two cases where it was available.

Table 4. Metrics Incorporated in Watershed Condition Indices for Three Regions

PUGET SOUND[a]	AUSTIN[b]	MONTGOMERY COUNTY[c]
Watershed forest	100-m buffer transport	Watershed roads
Watershed TIA	Watershed TIA	Watershed roofs
300-m buffer TIA	Watershed transport	Watershed TIA
300-m buffer forest	10-m buffer TIA	Watershed parking
50-m buffer TIA	100-m buffer TIA	Watershed native forest
50-m buffer forest	Local natural land cover	
300-m local paved + urban grass-shrub		

[a] Forest--≥ 86% of pixels in forest cover; TIA—total impervious area; local—300-meter diameter zone upstream of the sampling location.

[b] TIA—total impervious area; transport—any transportation land use; local—100 m on each side of the stream extending 1 km upstream from the sampling point.

[c] TIA—total impervious area.

Discriminant function analyses (DFA) were employed to compare the classification of stream sites in terms of biological health using the Watershed Condition Indices versus performing the same classification with the individual component variables making up the WCIs. The individual variables were treated in two different ways: including all in the DFA and entering them into the analysis in a stepwise fashion based on statistical acceptance criteria. Similarity in classification would demonstrate the relative robustness in the indices as formulated to place sites in their proper groups

The results revealed that sites in the highest and lowest integrity groups were generally more successfully classified by all three methods than those in the intermediate categories. There was no consistent pattern indicating that using the variables individually by either method either improved or diminished classification accuracy. Therefore, the WCI formulations appear to provide valid means of characterizing watershed conditions and conducting analyses involving the biology of streams in these catchments. The aggregate formulation offers the advantage of being easier to use in numerical and statistical analyses than a host of variables.

Plots of the first discriminant function scores for multiple-variable DFAs versus WCI produced relationships close to linear in all cases. This result is a sign that the development of WCI by a numerical indexing technique and DFA using WCI's component parts lead to similar outcomes, helping confirm the validity of both approaches.

Graphical Analyses of Biological Metrics Relative to WCIs

Figure 4 portrays B-IBI plotted against WCIs for the Puget Sound region. To allow inter-regional comparisons, both biological indices and WCIs are expressed as percentages of the "best" possible values, as outlined earlier in the Methods for Comprehensive Data Analysis. Letters on the graphs (e.g., **A**, **B**) denote watershed conditions generally necessary to reach certain levels of biological integrity. In the Puget Sound region achieving B-IBI $\geq$ 85 percent of maximum integrity requires that WCI be at least 75 percent of the best value (**B** on Figure 4), with most of the highest B-IBI scores lying above a WCI of 90 (**A**). While these watershed conditions are generally necessary for good biological health, they are not sufficient alone, as demonstrated by the numerous points representing lower biological integrity at relatively high WCI values. The land cover data collected in this work do not allow exploring the many potential reasons for the failures to achieve good biological conditions when watersheds are not heavily developed. Nevertheless, this analysis identifies the key watershed conditions that must be provided if there is to be any chance of meeting relatively high biological goals.

Point **D** on Figure 4 indicates that the B-IBI was inevitably below 50 percent of the best if WCI fell beneath 35 percent, and always dropped again to under 30 percent with WCI less than 20 percent (**E**). Therefore, while poor biology can occur even with moderate or even little disturbance by urbanization, this outcome is invariable with heavy levels of disturbance.

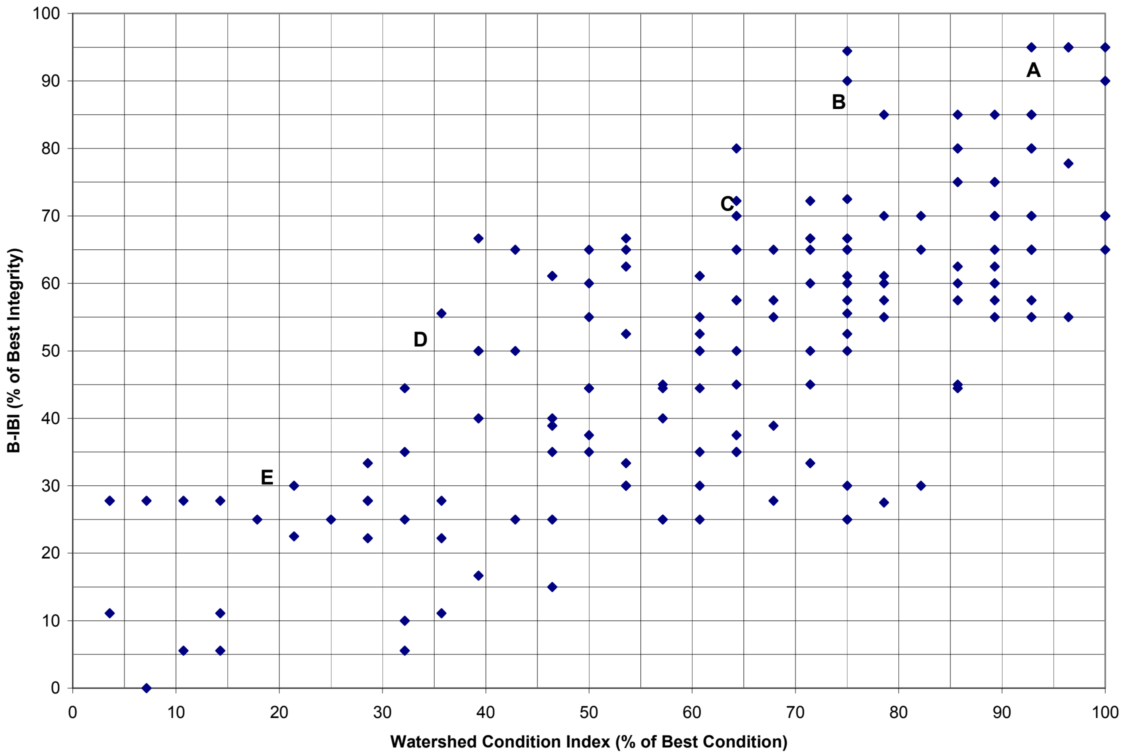


Figure 4. Benthic Index of Biotic Integrity in Relation to Puget Sound Watershed Condition Index [*Note:* **A**, **B**, **C**, **D**, *and* **E** *represent WCIs generally associated with B-IBI > 90, ≥ 85, ≥ 70, < 50, and < 30 percent of best integrity, respectively.*]

Similar plots of the relationships between the biological indices and WCIs for the other two regions were created. The trends are similar to those noted for Puget Sound although less sharply defined, mirroring the lower correlation and regression coefficients in these data seen in statistical examinations. The highest levels of health in the invertebrate communities were generally seen only when WCIs were > 80 and ≥ 70 percent of best scores for Austin and Montgomery County, respectively. The lowest levels always occurred with WCI < 25 percent in both cases.

Summary and Conclusions

1. Relationships Among Watershed Conditions and Stream Biology

Coordinated studies in three different regions in the United States related stream biological communities to land use and land cover attributes of the watersheds draining to their habitats. Biological communities were defined in terms of multi-metric benthic macroinvertebrate indices developed for each region and indices characterizing fish communities in two regions. Initially, watershed land cover was defined in terms of total impervious area (TIA); the proportion covered by forest; and six variables describing the extent, continuity, and vegetative cover of riparian buffer zones. Later, geographic information system (GIS) analysis more specifically delineated watershed pervious and impervious cover. The intent with this classification was to represent not only the amount of these general land surface types but also their character and the activities occurring there.

GIS data were used to develop multi-metric Watershed Condition Indices (WCIs) for each region. Metrics comprising the WCIs are either relatively highly correlated to biological indices or were identified in preliminary stepwise multiple and logistic regression exercises as instrumental in linking watershed and aquatic biological states.

Among all three regions the most prominent landscape variables relatively highly correlated to biological metrics are measures of total impervious area and forest cover at the watershed scale and in riparian buffer zones over a range of widths. The regression exercises pointed out, in addition, some instrumental features of local areas not necessarily in riparian zones but within certain distances near streams. WCI composition was fine-tuned by determining the combination of metrics giving the best linear or exponential model fits when biological indices were regressed on WCIs.

General Observations

Graphical portrayals of biological indices versus measures of watershed attributes, both the initial set and the WCIs, were very useful in revealing a number of relationships between stream biota and the upland surroundings. Most striking is that the highest biological indices in all cases are associated only with the highest WCI values, representing no or extremely low urban development, very high forest retention, and minimal human intrusion in riparian zones. It was therefore demonstrated in three different regions of the nation that the best biological health is impossible unless human presence is very low <u>and</u> the natural vegetation and soil systems are well preserved near streams and throughout watersheds. However, while these conditions are necessary for high integrity, they are not sufficient by themselves to guarantee it. Other circumstances not captured in the GIS-based watershed analysis must also be instrumental.

An additional observation common among regions was that biological responses to urbanization in combination with loss of natural cover do not exhibit thresholds of watershed change that can be absorbed with little decline in health. Instead, decline was seen to start in the earliest stages of land conversion to human occupation. Rates of change in biology are relatively rapid in these early stages and then progressively slow with further urbanization. This pattern is probably a reflection of biological communities populated, more and more in the progression of human influence, with organisms reduced in variety but increasingly tolerant of additional stress.

Furthermore, in all three regions comparatively high urbanization and natural cover loss make relatively poor biological health the inevitable outcome. Thus, little or no urbanization and widespread preservation of natural land cover allows the existence of rich aquatic biological communities, although does not guarantee them. In contrast, extensive conversion to impervious and less pervious surfaces does guarantee depauperate ecosystems.

Along with these common general trends among regions, there is a fair degree of unity in the specific watershed conditions associated with the highest and lowest levels of biological integrity. Taking at least 80 percent of best integrity as an example definition point for good benthic invertebrate community health, WCI in the range of 70 to 80 percent of best watershed condition is essential in all three regions to attain this biological state. A watershed index at least at the lower end of this range is also necessary for clear dominance (in the ratio of at least 3:1) of the over-wintering Puget Sound salmonid fish community by coho salmon instead of the more tolerant cutthroat trout.

At the opposite end of the biological spectrum, poor invertebrate community health (taken for example as under 40 percent of best integrity) occurs in each region, excepting only two cases, at WCI = 25-30 percent of best condition and below. Cutthroat trout dominance is also assured under these watershed conditions in the Puget Sound streams.

Quantification of Results

Several statistical and multivariate analytical techniques were applied to evaluate the Watershed Condition Index and devise formal mathematical constructs to increase its utility as an assessment and management tool. Discriminant function analyses validated the regional WCIs, and, independently, their component variables, as mechanisms for classifying biological integrity according to watershed condition. Sites in the highest and lowest integrity groups were generally more successfully classified in these analyses than those in the intermediate categories.

A second multivariate technique applied to the data was logistic regression analysis. This analysis produced equations forecasting the probability of a stream's invertebrate or fish community being in selected groupings of biological integrity based on WCI:

$P = e^L/(1 + e^L)$ and $L = b_0 + b_1(WCI)$

where: P = Probability of membership in a given biological integrity group (> 0.5 to assign membership);
e = Base of natural logarithms;
L = Logit function;
b_0 and b_1 = Constant and logistic regression coefficient derived in the analysis, respectively (Table 9); and
WCI = Watershed condition index (% of best condition).

When applied to the original data and an independent Montgomery County data set held aside for model verification, the equations were more successful in predicting that a site would not have a certain biological condition than forecasting that it would fall in the specified group. Hardest to forecast with these models is very good benthic community health. This consistent observation across regions is another reflection of the necessity but not sufficiency of relatively high WCI for high biological integrity, with many points representing fairly natural watershed conditions still being degraded biologically. The models were more successful, although still inconsistent, in predicting membership in degraded biological groups than in high quality categories.

The generally limited ability of the equations to predict group membership, in contrast to the greater success in forecasting exclusion from the group, makes this technique best suited to analyze if it is <u>possible</u>, with the existing or expected watershed condition, for a stream to achieve a high level of biological integrity or avoid a low level. The method is less reliable, and is not recommended, for assessing if the biological state actually <u>will</u> reach a certain level. The discriminant function analyses discussed above were more successful in judging actual membership in relatively high and low benthic community integrity groups. The techniques can be used in concert to assist in judging how likely a certain biological state is for a particular case. It must always be recalled, though, that actual achievement of the best biological health depends on some factors yet to be defined.

To bring in a more formally quantitative view supplementary to the earlier graphical observations, the Puget Sound logistic regression equations for the macroinvertebrate (B-IBI) and fish communities (coho salmon:cutthroat trout ratio) were applied to hypothetical watershed conditions. The results give strong evidence of very low probability for relatively healthy invertebrate and fish communities with WCI much under 70 percent, a conclusion agreeing with the graphical interpretation. WCI in the range from 79 down to 57 percent of best condition is a region of rapid loss of prospects for high biological integrity. B-IBI $\leq$ 45 percent of best integrity is highly probable as WCI goes below 45 percent. Decline of the coho salmon:cutthroat trout ratio to 1.0 is very likely around the same WCI. A heavily depleted benthic community (B-IBI $\leq$ 25 percent) becomes probable just under WCI of 20 percent. These tendencies too echo those observed on the graphs.

The more clear-cut results at relatively high compared to low urbanization render these methods most useful in the more urban areas to analyze how to prevent already deteriorated biological integrity to even lower levels, or to improve health somewhat. They can also be applied at very low urbanization, but only with the clear realization that favorable watershed conditions are necessary but not sufficient for confidently predicting good biological health.

<u>2. The Role of Structural Stormwater BMPs in Stream Biological Integrity</u>

Extensive and incisive investigation of how stormwater BMPs affect the portrait of aquatic biology in relation to overall watershed conditions was hindered by the very labor-intensive effort required to collect meaningful data on the numerous BMPs that often exist in urban watersheds. For example, the first approximately 40 stream basins or subbasins studied in the Puget Sound region have over 2600 BMPs. Meaningful evaluation would require detailed data of various kinds on BMP siting, design, and operation in relation to stormwater management objectives and contributing catchments.

With this dilemma the study proceeded in two directions: (1) a broad approach over all watersheds with recorded BMP presence to determine if the mere extent of BMP coverage, with no assessment of implementation quality, has an identifiable, positive effect on stream health; and (2) a deeper effort in a few watersheds to collect and evaluate the data necessary to gauge BMP implementation quality and its effect on aquatic systems.

The broad-scale approach was not very fruitful. Early graphical plots of biological versus urbanization measures for catchments with and without BMPs did not distinguish differences in biological quality between the two groups. Follow-up statistical examinations of BMP areal coverage expressed in several ways (e.g., per km^2, per unit of impervious cover), with overall watershed condition being a controlled variable, exhibited very weak or even negative partial correlations between biological integrity and BMP presence.

In the second, deeper approach, structural BMPs were intensively studied in several subbasins of two Puget Sound stream systems, one with perhaps the greatest consideration to stormwater management in the region and the other with less attention. Even in the first watershed, a minority of the developed area is served by runoff quantity control practices, and even less of it by water quality control BMPS. Much development was vested with approvals before BMP requirements took effect or was exempted on the grounds of falling below development size thresholds. Those BMPs installed are capable of mitigating an even smaller share of urban impacts, primarily because of inadequacies in design standards.

Even with these shortcomings, results indicate that structural BMPs appear to help in sustaining aquatic biological communities at fairly high urbanization levels. They give less evidence of benefit at moderate urbanization and greater natural land cover. If ecological losses are to be stemmed at high urbanization, structural BMPs appear to have a substantial role. In this situation development has taken forests and wetlands and intruded into riparian zones, reducing their roles in the watershed. In the most urban areas it seems that these roles can be assumed in part, but not in full, by structural BMPs.

The highest biological indices had no relationship to BMPs, because these high scores occurred only in watersheds with no or minimal development, where no BMPs were built. It thus could not be tested if BMPs can replace some loss in natural land cover through light urbanization and still maintain high biological integrity. However, the lack of obvious benefit seen with a moderate amount of development lends support to the hypothesis that the benefit would also be absent at low urbanization too, where relatively undisturbed streams house the most sensitive organisms.

Any conclusions from this analysis must be tempered according to the scope of the underlying data. There were no instances found of structural BMPs being exceptionally widely applied and designed to mitigate a large share of the known impacts of urbanization. Therefore, the fullest potential of these practices has not been examined, and it is possible that extremely thorough applications would demonstrate additional benefits not suggested in these data.

Recommendations

Unity in the results from three dispersed and differing areas of the nation support certain general watershed management recommendations for strong consideration elsewhere. This work also developed methods that can be broadly recommended to assist any region wishing to develop a basis for its own watershed analysis and management efforts.

A. General Watershed Management Recommendations

1. Base watershed management on specific objectives tied to desired biological outcomes.
2. If the objective is to attain an existing high level of biological integrity, very broadly preserve the extensive watershed and riparian natural vegetation and soil cover almost certainly present through mechanisms like outright purchase, conservation easements, transfer of development rights, etc.
3. If the objective is to prevent further degradation when partially developed areas urbanize further, maximize protection of existing natural vegetation and soil cover in areas closest to the stream, especially in the nearest riparian band. In the uplands, generally develop in locations already missing characteristic natural vegetation. As much as possible, preserve existing natural cover and limit conversion to impervious surfaces. The lower the level of existing development, the more important this recommendation is.
4. In addition, fully serve newly developing and redeveloping areas with stormwater quantity and quality control BMPs sited, designed, and operated at state-of-the-art levels. Attempt to retrofit these BMPs in existing developments. The higher the level of existing development, the more important this recommendation is; since much opportunity to apply the preceding recommendation is lost with extensive land conversion.
5. Where riparian areas have been degraded by encroachment, crossings, or loss of mature, natural vegetation, give high priority to restoring them to extensive, unbroken, well vegetated zones. This strategy could be the most effective, as well as the easiest, step toward improving degraded stream habitat and biology. Riparian areas are more likely to be free of structures than upland areas and more directly influence stream ecology. Also, riparian restoration fits well with other objectives, like flood protection and provision of wildlife corridors and open space
6. The above recommendations suggest that federal and state environmental management agencies should reconsider their existing water body classification systems and the associated water quality standards. This is consistent with the recommendation of the National Academy of Sciences (NRC 2001) review of the nation's total maximum daily load (TMDL) program that states needed to conduct use attainability analyses and appropriate designate the beneficial uses of water

bodies. State watershed managers need to work closely with local communities to develop water body classifications that accurately reflect the desired and achievable goals of the community for its aquatic ecological systems.

B. Recommendations for Developing Regional Watershed Analysis and Management Approaches

1. Systematically collect data on regionally representative stream benthic macroinvertebrate and fish communities. Extend the program's coverage over the full range of urbanization, from none to the highest levels with above-ground streams. Use the data to develop regionally appropriate biological community indices.
2. Develop a geographic information system to organize and analyze watershed land use and land cover (LULC) data. Collect data on regionally appropriate LULC variables, particularly measures of impervious and forested cover in the watershed as a whole, at least two riparian bands extending to points relatively near and far from the stream, and in other local areas fairly close to the stream.
3. Investigate which LULC variables are statistically best associated with biological indices, using analyses like correlation and stepwise multiple and logistic regressions.
4. Define a tentative Watershed Condition Index (WCI) using the best associated variables.
5. Choose the optimum LULC variables for the WCI on the basis of the combination yielding the best fits in statistical regressions of biological indices on WCI. These regressions are useful for fine-tuning the WCI but are unlikely to offer very good tools for predicting biology as a function of WCI.
6. Validate the resulting WCI with discriminant function analyses as described in the project's report.
7. Graph biological indices versus WCI and examine trends signifying potentially fruitful regional watershed management strategies.
8. Perform logistic regression analyses to develop means of classifying probable groupings of aquatic biological health in relation to WCI. This type of analysis was found in this study to be better at predicting if a particular case would not be in a group than if it would be.
9. Supplement the logistic regressions with discriminant function analysis, which was found to be better at forecasting if a case would fall in a group.
10. Use the two techniques in concert to make judgments like: (1) With prevailing or expected watershed conditions, is it possible for a biological state to be at the highest level or, in other situations, avoid the lowest level? (2) With these conditions, how likely is it that the state will actually attain that level? (3) What management strategies can be considered, and are most likely to be feasible and successful, to adjust watershed conditions in a way that will maximize the chance of attaining a biological objective?

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Low Impact Development at Big Box Stores

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Abstract

The use of LID presents an opportunity to change stormwater management from a compliance program to an economic asset component of the land development process. Communities across the nation are faced with the growing impact of Big Box retail development. These developments have large-scale impervious area impacts to the site as well as generating additional off-site infrastructure and development impacts. Many of the landscaping, green roof, and water and energy conservation measures used in LID can be used to reduce stormwater impacts but also provide economic and "branding" (e.g. green development and LEED) benefits to retailers. The LID Center has been working with Big Box retailers through a grant with USEPAOW. This paper will present some of the findings and opportunities that have been discovered through this effort.

Managing Impacts of Development on Stormwater Runoff: Benefits of Low Impact Development Approaches

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Abstract

Relevant information regarding the relationship between land uses and water quality was obtained through an evaluation of water quality data collected by the City of Austin, Texas. Data suggest that one way to address water quality problems in receiving streams is through implementation of Low Impact Development (LID) strategies. This paper presents the results of the data analyses, as well as a discussion of the benefits and challenges associated with implementing LID.

Introduction

The last half-century has seen an increasing awareness of the need to manage the effects of urbanization on stormwater runoff quantity and quality. Initially the regulatory focus was on mitigating increased peak flows to avoid flooding impacts in downstream areas. This focus led to the requirements for improved conveyance systems and stormwater detention structures. In the last-quarter century there has been a growing realization that urbanization also impacts water quality. In response, regulations moved toward better stormwater management and treatment. Florida has been one of the pioneer states on this issue as our current stormwater regulations date back to 1984. The required treatment volume for the most commonly used type of facility is 2.5 cm (one inch) of runoff over the drainage area, which for a typical residential area represents the runoff produced by a 5-cm (2-inch) storm. This, in turn, translates into treating the entire volume generated for 90 percent of storm events.

In spite of these efforts, results of FDEP analyses conducted as part of the TMDL program indicate that many of our streams, lakes, and estuaries are not meeting water quality standards. Stormwater runoff has been identified as the major cause for the impairment conditions. Therefore, efforts are being undertaken to identify new ways to control runoff pollution. One option is to focus on the design features of new development based on a better understanding of the relationship between land uses and water quality. Relevant information regarding that relationship was obtained through a recent project that focused on the evaluation of water quality data collected by the City of Austin, Texas.

Discussion

For about two decades, the City of Austin has implemented a runoff quantity and quality monitoring program that includes numerous sites distributed throughout the city. During design of the monitoring program, the sites were grouped into two categories: a) smaller drainage catchments with uniform land uses; and b) larger drainage basins that comprise multiple land uses and receive discharges from the smaller catchments.

Data analysis results for the smaller catchments indicated that there was no significant correlation between the average **concentrations** of pollutants in the runoff and the amount of impervious cover. However, major increases in the **volume** of runoff were evident with increasing imperviousness. Figure 1 shows the long-term event mean concentration (emc) of total suspended solids (TSS) for numerous drainage catchments as a function of impervious cover. The same pattern is maintained for the other chemical parameters tested. It should be noted that a specific characteristic of the smaller sites is that the drainage systems are composed of grass ditches or drainage pipes that are not subject to erosion even during heavy rainfall events.

In contrast, data for the larger drainage basins show significantly higher pollutant concentrations with increasing impervious cover. Moreover, pollutant concentrations are often higher than those from the small sites that drain to those systems. Figure 1 also shows the long-term emc for total suspended solids for several of the larger drainage basins. Similar to the smaller catchments, the same pattern is maintained for the other chemical parameters tested. Natural channels that can be subject to erosion, sedimentation, and sediment re-suspension processes generally drain these larger basins

An analysis of the data suggests that:

a) Land uses associated with larger impervious cover generally produce a larger mass of stormwater runoff pollutants. However, that mass does not translate into higher pollutant concentrations.

b) The difference in pollutant concentration between the smaller catchments and larger basins is likely caused by erosion or sediment resuspension, which in turn is caused by larger runoff volumes and flows.

c) An efficient way to remove the additional pollutant load would be to **focus on the generated runoff quantity**. One way to address this issue is to implement practices collectively referred to as Low Impact Development (LID).

LID is a site design strategy that seeks to match the pre- and post-development runoff flow and volume characteristics (hydrologic regime) through the use of design techniques that help create a functionally equivalent hydrologic landscape. Hydrologic functions of interception, storage, infiltration, evapo-transpiration, and groundwater recharge, as well as the volume and frequency of discharges, are maintained through the use of integrated and distributed micro-scale techniques. LID tools may include: minimizing impervious cover, disconnecting impervious areas, creating stormwater retention and detention areas within each property, lengthening runoff flow paths, and a host of other small features such as rain gardens and improved infiltration measures. The key is that these tools must be used in an integrated on-site design to achieve the most cost effective and aesthetically desirable result.

The focus of LID is taking measures to control the problem at the source, rather than relying on downstream structures that serve relatively large areas of development. LID strategies, if implemented properly, can provide cost savings by reducing the size of required drainage infrastructure. LID also provides environmental benefits by reducing erosion and minimizing or eliminating habitat destruction in receiving waterbodies due to hydrologic changes. In addition, the reduced size of drainage structures is likely to result in a reduction of maintenance costs to the systems' operators, commonly local governments.

We conducted an informal survey of individuals involved with land development work to investigate the reasons why the development community is reluctant to implement LID strategies. Following are some of the responses:

a) No regulatory incentives exist to implement LID. In fact, there is the perception that regulators look at LID development with certain suspicion, at least during review of drainage calculations, because the designs deviate from the norm.

b) The use of shallow swales, instead of curb-and-gutter designs, often requires leaving a 7 to 10 m (20 to 30 ft) wide dedicated strip of land at the front of a lot, which translates into larger lot sizes and reduced total number of lots in a subdivision.

c) The elimination of curbs around vegetated areas in parking lots may result in creating hazardous conditions, which may translate into issues of liabilities.

d) Although maintenance costs may be reduced for local governments, those costs may be passed on to the property owners or developers. Overall, LID facilities may be more expensive to maintain than traditional facilities.

e) A development that incorporates LID techniques is considered to be less attractive to potential buyers.

Conclusions

Much work appears to be needed to resolve these issues. However, all criteria listed above have two underlying common factors: public perception and costs. There is a need to overcome the perception factor, which will then help eliminate the cost factor. Facilitating implementation of LID practices will require both public education and incorporation of these concepts into land development regulations. Local municipalities and public agencies are beginning to focus in that direction. As part of the education program, it is necessary to convey to the public that there is a cost associated with not meeting water quality standards. The TMDL program, through the BMAP process, may be the mechanism to achieve that objective.

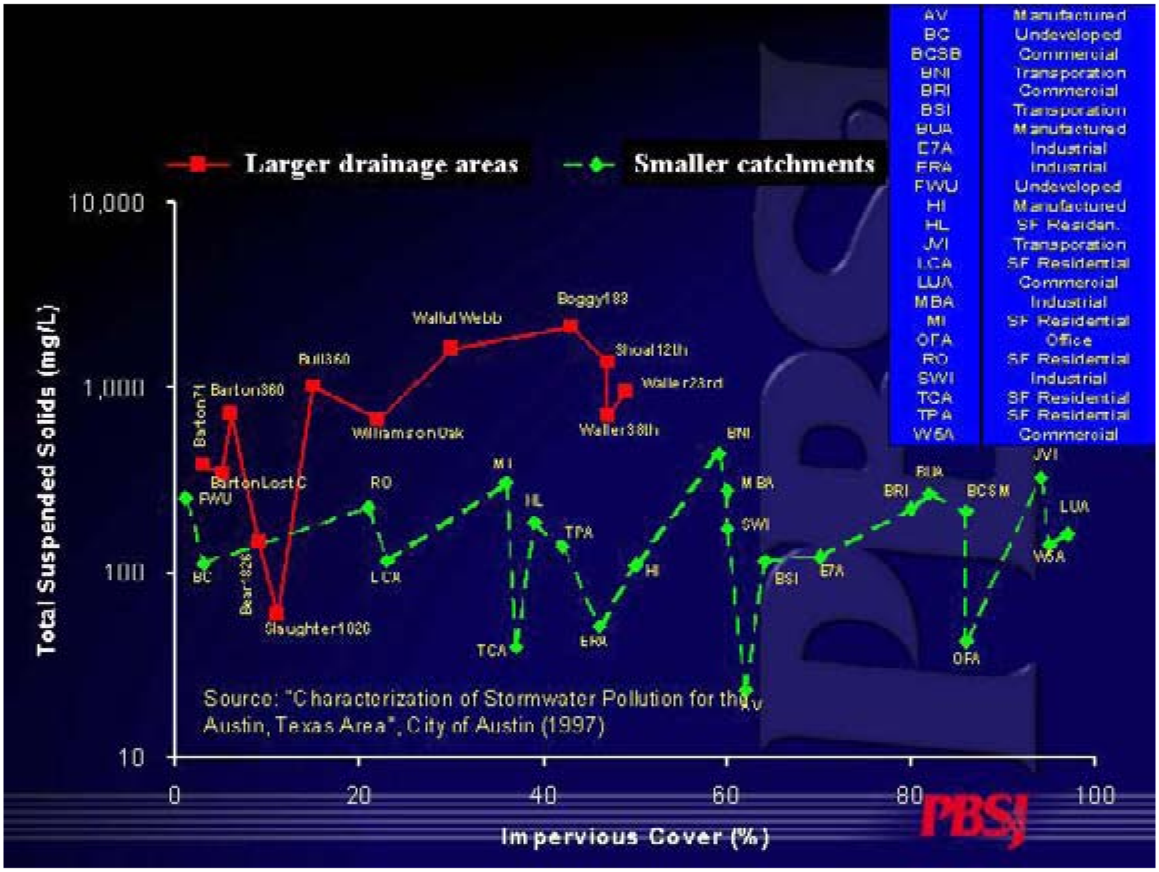

Figure 1. TSS Mean EMCs for Larger Drainage Areas and Smaller Catchment Sites

Case Studies on the Performance of Bioretention Areas in North Carolina

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Abstract

Continued closures of coastal fisheries, increased flooding, and high streambank erosion, has led to increased use of Best Management Practices (BMPs) for stormwater treatment. Because bioretention areas have the ability to fulfill both landscape and water quality needs in a small area, they have received increased recognition as an integral part of Low Impact Development (LID); however, questions on design implications persist. Two paired, field-scale bioretention studies have been conducted in central North Carolina to study removal of phosphorus and nitrogen, hydraulic retention, and effectiveness of an induced saturated zone. Both pairs comprise nominally 5% of their respective watersheds and are planted with trees and shrubs. One conventionally drained cell and one containing an induced saturated zone (previously termed anaerobic zone) of 0.45m (18 in.), were continuously monitored from June, 2002 to Dec, 2004. Groundwater recharge, ET, and the significance of soil media for bioretention have been quantified by comparing outflow from two other conventional cells constructed in winter of 2003 with non-agricultural soils: one lined with plastic, and one unlined. Lower outflow frequency was found for the induced saturated zone design. During outflow events, TP concentrations were significantly lower ($P<.01$) than the conventional cell, although both designs resulted in higher outflow concentrations. Seasonal differences resulted for TP and TN removal and hydrology for all bioretention areas in North Carolina. NO_3 concentrations were reduced by 77% however, TKN and NH_4 concentrations were increased where the induced saturated zone was incorporated. The non-agricultural fill soils resulted in average concentration reductions of 40-53% TP, and 25-60% TN for 10 storms. Exfiltration to groundwater is relatively high regardless of the tight parent soils underlying the bioretention areas.

Introduction

With the increased impervious surfaces due to the rapid development experienced in the past few decades, there has been an increase in stormwater runoff and surface water pollution. This has lead to intensive stormwater flows which scour stream channels and cause downstream flooding, often resulting in the loss of habitat for aquatic life. The rapid transport of rainfall as stormwater allows little time for natural water treatment and recharge of groundwater aquifers. Pollutants which exist in rainfall and those that are picked up from hardened surfaces are carried directly to rivers, estuaries, and other coastal water bodies. The combustion of fossil fuels is the largest anthropogenic source of nitrogen emitted to the atmosphere, and can later be found on hard surfaces and in surface water by wet and dry deposition (Paerl, 1993). Studies have also shown high levels of heavy metals, nitrogen, and phosphorus concentrations in urban stormwater (Barrett *et al*. 1998, Wu *et al*. 1998). These nutrients have lead to contamination of coastal fisheries and the discovery of toxic *Pfisteria*, a fish-killing organism found in nutrient-laden estuarine waters. This has raised awareness to the relation of nutrients arriving from the stormwater network (Burkholder, 2001).

Although historical regulation of point source pollution in the early 1970's with the Clean Water Act and more recently the EPA's National Pollutant Discharge Elimination System (NPDES) have led to a decrease in polluted waters, problems still persist. The continued closings of coastal fisheries and the contamination of beaches have led Federal, State, and Local agencies to regulate non-point source pollution, in addition to point-source dischargers by implementing NPDES Phase I and II rules. Under Phase I and II, operators of municipal separate storm sewer systems (MS4's) must implement stormwater programs which reduce pollutants in post-construction stormwater from new and re-development areas (EPA, 2000a).

BMPs and Low Impact Development (LID) strategies have been developed as measures to control stormwater volume and quality. The "National Menu of Best Management Practices for Stormwater Phase II" (EPA, 2000b), was also published in an effort to aid in the selection of BMPs for MS4 operators along with state publications such as those by NCDENR (1999). Some examples of these BMPs include: stormwater wetlands, permeable pavement, wet detention ponds, infiltration trenches, grassy swales, green-roofs, and within the past ten years in North Carolina, bioretention.

Bioretention has recently become a popular stormwater treatment practice in highly urbanized areas, because of its ability to both improve water quality and meet landscape needs for a small urbanized watershed. Bioretention is typically a small, aesthetically pleasing depression, containing a light fill soil. Storm water is directed from the impervious surfaces to the bioretention area. It is allowed to pond while infiltration occurs eventually drawing down to the soil surface between 24 and 48 hours. Here after the soil column is drained by exfiltration to the groundwater and under-drains where soils prohibit rapid draw down. Once planted with shrubs and trees, the bioretention area can become a natural ecosystem able to retain, infiltrate, and treat large quantities of polluted stormwater. Site-specifically designed for stormwater treatment, bioretention areas can enhance a barren, urban landscape without compromising large areas of land. A typical bioretention area is shown in Figure 1. Research of bioretention areas is important in order to give correct nutrient removal credit for their use, and to further develop their design for increased performance.

To date, limited research has been conducted, and even less on field implementation, thus leading to designs based on minimal data. This study is an effort to add to the current information on bioretention and attempt to enhance the design of bioretention areas to maximize pollutant removal and minimize surface area. Four field-scale bioretention areas were monitored in central North Carolina in order to evaluate pollutant load removal of key pollutants, runoff volume reduction, implications of adding an induced saturated zone (previously termed anaerobic drainage configuration), and to assess the importance of correct fill soil usage.

Bioretention Background

Although limited research has been conducted on bioretention areas, it has been found that bioretention can effectively reduce several pollutants in stormwater. Lab studies conducted by Davis *et al.* at the University of Maryland, have shown greater than 90% reduction of heavy metals (copper, lead, zinc) within the top 20 cm of the soil media, and 60-80% reduction in Total Kheldal Nitrogen (TKN), ammonium and total phosphorus; however, minimal reductions in nitrate and ammonium were observed (Davis *et al*. 2001, 2003, Hunho *et al.* 2003).

Some alterations to the conventional design have been studied by researchers at North Carolina State University and the University of Maryland. Studies by Hunho *et al.* were directed at increasing nitrate reduction by introducing an electron donor source to an 18 cm deep anaerobic zone at the bottom of a 36 cm deep box column (2003). A synthetic runoff was applied with concentrations varying between 2.0 mg/L and 8.0 mg/L of nitrate, at different flow rates. It was found that nitrate/nitrite was reduced 70%-80%, but increased TKN and ammonium in the effluent. It was suggested that ammonification occurred due to a low carbon/nitrogen ratio, resulting in the higher concentrations (Hunho *et al.,* 2003).

Figure 1. Bioretention cell in Greensboro, NC.(2004)

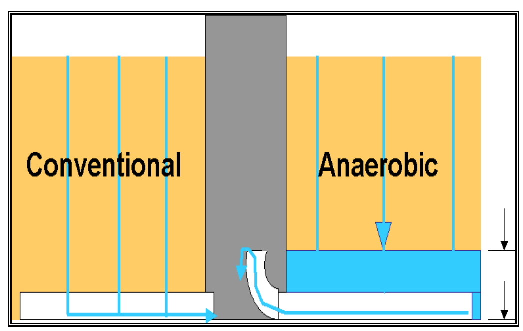


Figure 2. Design with induced saturated zone. (Hunt, 2003)

By adding an anaerobic process to a bioretention area, nitrate can potentially be converted to nitrogen gas by denitrification, further reducing the nitrogen component in runoff. Hunt *et al*. (2003) conducted one of the first field studies at North Carolina State University, and reported similar results for a conventionally drained bioretention cell as the lab studies by Davis *et al.* (2001, 2003). Hunt's study was also directed at analyzing the effectiveness of an induced saturated zone to increase reduction of nitrogen.

Two cells with a soil media depth of 1.2 m (4 feet) were constructed at a shopping center in Greensboro, NC during 2000 and 2001 and have been monitored continuously since June 2002. The bioretention areas each make up 5% of their respective 0.2 ha (0.5 acre) watersheds. An organic sandy soil media, with a hydraulic conductivity of 0.11 mm/s (15 in/hr), was prepared and filled in each cell. The first cell (G-1) was constructed with a 1.5 m (18 in.) induced saturated zone (formerly termed anaerobic zone) where an upturned drain pipe forces water to saturate the bottom portion of the soil media. The second cell (G-2), was not forced to saturate in the bottom portion of the soil media. Figure 2. shows a cross sectional diagram of a conventional cell and one containing induced saturation. Further research is being conducted using these cells in the comparison of the hydraulics of the two design types, results of which are included below. TN load removal of 40% were observed for both cells, while TN load was reduced (Hunt *et al.* 2003).

It was found that a TP load increase of 240% was found in the conventional cell, where a site in Chapel Hill, NC also studied by Hunt *et al.*, resulted in 65% reduction. Hunt supposed that the increased removal of TP in Chapel Hill may have been due to the lower soil-test P in the fill soil (Hunt 2003). This means there was a large quantity of phosphorus bound by the fill-soil in Greensboro which could potentially be lost due to the inevitable saturation in parts of the bioretention cell. The drawbacks from the use of high soil-test P in fill soils is currently being investigated with the inclusion of two new cells finished Spring, 2004 in North Carolina, results of which are discussed below.

In Hunt *et al's* (2003) study it was also noticed that including an induced saturated zone could potentially reduce outflow through the underdrains, more so than the conventionally drained cell. The study was not conclusive and further research on the reduction of flows by a bioretention area continued. The Greensboro site continued to be monitored through December, 2004 results of which are later discussed.

Research Goals

Given the many questions still prevalent regarding bioretention areas, research has continued at N.C. State. The goals of this research will be to:

1. verify the need for a low P-Index fill soil,
2. investigate pollutant removal of a conventional cell and one containing an induced saturated zone,
3. quantify bioretention hydrology: (exfiltration, evapotranspiration), and
4. investigate bioretention hydrology with inclusion of an induced saturated zone.

Monitoring of two newly constructed cells in Louisburg, NC will address goals 1 and 3, whereas continuation of the current study sites in Greensboro, NC address goals 2 and 4. A soil column study will also be conducted in Spring, 2005 to pinpoint the ideal soil to be used in a bioretention system where phosphorus is the limiting pollutant. By filling these gaps in information, designers may eventually be able to prescribe a specific bioretention design to meet their specific pollutant or volume reduction needs while being given correct credit.

Methodology

Greensboro Site

Each cell in Greensboro has a 10 mm (4 inch) drainage layer, placed at the bottom of the cell, consisting of washed P-57 stone enveloping a pair of drain pipes. The first cell (G-1), placed in the front of a shopping center, was constructed with a 0.45 m (18 in) induced saturated zone. The second cell (G-2) is conventionally drained with no induced saturated zone. Both cells comprise 5% of their watersheds consisting of a small shopping center and parking lot. Both cells were planted with trees and shrubs, and 7-10cm of hardwood mulch has been maintained. Both cells here were also the subject of the previous study by Hunt *et al.* Monitoring of the cells is later discussed.

Louisburg Site

Two conventionally drained cells with a nominal treatment depth of 1 m (36in.) were completed March, 2004 in Louisburg, NC alongside the Tar River, recently declared Nutrient Sensitive Waters (NSW) by the State of North Carolina. The cells both comprise 4.5 % of their respective watersheds. One cell has been lined with plastic, and both cells planted proportionally. Soils in the region are generally tight and high in clay minerals. Gleaning from Hunt *et al's* (2003) results obtained in Greensboro and Chapel Hill, soil media with a very low P-index containing approximately 90% sand and 8% clay was used for fill. Peat was added at a rate of ¼ inch per square foot. Figures 3. and 4. show the two cells after excavation and placement of the liner. Both cells were designed to capture and contain a minimum 29 mm (1.2 in.) rainfall event. The sites were constructed as would a typical installation with minor adjustments to allow for monitoring. These cells were also planted with trees and shrubs and topped with 7-10cm. of double-shredded, hardwood mulch. By measuring flow leaving the lined cell through the underdrains and subtracting the amount entering as runoff, a value of evapotranspiration results. The excess loss of water in the un-lined cell is that lost to exfiltration.

Figure 3. Lined Cell, Louisburg, NC. (Nov 2003)

Figure 4.Un-lined Cell, Louisburg, NC. (Nov 2003)

Monitoring

Five minute rainfall data was collected for each cell, and runoff entering the cell was computed using an SCS Curve Number of 98 (100% imperviousness) in Greensboro, and 95 in Louisburg. Rainfall events were separated where rainfall stopped for more than six hours. Outflow was continuously monitored with weir boxes attached to the outlet under-drain from each cell by automated samplers (Sigma 900 Max. and ISCO 6712). At the Louisburg site, surface ponding depths were also monitored in order to calculate the volume of water which by-passed the soil system to the overflow drop box. Mixing of overflow and underdrain flow was prohibited. Flow weighted composite samples, by rainfall for inflow samples and by outflow for outflow samples, were taken with automated samplers from troughs at the parking lot edge and from the weir boxes at the outlets. These were collected within 24 hours of the storm, acidified with 36 molar H_2SO_4 and then taken to Soil Sciences Analytical Services in the Soil Science Department at North Carolina State University for water quality analysis. Pollutants analyzed were TKN, NO_3, NH_4, TP, PO_4, Zn, Cu, and Fe. Given both volume of flow and concentration of flow, inflow and outflow pollutant loads are calculated. Both pairs of cells were monitored in the same fashion and results were analyzed for the growing and non-growing seasons.

Results And Discusion

Greensboro Site

Hydrologic Results

Seasonal rainfall and flow data results from the two cells in Greensboro are found in Table 2. Runoff which would have resulted from the rainfall is considerably reduced by each bioretention area. Biologic activity and evaporation potential are lower in late fall and winter months leading to less uptake of water by the bioretention area. During Spring and Summer, a greater demand for water by evapotranspiration creates a larger uptake of water. This will also affect pollutant load removal from runoff due to the lower percent reduction of rainfall to outflow.

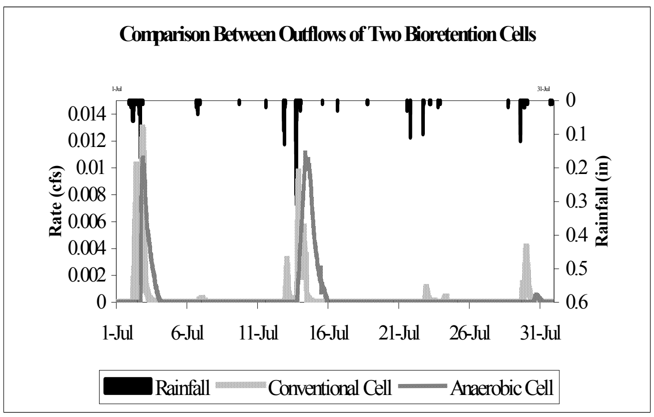
Table 2. Rainfall and outflow volumes for two cells in Greensboro starting July, 2003.

Season	Rainfall (inches)	G1 outflow (cubic feet)	G1 outflow (inches)	G2 outflow (cubic feet)	G2 outflow (inches)
Summer (Jul.-Sept.)*	23.71	7012.73	3.86	1652.63	0.91
Fall (Oct.- Dec.)	6.95	785.40	0.43	317.10	0.17
Winter (Jan.-Feb.)	Limited Data				
Spring (Mar.-May)	6.5	484.68	0.27	435.02	0.24

*Water lost to overtopping not taken into account(3-4 events)

It is expected that for events in rapid succession, lower pollutant removal efficiency will be found than for events with delayed succession. Preliminary flow data show a longer duration and less frequency of outflow from G-1 than G-2. Because both cells have equal surface area to watershed area proportions, and both have the same rainfall and similar imperviousness within their respective watersheds, the inflow volumes are assumed to be essentially equal. Since a bioretention cell is limited by its hydraulic conductivity, no significant difference in peak flow is expected between the two cells.

Figure 5 shows a representation of the outflow for each bioretention cell during July, 2003, when 8.41 in of rain fell. Essentially the anaerobic cell flowed twice where the conventional cell flowed six times. It can be seen that for smaller events, the anaerobic cell does not produce outflow; whereas, the conventional cell does. In contrast, larger storms and those in rapid succession cause the anaerobic zone of G-1 to be filled and an equal, if not larger, volume of outflow to occur. It can also be seen that the anaerobic cell is producing longer flow durations. A second figure (6) is presented displaying the same phenomenon occurring in Spring 2004. A 23 mm (0.89 in) event triggered outflow from the conventionally drained cell, but no outflow occurred from the cell with anaerobic drainage configuration. Outflow does not occur until the storm is well underway, highlighting a bioretention cell's ability to dampen peak flow.



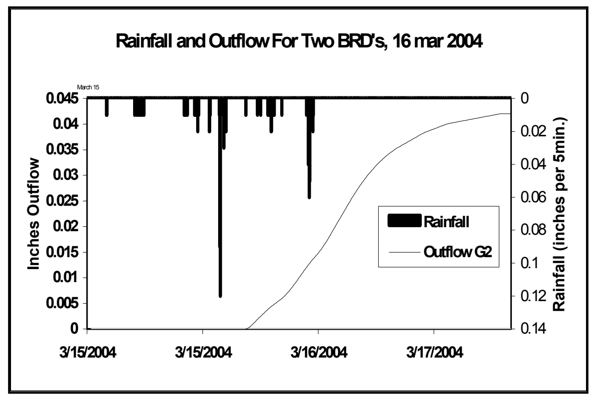


Figure 5. Outflow Hydrographs for cells G-1 and G-2 (July 2003).
Figure 6. Cumulative Rainfall and Flow Event no outflow from G-1.

Water Quality Results

Median pollutant concentrations from the Greensboro cells are found in Table 3. Because bioretention decreases flow volumes, a decrease in pollutant loads leaving the cell is likely. It can be seen that a decrease in nitrate concentration is evident for both cells. It was observed that for other forms of nitrogen, namely NH_4 and TKN, each cell has increased concentrations in outflow over inflow. It can also be noticed that NH_4 concentrations are higher in outflow from G-1 than for G-2. This phenomenon was also noted in Hunho *et al's* (2003) results due to anaerobic digestion.

Table 3. Median concentrations entering and leaving two Bioretention cells in Greensboro, NC. (concentrations from composite samples taken July- Dec, 2003)

		G2-Conventionally Drained Cell		G1-Anaerobically Drained Cell	
Analyte		**Inflows**	**Outflow**	**Inflow**	**Outflow**
mg N/L	**NO3**	0.24	0.13	0.22	0.05
mg N/L	**NH4**	0.25	0.56	0.20	2.55
mg N/L	**TKN**	0.40	4.30	1.20	4.15

Figure 7 shows the outflow concentrations of total phosphorus from both bioretention cells from September 2002 through December 2004. Of the 21 events where outflow concentrations were analyzed from both cells, in only 1 were the concentrations from the conventionally drained cell lower than those of the anaerobic cell configuration. The conventional cell has significantly higher effluent concentrations of TP ($P<0.01$) than the anaerobic cell. It was hypothesized that the anaerobic configuration (G-1) provides a sump for sediment-borne phosphorus; whereas, this phosphorus could “flow through” a conventional configuration (G-2).

Total Nitrogen concentrations from both cells have not yet resulted in a significant difference in annual concentration averages ($P > .1$). However, it has been noticed that inclusion of the induced saturated zone may potentially vary from the conventional cell by season in terms of both hydrology and pollutant removal. Generally, outflow concentrations for both TN and TP leaving the conventional cell and volume of outflow tend to increase for the non-growing season Table 4. The low intensity storms common in winter months can be fully contained by the induced saturated zone, placing the outflow volume lower than that of the conventional cell. This would in turn create higher load removal by the anaerobic cell and potentially create a significant difference by season.

Table 4. Seasonal Averages of Total Phosphorus and Total Nitrogen for Data From Sep, 2002 thru Dec, 2004

Total Phosphorus	**G1in**	**G1out**	**G2in**	**G2out**
Growing Season	0.22	0.65	0.24	1.73
Non-Growing Season	0.35	0.43	0.09	6.22
Total Nitrogen				
Growing Season	2.87	5.49	1.64	5.25
Non-Growing Season	2.96	4.14	1.32	10.10

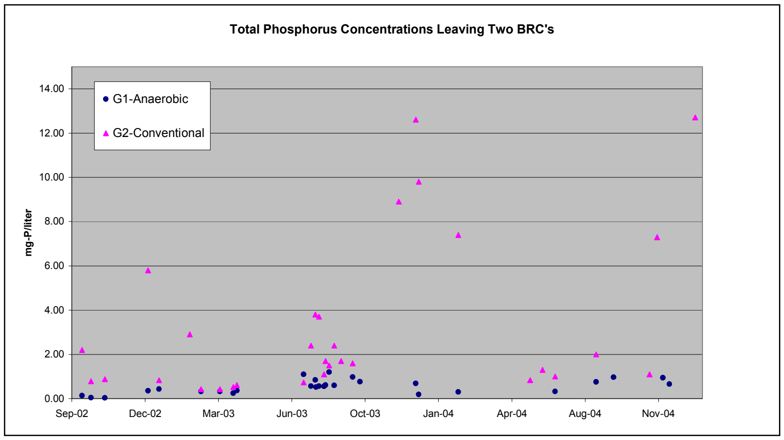


Figure 7. TP concentrations leaving G-1 and G-2 from Sep, 02 to Dec, 04

Louisburg Site

Hydrologic Results

Preliminary results were analyzed for data from June, 2004 to Dec, 2004. Differences in percent volume reduction were noticed between the growing and non-growing seasons. The growing season was based on the Franklin County Soil Survey, and runs from mid April to mid October. Flow results showing total rainfall and outflow for the two cells from June 15 to July 31, 2004 and November, 2004 are summarized in Table 5. One can notice that rainfall was reduced by lower percentages for the November period than for the June/July period. This is concurrent with the fact that evapotranspiration is lower in the winter months than in the summer months and less water is removed by the bioretention system.

Figure 8 shows cumulative rainfall and outflow from the two cells. Each step in the three lines corresponds to a rainfall event. This shows the intensity of volume reduction by the two cells, and furthermore the lined cell converting more rainfall to outflow than the un-lined cell. Where the two cells differ, is the quantity lost to exfiltration in the un-lined cell. Preliminary analysis resulted in a difference of 2.54 cm (1 in.) presumably lost by the un-lined cell to exfiltration.

Table 5. Total Inches of Rainfall and Outflow for June 15 to July 31 and Nov. 2004

	June-July	November
Rainfall	6.07	4.22
Lined Cell Outflow	1.43	2.54
Percent reduction	0.76	0.40
Unlined Cell Outflow	0.44	2.37
Percent reduction	0.93	0.44

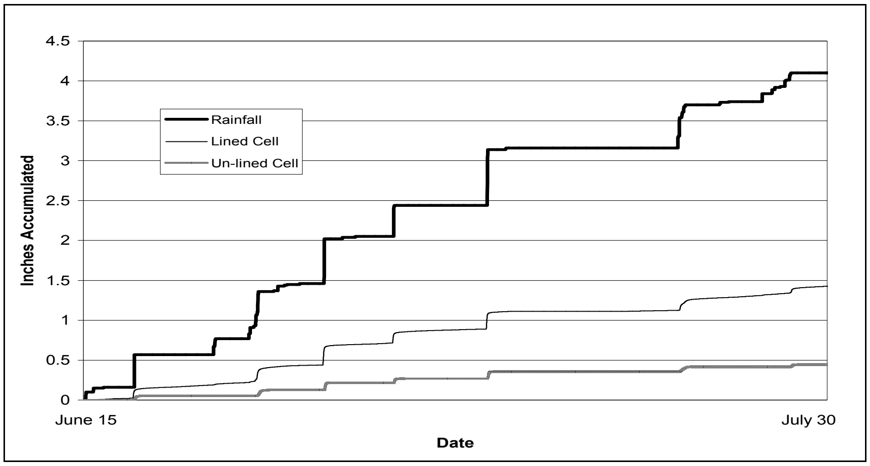


Figure 8. Cumulative Inches of Rainfall and Outflow for both cells June15 –July 31, 2004.

This data can be used to estimate a water balance for bioretention systems. Further analysis is to be conducted on the quantity lost to overflow by investigation of surface ponding data. This will fill all components of volume loss by a bioretention system, and design based on volume reduction can be done.

Water Quality Results

Water quality data has been analyzed for 7 complete sets for storms during May to September in Louisburg, NC. Contrasting to what was expected, slight differences between the two cells have been observed Table 4. The difference in TN reduction between the cells is largely due to the difference in NO_3 and TKN. It has been noticed that the un-lined cell flows much longer than the unlined cell, sometimes flowing, although at low intensities, continuously for weeks at a time. This would give reason to believe the soils have become reduced within the lower portion of the cell, thus allowing for nitrate reduction to occur. The increase in iron concentration, and low turbidity of the outflow confirms the reduction of soils. Because the upper portion of the cell is not reduced, NH_4 can still be transformed to NO_3 allowing for its reduction as well.

It is noticed that TP reduction is slightly greater for the lined cell than the un-lined. This could potentially be explained by one or both the lack of PO_4 in inflow of the lined cell or the loss of available phosphorus adsorption sites due to reduction of iron. Because phosphorus is likely to attach to sites in the soil containing iron, losing iron will presumably adsorb less phosphorus. This cannot be determined due to the low inflow concentrations found for this cell.

In either case, a reduction in outflow concentrations of TP and TN is found. Given the cells also decrease outflow volumes, load reduction is expected to be high. When analysis is completed, a significant load reduction is expected by both cells. This would give good reason to prescribe a low P-Index fill soil in bioretention areas, and especially when located in nutrient sensitive waters. concentrations entering this cell.

Table 4. Average concentrations at Louisburg from 9 events during Summer 2004.

Analyte		*Un-lined Cell* Inflow (7 samples)	Outflow (8 samples)	% reduction	*Lined Cell* Inflow (8 samples)	Outflow (9 samples)	% reduction
TN*	**Mg /L**	2.11	1.55	27	2.76	1.11	60
NO3	**Mg N/L**	0.39	0.29	26	0.52	0.14	73
NH4	**Mg N/L**	0.25	0.06	77	0.31	0.05	84
TKN	**Mg N/L**	1.72	1.26	27	2.24	0.97	57
PO4	**Mg P/L**	0.18	0.08	54	0.02	0.03	NA+
TP	**Mg P/L**	0.30	0.18	39	0.23	0.11	53
Fe	**Mg /L**	0.43	0.50	-18	0.20	1.48	-642

*TN is the sum of TKN and NO3
+Inflow and outflow conc. Reaching det. Limit

Some complications were encountered during the investigation of this site. Since the fill soil was in the very low P-Index category (1to 2), plants within the bioretention area lacked vigor. Plants require phosphorus among other nutrients to survive, and generally soils with P-Indicies greater than 30 are used where plant survival is of sole concern. Given this drawback, an ideal P-Index level needs to be developed. Thus the focus of a third study which will take place in Spring, 2005 to investigate phosphorus removal by soils at differing P-Index levels.

Summary and Conclusions

Bioretention areas are a developing technology used to treat urban stormwater. By continuing field research of bioretention areas, design standards for implementation into a stormwater management plan can be further refined. Findings from this study, along with those from previous research will influence future bioretention design by defining and quantifying:

1) benefits of an induced anaerobic zone in terms of hydrologic attenuation and pollutant removal,
2) nutrient reduction when low P-index soils are used, and
3) a water balance (ET, outflow, exfiltration) for use in sizing standards.

Data collected in Greensboro, NC from 2003 and 2004 show that the anaerobic cell delays the outflow hydrograph and reduces the flow frequency over that of a conventionally drained cell. Regardless of drainage configuration, bioretention areas considerably reduce runoff volumes and in many cases, pollutant loads. The saturated zone could also function as an underground storage area giving more contact time inside the cell, and nutrients are given more time to be treated. When considering pollutant removal, the anaerobic drainage configuration led to significantly lower ($P<.01$) Total Phosphorus concentrations in outflow than the conventional cell.

Soils used in a bioretention area are an important part of the design. Data presented here shows a strong correlation between TP reduction rates and the P-Index of the fill soil. It is recommended upon investigation of bioretention areas in North Carolina that non-agricultural soils containing a low P-Index be used for fill, especially in areas near nutrient sensitive waters.

Given the seasonality of bioretention functions, and data presented here, one could propose that a pollutant removal rate based on an annual average concentration would insufficiently describe a bioretention area's ability to remove key pollutants. Further research is recommended, and further analysis of existing data is proposed concerning the seasonality of bioretention areas.

Acknowledgements

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A Monitoring Field Study of Permeable Pavement Sites in North Carolina

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Abstract

Asphalt surfaces have greatly increased the amount of pollutant-carrying runoff entering surface waters. To counteract this, permeable pavement can be installed to allow water to infiltrate, thus reducing runoff and maybe acting as a filter. Three permeable interlocking concrete pavements (PICP) sites were monitored across North Carolina in Cary, Goldsboro, and Swansboro. The Cary site was located in clay soil and flowrates and samples of exfiltrate and rainfall over 10 months were collected and analyzed for pollutant concentrations. The Goldsboro site was constructed to compare the water quality of asphalt runoff to exfiltrate of permeable pavement. The site was located on a sandy soil and samples were analyzed for pollutants over a span of 18 months. The Swansboro site was constructed and instrumented to monitor runoff flow and rainfall rates and collect exfiltrate and runoff samples from the permeable pavement lot over 10 months. The site was located on a very loose sandy soil and experienced no runoff. PICP exfiltrate from the Goldsboro site had significantly lower concentrations of Total Phosphorus and Zinc compared to asphalt runoff. Total Nitrogen (TN) concentrations were close to significantly lower in exfiltrate, but did show an increasing trend of TN removal.

Introduction

Permeable pavement is an alternative to traditional asphalt and concrete surfaces. It allows stormwater to infiltrate into either a storage basin below the pavement or exfiltrate to the soil and ultimately recharge the water table, while also potentially removing pollutants (EPA, 1999). Urbanization has had a detrimental effect on our surface waters systems. Increased runoff rates from paved surfaces have increased peak flow, time to peak, runoff volumes, and pollutant loads through stream channels causing erosion and stream bank instability along with overland erosion (NRCS, 1986). Parking lot runoff also carries pollutants, such as sediments, nutrients, and heavy metals, into surface waters. In an effort to reduce these effects of urbanization, several municipalities in North Carolina established regulations that limit the amount of impervious surfaces (Bennett, 2003). Permeable pavement may be a solution; reducing both runoff and pollutants. As a result, the use of permeable pavement is poised to grow.

North Carolina has implemented a stormwater credit system for developed sites to manage onsite runoff. Several best management practices (BMPs) were given credits for pollutant reduction, sediment reduction, and peak flow detention, but permeable pavement was not included. Permeable pavement use is only allowed as a BMP under the "innovative BMP" classification. Innovative BMPs however must be monitored on an individual basis to assess their performance (Bennett, 2003), but few landowners have been willing to assume the cost of the required monitoring.

Recent studies have found positive results using permeable pavement with respect to both runoff reduction and water quality improvement. The use of permeable pavement, instead of traditional asphalt, has been shown to decrease surface runoff and lower peak discharge significantly (Pratt 1995; Booth, 1996; Hunt et al., 2001). Permeable pavements have also been shown to act as a filter of such pollutants as lead and automotive oil (Pratt, 1995; Brattebo and Booth, 2003).

The goals of this study were as follows: (1) develop an SCS curve number and rational coefficient for two field sites, (2) monitor pollutant levels of PICP exfiltrate and PICP runoff, and (4) offer basic siting guidelines based upon these results.

Study Sites

Three PICP sites were instrumented for water quality testing, two of which were also instrumented to measure water quantity (Fig. 1). The western-most site, in Cary, was

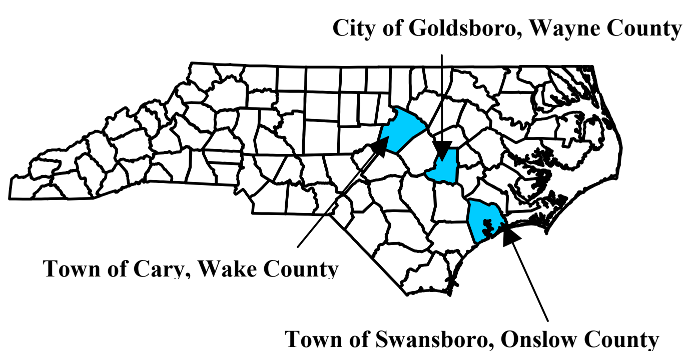

Figure 1. North Carolina map illustrating research site locations.

instrumented to collect rainfall and measure rainfall rates. PICP exfiltrate was collected for water quality analysis and exfiltrate flow rates from under drains were recorded. About 70 miles east of the Cary site, the Goldsboro site was constructed to collect asphalt runoff and exfiltrate for analysis and comparison. The site is a divided parking lot, where the parking stalls are PICP and the drive paths are asphalt. The eastern-most site, Swansboro, is a public parking lot. The site was constructed with PICP to collect runoff and exfiltrate samples for water quality and monitor rainfall intensities and runoff rates. Each monitoring site is located in a different geographic region: the piedmont, coastal plain and coast, respectively.

Cary

The PICP driveway in Cary (Fig. 2a) was constructed in Fall of 2003 with a surface area of 480 m^2. The pavers are 8 cm (3 in.) thick, and were laid over a compacted layer of at least 25 cm (10 in.) of washed No. 57 stone (ASTM D448). The storage basin under the pavers is divided into two separately drained basins. Two 10 cm (4 in.) drain pipes provide outlets for exfiltrate. Precipitation was measured by an ISCO rain gauge. The storage basin was lined with an impermeable geo-textile to prevent seepage into the soil, so any water not draining from the underdrains was assumed to be runoff.

Goldsboro

The parking lot in Goldsboro, (Fig. 2b) was constructed in the summer of 2001. An 8 cm (3 in.) drainage pipe was installed under a section of the PICP during construction to collect exfiltrate samples. The pipe drains about 120 m^2 of permeable pavers. The 8 cm thick pavers overlay 8 cm of No. 78 stone, which are, in turn, over 20 cm of washed No. 57 stone (ASTM D448). To capture asphalt runoff, the drive path was graded so that runoff would flow towards a metal channel, where it could be collected by a Sigma 900™ automated sampler.

Swansboro

The Swansboro parking lot (Fig. 2c) was constructed in the fall of 2003 with an area of 975 m^2. Pavers, 8 cm (3 in.) thick, were overlaid 8 cm (3 in.) of No. 78 stone, which overlaid 20 cm (8 in.) of washed No. 57 stone (ASTM D884). An 8 cm (3 in.) drain pipe was installed during construction to collect exfiltrate for water quality analysis. The site was slightly sloped so that runoff would flow to a concrete swale, which emptied into a weir box to measure runoff rates.

Figures 2a, 2b & 2c. Photographs of Cary, Goldsboro, and Swansboro PICP sites.

Materials & Methods

Both the Cary and Swansboro sites were equipped with ISCO 6712 automatic samplers for flow monitoring and sample collection. At Cary, the two exfiltrate drainage pipes each flowed into a weir box with a baffle and a 90^o V-notch weir. One box was equipped with a pressure transducer to record the water level every five minutes for flow measurement. The other weir box was equipped with an ISCO 6712 with a 730 Flow Bubbler Module for flow measurement and sample collection. At the Swansboro site, the runoff weir box was also intended to collect samples and monitor flow. However, since no runoff occurred, no runoff samples or measurements were collected or recorded. ISCO Rain gauges were also installed at the Cary and Swansboro sites. By quantifying the volume of water entering the Cary site (rainfall), and measuring exfiltrate rate, the volume of runoff could be determined. In Swansboro, the runoff volume was known, and the exfiltrate volume was calculated. Each rain gauge had the same accuracy of 0.025 mm (0.01 in.) of rainfall per tip.

For water quality evaluation, the Cary and Swansboro ISCO samplers collected 200 ml of exfiltrate or runoff every 5 minutes while the water level was higher than the height of the weir invert. At the Cary site, rainfall was captured using a plastic motor oil catch basin. At the Goldsboro site, runoff was collected where the curb opens into a grassy swale. A Sigma 900™ suctioned 75 ml (0.03 oz.) of runoff every 20 minutes from a metal channel installed between the asphalt and swale. Exfiltrate for both sites was collected by opening a hand valve at the end of a drain pipe running under a the PICP cell. For sampling, the hand valve would initially be opened to flush any residual exfiltrate from the previous storm event. After the initial flush, the valve was closed and then reopened to collect a sample into either a 250 or 500 ml (8.5 or 17 oz.) bottle. After the sample was collected the valve remained open to allow additional exfiltrate to drain out. Once the pipe was empty the valve was closed again for the next storm.

All samples were either frozen or acidified with H_2SO_4 within 24 hrs. One drop of sulfuric acid was added for every 50 ml (1.7 oz.) of sample. All samples from the three monitoring sites were analyzed for concentrations of: Total Kjeldahl Nitrogen in Water (TKN) [EPA 351.2], Nitrate-Nitrite in Water (NO_{3+2}-N) [EPA 353.2], Total Nitrogen (TN), and Total Phosphorus (TP) [EPA 365.4]. The initial eight sets of samples from Goldsboro were also analyzed for Copper (Cu) [EPA 200.8] and Zinc (Zn) [EPA 200.8] concentrations, while the final six storms were analyzed for Ammonia in Water (NH_4-N) [EPA 350.1] and Phosphate (PO_4) [EPA 365.1]. Either the Soil Science Analytical Lab at North Carolina State University or Tritest of Raleigh performed analysis. All runoff and exfiltrate from Goldsboro and exfiltrate from Cary were also analyzed for total suspended solids (TSS) except for storm 13 and storm 15 for Goldsboro and Cary, respectively. TSS samples were analyzed at the NCSU Water Quality Group lab [EPA 160.2]. For statistical analysis, pollutant concentrations found less than the minimum detectable level, were set to be half of the minimum detectable level.

Results: Hydrologic Performance

Runoff and rainfall data from the Swansboro site were collected for ten consecutive months, from March until October of 2004. Exfiltrate and rainfall data were analyzed at the Cary site for only two months, July and August of 2004, due to many technical issues.

During the entire monitoring period at the Swansboro site, from March 1 until December 31, 107 cm (42 in.) of rainfall fell and no runoff occurred. The largest storm recorded was 8.8 cm (3.5 in.). Four storms occurred with over 5 cm (2 in.) rainfall totals. An SCS curve number of 35 (the minimum) was determined by back calculating through the SCS runoff curve number method (NRCS, 1986) using rainfall depth totals ranging from 4.3 cm – 7.7 cm (1.7- 3 in.). A minimum rational coefficient of 0 was determined by back calculating the Rational Method (APWA, 1981). During the summer of 2004, a single ring infiltration test was conducted for a study by the authors and found extremely high surface infiltration rates, 2000 cm/h (800 in./h) mean surface infiltration rate (Bean et. al., 2004). The combination of being located on very permeable soil, having a large storage volume and having a surface free of fines was the explanation for no runoff from the site. Table 1 shows results from three storms that occurred during July and August at the Cary site. In 2003, the site had a surface infiltration rate of 230 cm/h (90 in./h) (Bean et al., 2004). The Cary PICP attenuated the runoff in three ways (1) Runoff Volume (66% of water entering the site left through exfiltration, leaving 34% to runoff), (2) Peak Runoff Rate (reduced by 67%) and, (3) Peak Outflow Delay (78 minutes). It should be noted that only three storms had sufficient data to be fully analyzed. However, the data is fairly consistent for three storms, each separated by approximately seven days each. More data needs to be collected from this site.

Date	Rainfall Totals (cm)	Volume Attenuation %	Peak Attenuation %	Delay to Peak (hrs)
7/22/2004	1.5	88	81	1.3
7/29/2004	1.6	53	44	1.5
8/5/2004	1.7	57	75	1.1
Mean	**1.6**	**66**	**67**	**1.3**

Table 1. Hydrologic summary of results from Cary PICP site.

Results: Water Quality

Water quality data was collected for 14 storms from the Goldsboro site from June, 2003, until December, 2004. Table 2 summarizes the mean pollutant concentrations and factors of significance. Data was analyzed using paired t-tests to determine p-values (SAS, 2003). It was hypothesized that concentrations of these pollutants from exfiltrate samples would be significantly (p-value $\leq$ 0.05) lower than asphalt runoff concentrations. Table 2 shows that exfiltrate concentrations of Zn, NH_4-N, TP, and TKN were significantly lower than concentrations of the same pollutants in the runoff. Cu had substantially lower exfiltrate concentrations than runoff concentrations, but not significantly. TN, TSS

Pollutant Analysis	Mean Runoff	Mean Exfiltrate	p-value	Storms
Zn by ICP/MS-Water mg Zn/l	*0.067*	0.008	**0.0001**	1-8
NH_4-N/Water mg N/l	*0.35*	0.05	**0.0194**	9-14
TP/Waters mg P/l	*0.20*	0.07	**0.0240**	1-14
TKN/Water mg N/l	*1.22*	0.55	**0.0426**	1-14
Cu/MS-Water mg Cu/l	*0.016*	0.006	*0.0845*	1-8
TN Calculation mg N/l	*1.52*	0.98	0.1106	1-14
TSS mg/l	*43.8*	12.4	0.1371	1-12,14
PO_4 mg P/l	*0.06*	0.03	0.2031	9-14
NO_{3+2}-N/Water mg N/l	0.30	*0.44*	0.2255	1-14

Table 2. Pollutant summary for Goldsboro site.

and PO_4 were lower in concentration in the exfiltrate, but not significantly. However, TN (Fig. 3) shows a possible removal trend that, with more sampling, could become significant. NO_{3+2}-N was the only pollutant to have higher concentrations in the exfiltrate than the runoff.

For each of the six exfiltrate samples (9-14) analyzed for NH4-N, concentrations were less than the minimum detectable level. One explanation for NH_4-N removal and the increase in NO_{3+2}-N could be that NH_4-N was converted into NO_3-N through bacterial nitrification. However, it is unknown whether NH_4-N was converted or filtered.

Figure 3 shows concentrations of TN for each storm. Initially, for storms 1-6, there seems to be no trend developing due to variable concentrations. However for storms 7-14, except 9, exfiltrate concentrations were less than runoff concentrations. This seems to

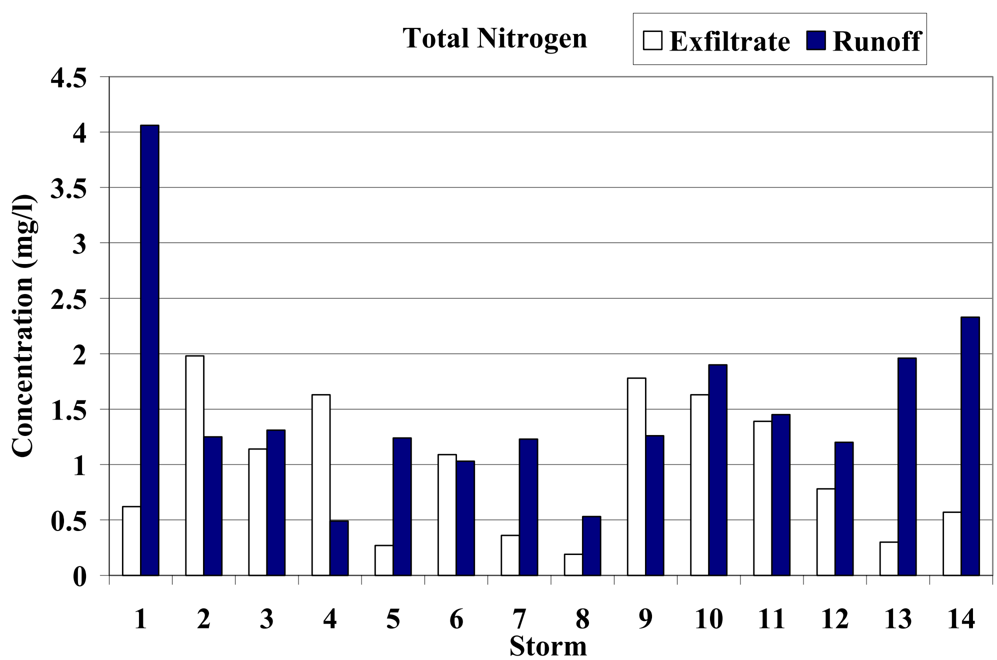

Figure 3. Total Nitrogen Concentrations for Goldsboro site.

indicate a possible developing trend. With a p-value of 0.06, significance may be shown with more samples. Since exfiltrate TN levels were substantially lower than runoff TN levels, TN either stayed in the subbase or was converted and escaped as N_2 gas. It is possible that denitrification of NO_3-N could be occurring at higher rates than nitrification of TKN and NH_4-N. This would result in more nitrogen leaving as N_2 gas, and thus reducing the amount of TN leaving the system in the exfiltrate.

Figure 4, shows TP concentrations in runoff and exfiltrate. For all but two storms (3 and 10), runoff concentrations were greater than exfiltrate concentrations. Exfiltrate concentrations were greater than 0.1 mg/l only twice. Total phosphorus concentrations were significantly lower in the PICP exfiltrate than the asphalt runoff. This suggests that the PICP system significantly ($p \leq 0.05$) lowered the concentration of TP. This could be a result of phosphorus becoming bound to the storage basin material. Another possible scenario is that phosphorus is infiltrating while bound to sediments, and even though sediment is not significantly reduced, exfiltrate concentrations of TSS are substantially less than runoff. Therefore, the phosphorus could have been removed through sediment filtration, along with possible binding.

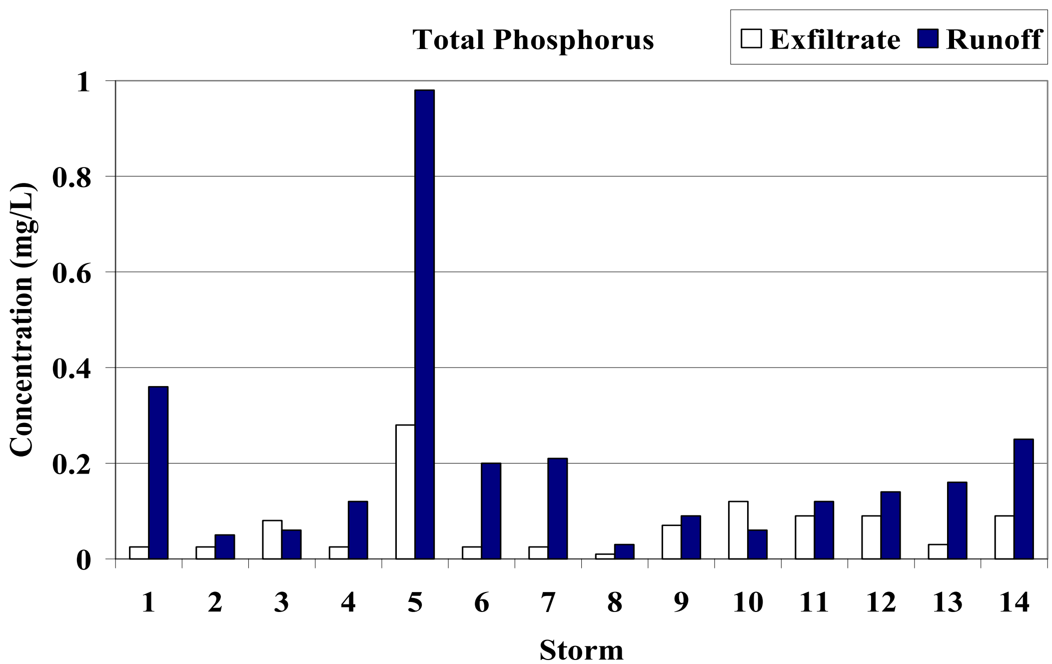

Figure 4. TP concentrations for asphalt runoff and PICP exfiltrate from Goldsboro.

The Cary site was constructed so that inflows would be entirely composed of rainfall and dry deposition; no contributing runoff would enter into the site. Per studies by Wu et al. (1998), this is a reasonable, slightly conservative assumption for TN. However, for TP this assumption is extremely conservative and thus could under predict TP removal rates.

Water quality data from Cary for 15 storms is listed in Table 3. Mean inflow and outflow concentrations are listed along with p-values (paired t-test, (SAS, 2003)) to determine significance. Ammonia was the only pollutant significantly ($p \leq 0.05$) lower in exfiltrate concentration than rainfall concentration. However, NO_3-N is significantly higher in

Pollutant	Rainfall (Inflow)	Exfiltrate (Outflow)	p-value
NO3-N (avg. mg N/l)	0.39	*1.66*	**0.043**
NH4-N (avg. mg N/l)	*0.64*	0.06	**0.034**
TKN (avg. mg N/l)	*2.33*	1.11	0.143
TN (avg. mg N/l)	2.71	*2.77*	0.964
PO4 (avg. mg P/l)	0.08	*0.34*	0.133
TP (avg. mg P/l)	0.26	*0.40*	0.424
TSS (avg. mg/l)	N/A	*12.3*	N/A

Table 3. Mean pollutant concentrations and factors of significance for Cary site.

exfiltrate concentration than in rainfall concentration. Since TN was not significantly removed, this suggests that inflowing NH_4-N and TKN were converted, by ammoniafication and subsequent nitrification, contributing more nitrate to the exfiltrate.

Unlike the Goldsboro site, TP exfiltrate concentrations were, based on means, substantially higher than rainfall concentrations. Wu et al. (1996) showed that 10 – 20% of TP in runoff could be attributed to dry deposition. Since this site also had a lower surface infiltration rate than the Goldsboro site (4000 cm/h) (Bean et al., 2004), due to the presence of fines, increased loadings are most likely due to sediment deposition of clay particles by vehicular traffic. Exfiltrate TSS concentrations for Goldsboro and Cary were essentially equal. This could possibly give a predictable exfiltrate TSS concentration. TN, for the Cary site, was not significantly removed and both rainfall and exfiltrate were higher, based on means, than the Goldsboro site concentrations. This could be attributed to either the Cary site being lined or it being adjacent to a fertilized lawn with.

Since no runoff occurred at the Swansboro site, water quality was not compared between runoff and exfiltrate. However, Table 4 summarizes exfiltrate concentrations. NH_4-N concentrations for each storm were less than the minimum detectable level. The mean TP concentration for Swansboro was comparable to the mean for Goldsboro exfiltrate as well as the range (0.025 – 0.28 mg/l). These two sites were relatively free of fines (Bean et al., 2004), and concentrations of TP in exfiltrate around these concentrations (0.005 – 0.28 mg/l) could be expected for PICP sites free of fines in sandy soil regions.

	NO_3-N mg N/l	NH_4-N mg N/l	TKN mg N/l	TN mg N/l	PO_4 mg P/l	TP mg P/l
Maximum	0.36	0.05	0.65	0.93	0.08	0.14
Mean	**0.17**	**0.05**	**0.18**	**0.36**	**0.03**	**0.06**
Minimum	0.05	0.05	0.05	0.10	0.005	0.005

Table 4. Max., mean and min. pollutant concentrations for Swansboro exfiltrate.

Table 5 presents pollutant loads passing through the Swansboro PICP site. A weighted average of each pollutant was determined by rainfall totals for analyzed storms. The weighted average concentrations were then converted to mass loads. Mass loads were then scaled up from the analyzed storm depths to the total rainfall for the entire monitoring period. TN reduction was seven times higher than TP reduction. Over 10 months, for a 0.01 ha (0.24 ac) area, 0.4 kg (0.9 lb) of TN and 0.06 kg (0.12 lb) of TP entered the site. For one complete year, TN could be expected eliminate from runoff approximately 0.5 kg (1.1 lb) or 5 kg/ha/yr of TN and 0.07 kg or 0.7 kg/ha/yr of TP.

Pollutant	kg	kg/ha	lbs	lbs/ac
TN	0.40	4.08	0.88	3.64
TP	0.06	0.58	0.12	0.51

Table 5. Total pollutant mass having passed through Swansboro PICP site.

Conclusions

Both flow monitoring sites, Cary and Swansboro, had partial and total infiltration, respectively. No runoff occurred at the Swansboro site from March 1st until December 31st. The Cary site shows the potential for being a well performing site, infiltrating 66% of inflow and attenuating peak inflows by 67%. However, more storm data sets need to be collected before general conclusions can be made about the effectiveness of the site.
At the Goldsboro site TP, TKN, NH_4-N and Zn were all present in significantly lower concentrations in exfiltrate samples compared to runoff. At the Cary site, most NH_4-N and some TKN were converted to NO_3-N, but TN concentrations were essentially equal for rainfall and exfiltrate. Sedimentation of clay fines likely contributed to higher TP concentrations in exfiltrate.

As a result of this study, siting guidelines and assessments are listed as follows: (1) sites should be kept clear of fine sediment accumulation to limit TP exfiltrate concentrations; (2) Exfiltrate TSS concentrations could be around 12 mg/l. 3) PICP sites located in sandy soils should perform extremely well as long as a) they are kept free of sediment accumulation, b) they have a several centimeter thick washed No. 57 stone subbase and c) they are unlined over a highly pervious base soil. 4) Low traffic, high infiltrating coastal PICP sites could expect to eliminate 5 kg/ha/yr and 0.7 kg/ha/yr of TP and TN, respectively, from runoff. 5) Permeable pavements in clay have the potential for substantial attenuation of runoff, dependent on maintenance and minimization of fine sediment accumulation. 6) Lined PICP sites in clay soils may have no benefit for Total Nitrogen reduction. 7) PICP significantly removes Zn and substantially removes Cu.

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Description of a Continuous Simulation Model (WEANES) for Estimating Long-Term Efficiencies of Wet Retention/Detention Basins

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Abstract

A macro spreadsheet model, WEANES (Wet Pond Annual Efficiency Simulation Model), has been developed to predict the long-term (multi-year) or annual removal efficiencies of wet retention/detention basins. The model can serve as a design, planning, and permitting tool for consulting engineers, planners and government regulators. The model uses historical, site-specific, multi-year, rainfall data, usually available from a nearby National Oceanic and Atmospheric Administration (NOAA) climatological station to estimate basin efficiencies which are calculated based on annual mass loads. Other required input parameters are: 1) watershed parameters; drainage area, pervious Curve Number, directly connected impervious area, and time of concentration, 2) pond parameters; control and overflow elevations, pond side slopes, surface areas at control elevation and pond bottom; 3) outlet structure parameters; 4) pollutant event mean concentrations; and 5) pond loss rate which is defined as the net loss due to evaporation, infiltration and water reuse. The model offers default options for parameters such as pollutant event mean concentrations and pond loss rate.

Introduction

One of the Best Management Practices (BMPs) being extensively used for treatment of stormwater is wet retention/detention basins. This particular BMP is being favored because it controls flooding, is aesthetically pleasing, and also provides treatment to the stormwater. Like most BMPs, a problem confronting the planners and engineers designing wet detention basins, to meet targeted pollution reduction goals, is the lack of accurate data regarding the long-term or annual removal efficiency of these ponds (Pandit and Youn, 2002 and 2003). A macro spreadsheet model, WEANES (Wet Pond Annual Efficiency Simulation Model), has been developed to predict the long-term or annual removal efficiencies of wet retention/detention basins. The model uses historical, site-specific, multi-year, rainfall data, usually available from a nearby National Oceanic and Atmospheric Administration (NOAA) climatological station to estimate pond efficiencies. The objective of this paper is to show how the model results were calibrated and validated, and to provide examples showing how WEANES can be used for:

1. **Permitting**: The model should have the ability to determine the long-term removal efficiency of an existing or proposed wet retention/detention pond at any location, and
2. **Pond Design and Planning**: The model should be able to identify key design parameters that most affect the long-term removal efficiency of a proposed wet detention pond and should aid in designing a pond that would meet targeted pollution reduction goals.

Modeling Processes Used in WEANES

WEANES determines the long-term efficiencies of wet detention ponds by modeling the following processes, described in Figure 1:

1. Converting rainfall hyetographs to inflow hydrographs into the wet pond using the Santa Barbara Urban Hydrograph (SBUH) method,
2. Determining the outflow hydrographs from the weir and the orifice by routing the inflow hydrographs through the wet detention pond using the Level Pool Method,
3. Obtaining the average annual inflow and outflow volumes (from the weir and the orifice) from hydrographs, and
4. Determining the pond efficiency based on annual mass inflow and outflow by the following equation:

$$AARE = \left[\frac{(1-n)(AARV)_{or} + (1-m)(AARV)_{w} + (AARV)_{l}}{(AARV)_{i}} \right] \times 100 \tag{1}$$

where AARE is the average annual removal efficiency of a wet detention pond, with respect to any pollutant, $(AARV)_i$ is the average annual runoff volume entering the pond during storm events, $(AARV)_{or}$ and $(AARV)_w$ are the respective average annual runoff volumes leaving the outflow through the orifice and the weir, $(AARV)_l$ is the net average annual loss duet to combined effects of evaporation and groundwater seepage, $n = (AAEMC)_{or}/(AAEMC)_i$, and $m = (AAEMC)_w/(AAEMC)_i$, $(AAEMC)_i$ is the average annual event mean concentration of the pollutant in the inflow pipe at the basin entrance, and $(AAEMC)_{or}$ and $(AAEMC)_w$ are the respective average annual event mean concentrations of the pollutant in the water being discharged from the orifice and the weir.

Model Calibration and Validation

Introduction

WEANES was calibrated and validated for a 56-day period between May 6 through June 30, 2002, using measured data from a pond referred to as the Basin 7 Wet Detention Pond which is located in Palm Bay, Florida. The pond receives runoff from a 75.87-acre watershed as shown in Figure 2. The information for the watershed, pond, and outfall structure for the pond, shown in Table 1, were provided by the St. Johns River Water

Management District (SJRWMD). The daily and monthly rainfall depths were measured at two rainfall stations known as Palm Bay STP and Basin 7. The daily pond loss rate was estimated during dry-weather conditions, i.e. during days when there was no rainfall in the previous 24 hours, and was computed by finding the difference in measured water levels. The daily loss rate for the measurement period was 0.22 in/day (Youn, 2004). The following parameters were also continuously measured at the pond from May 6, 2002 to December 31, 2002, by SJRWMD; inflows to the pond, outflows from the pond, and total suspended solid (TSS) concentrations at the inflow and outflow pipes.

Calibration Procedure

The calibration was conducted in the following two steps:

- **Step 1 - Adjustment of Side Slopes to get desired treatment volume (TV) and permanent pool volume (PPV)**: The side slopes and corresponding TV and PPV for the pond at various stages, obtained from the design engineer, are shown in Table 2. The design treatment and permanent pool volumes for the pond were 4.036 ac-ft and 13.589 ac-ft, respectively. The model converts irregular shapes, such as that of the Basin 7 Wet Detention Pond, to a prismatoid shape. When the side slope data were entered into the model, it yielded 55% and 54% lower TV and PPV values than the actual values supplied by the design engineer. Pond slopes were, therefore, adjusted until the TV and PPV values were exactly equal to 4.036 ac-ft and 13.589 ac-ft, respectively. Values of the adjusted slopes and the calibrated PPV and TV are also shown in Table 2.
- **Step 2 - Adjustment of DCIA value**: The design engineer supplied DCIA value was changed from 24.54% to 19.63% to provide improved comparisons between the measured and model simulated water levels as shown in Figure 3. The maximum difference between model simulated water levels and the measured water levels was 5%, while the average difference between was 1%.

Validation Procedure

The model was validated by:

1. Comparing measured and model simulated water levels, and
2. Comparing measured and model simulated monthly TSS removal efficiencies.

The model simulated water levels are compared to the measured water levels for the 184-day validation period between July 1 through December 31, 2002 in Figure 4. The maximum difference between model simulated water levels and the measured water levels was 8%, while the average difference was 3% during the validation period.

The model simulated monthly TSS removal efficiencies are compared with the measured monthly TSS removal efficiencies for a 8 month period between May through December 2002 in Figure 5. The largest difference for the model predicted monthly efficiency and the measured monthly efficiency for TSS is 31.8% for the month of September while the lowest difference is zero for the month of May when both the model predicted and measured monthly efficiencies are 100%. The model predicted efficiency for the entire 8 month period is within 4.6% of the measured efficiency.

The difference between the measured and model predicted monthly efficiencies during some of the months can be due to several reasons:

1. The measured monthly (for example, September) loads and efficiencies are calculated based on the average monthly inflow and outflow concentrations for that month (September), while WEANES estimated the monthly loads using the average inflow and outflow concentrations for the entire 8 month period. Therefore, large variations in the average monthly concentrations and the average concentration for the entire 8 month period will lead to higher differences between measured and model simulated monthly removal efficiency values,
2. The model predictions can probably improve even more if the model used different values of n for the wet and dry seasons. By using a single n value for the entire simulation period, the model generally under predicts in dry months and over predicts during wet months, and
3. The model assumes that the discharge over the weir is untreated and therefore, TSS loads discharged over the weir were estimated based on average measured inflow concentrations. The model calculates the loads discharged through the orifice based on measured outflow concentration at the orifice. However, in practice, the outflow concentrations were measured at a location where discharges through the orifice and the weir had already mixed, instead of at the orifice.

Model Applications

The Basin 7 Wet Detention Pond was selected for the purpose of illustrating how WEANES can be used for model permitting, and pond design and planning.

Permitting

Let us assume that a wet detention pond, with characteristics identical to the Basin 7 Wet Detention Pond, is to be constructed in the following three Florida cities: Jacksonville, Melbourne and Miami. Further, the three pollutants under consideration are total suspended solids (TSS), total phosphorus (TP) and total nitrogen (TN). The model is to be used to determine the long-term efficiency of the pond in removing these pollutants using the local 10-year rainfall distribution between the years of 1993 and 2003. Let us assume that the target removal efficiencies for the three pollutants, TSS, TP, and TN are respectively 85 %, 70 % and 35 %. The average 10-year rainfall depth at the three stations for this period ranged from 50.75 inches to 66.72 inches as shown in Table 3. The model calculated AARE values for TSS, TP and TN for the three cities are shown in Table 3. The differences in AARE values stem not only due to the differences in average rainfall depths of the three cities but also due to the differences in the historical rainfall patterns. It should be noted that while Melbourne has a lower average rainfall depth for the 10-year period than Jacksonville, the pond is more efficient when it is located in Jacksonville. Based on these calculations the pond would not receive permitting in any of the three locations as it does not meet the targeted goals. The next section will indicate what improvements can be made to the pond in the three locations to obtain the desired results.

Pond Design and Planning

Sensitivity analyses indicated that the targeted goals can be met by increasing either the treatment volume (TV) or by increasing the pond loss rate. Pond loss rates can be increased by adding exfiltration systems. It should be noted that there can be other ways for meeting targeted goals. For example, one could try increasing both the TV and the pond loss rate. The results of the model analyses are shown in Table 4. These results indicate that targeted goals can be achieved by increasing the TV from the design value of 4.036 ac-ft to 8.945 ac-ft for Melbourne, Florida. A higher treatment volume (11.275 ac-ft) would be required if the pond was located in Miami while a slightly lower (7.288 ac-ft) TV would be required if the pond was located in Jacksonville, Florida. The targeted goals can also be achieved if the pond loss rates were increase from a design value of 0.22 in/day to 0.36 in/day, 0.43 in/day and 0.80 in/day if the pond was respectively located in the cities, Jacksonville, Melbourne, and Miami.

Conclusion

A macro spreadsheet model, WEANES (Wet Pond Annual Efficiency Simulation Model), has been developed to predict the long-term (multi-year) or average annual removal efficiencies of wet retention/detention basins. The model has the ability to use real, historical, multi-year rainfall data from a National Oceanic and Atmospheric Administration (NOAA) climatological station near the project area to estimate pond removal efficiencies. Moreover, it has been demonstrated that WEANES can also be used to obtain monthly or annual pond removal efficiencies if so desired by the user. As part of the process of calculating long-term pollutant removal efficiencies, WEANES also continuously predicts the water level in the pond, and allows the user to graphically observe the periods when the water levels exceed the control elevation or the overflow elevation. The model can be used for model permitting, and pond design and planning.

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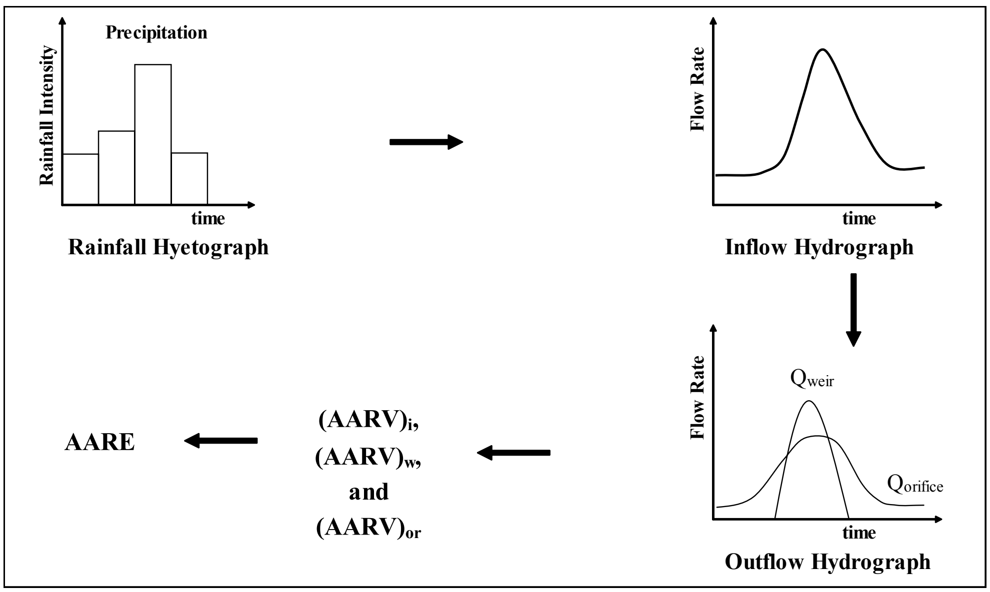

Figure 1. Description of Processes Modeled by WEANES

Figure 2. Site Location of Basin 7 Wet Detention Pond, Palm Bay, Florida
Source: St. Johns River Water Management District

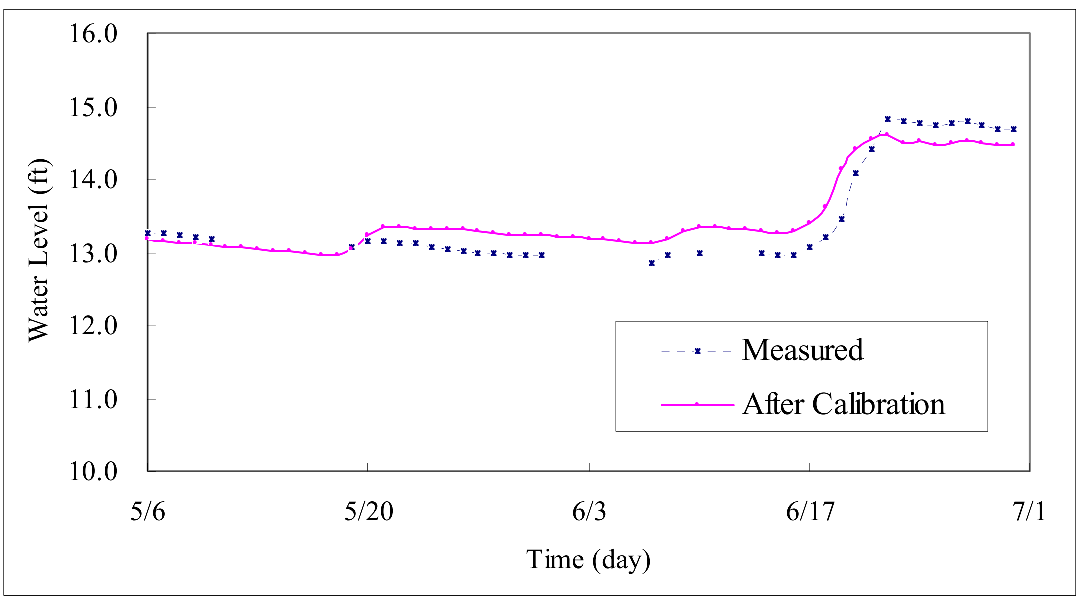

Figure 3. A Comparison of WEANES Simulated Water Levels with Measured Water Levels for Basin 7 Wet Detention Pond from May 6, 2002, to June 30, 2002, After the Calibration Process

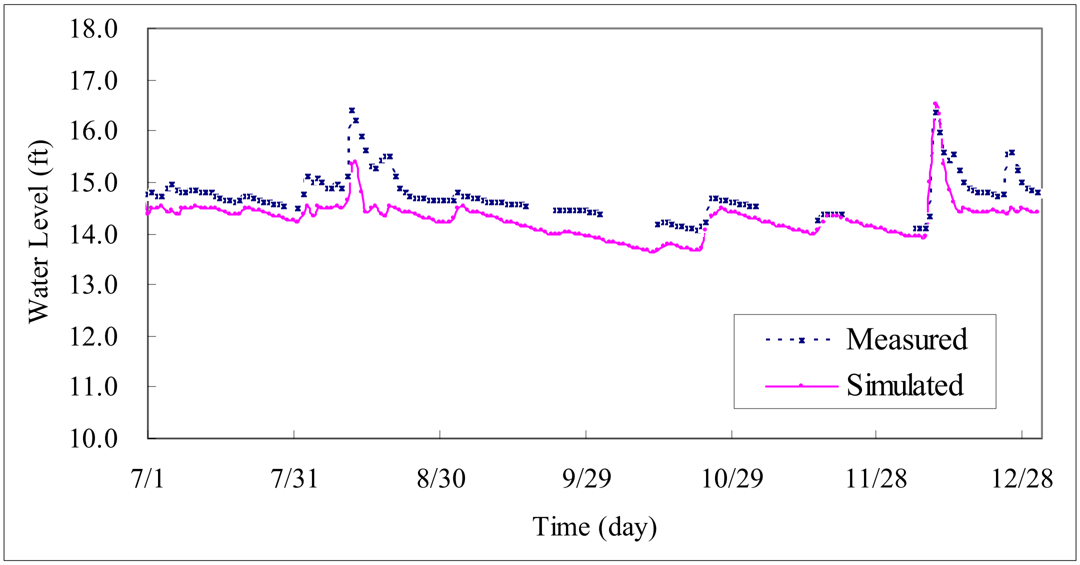

Figure 4. A Comparison of WEANES Simulated Water Levels with Measured Water Levels for the Basin 7 Wet Detention Pond from July 1, 2002, and December 31, 2002

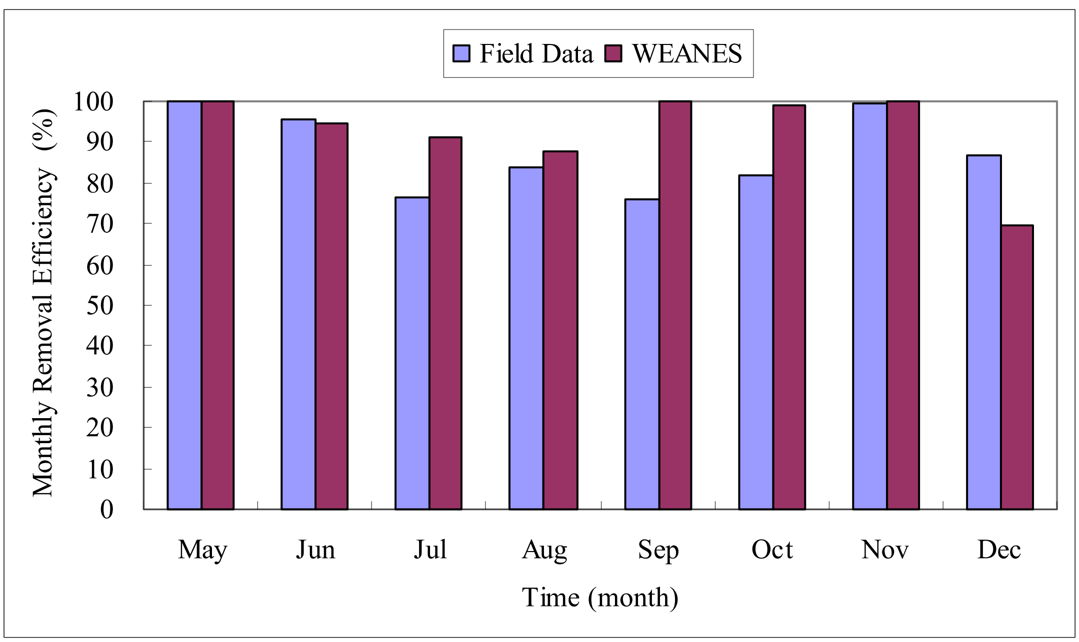

Figure 5. Comparison of WEANES Simulated TSS Efficiencies with Measured Values for Basin 7 Wet Detention Pond

Table 1. Information for Watershed, Pond, and Outfall Structure for Basin 7 Wet Detention Pond Provided by St. Johns River Water Management District

Watershed	Basin Area (ac)		78.15
	Pervious Curve Number, CN_p		69.1
	Directly Connected Impervious Area, DCIA (%)		24.54
	Time of Concentration, t_c (hr)		14.5
Pond	Key Elevation	Overflow Elevation, OE (ft)	15.95
		Control Elevation, CE (ft)	14.32
		Bottom Elevation, BE (ft)	4.00
		Side Slope Elevation, h_1 (ft)	12.00
		Side Slope Elevation, h_2 (ft)	14.00
	Side Slope (Horizontal to Vertical)	Side Slope from BE to h_1	2 to1
		Side Slope from h_1 to h_2	4 to1
		Side Slope above h_2	4 to1
	Configuration	Ratio (Length to Width)	18 to 1
	Area	Surface Area at Control Elevation (ft)	99217
		Bottom Area at Bottom Elevation (ft)	14375
Outfall Structure	Rectangular Weir	Weir Length, L_w (ft)	1.25
	Orifice	Orifice Diameter, d (in)	6.00

Table 2. Comparison of Calibrated Values with Design or Field Estimated Values for the Basin 7 Wet Detention Pond

Stage (ft)	[1]Side Slopes (H: V)	Area (ac)	[2]Permanent Pool Volume (ac-ft)	[2]Treatment Volume (ac-ft)	Adjusted Slopes	[3]Permanent Pool Volume (ac-ft)	[3]Treatment Volume (ac-ft)
4.00	2: 1	0.330			6.64: 1		
12.00	2: 1	1.673			6.64: 1		
14.00	4: 1	2.193			6.54: 1		
14.32	4: 1	2.278	13.589		2.65: 1	13.589	
15.95	4: 1	2.714		4.036	2.65: 1		4.036
19.00	4: 1	3.550			2.65: 1		

[1]Values obtained from the design engineer. [2]Values estimated by WEANES based on the design slopes. [3]Values estimated by WEANES based on the adjusted slopes.

Table 3. Model Permitting for Basin 7 Wet Detention Pond Using Calibrated Values

Climatological Station	10-Year Period (1993 - 2002) Average Rainfall Depth (in)	Average Annual Removal Efficiency (AARE)		
		TSS (n* = 0.15) (%)	TP (n* = 0.43) (%)	TN (n* = 0.87) (%)
Jacksonville	51.11	84.0	65.0	35.5
Melbourne	50.75	81.7	62.6	33.1
Miami	66.72	75.9	56.7	27.0

*n is an orifice treatment factor, for n =0.15, i.e., the pollutant concentration at the orifice is 15 % of the pollutant concentration at the inlet.

Table 4 Adjustment of TV or Loss Rate to Meet Targeted AARE Goals for the Basin 7 Wet Detention Pond, Hypothetically Located in Jacksonville, Melbourne, or Miami

Location	Redesign*	Design Parameter	Value	Average Annual Removal Efficiency (AARE)		
				TSS (n = 0.15) (%)	TP (n = 0.43) (%)	TN (n = 0.87) (%)
Jacksonville	Case 1	Treatment Volume (ac-ft)	7.288	88.5	70.2	42.0
	Case 2	Loss Rate (in/day)	0.36	85.9	70.4	46.3
Melbourne	Case 1	Treatment Volume (ac-ft)	8.945	88.2	70.4	42.9
	Case 2	Loss Rate (in/day)	0.43	84.7	70.1	47.5
Miami	Case 1	Treatment Volume (ac-ft)	11.275	86.6	70.1	44.6
	Case 2	Loss Rate (in/day)	0.80	81.5	70.1	52.4

*Two types of redesign are shown; Case 1 implies increase in TV and Case 2 implies increase in loss rate by putting in exfiltration systems.

Hillsborough River Basin Assessment Using WAM

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Abstract

The Watershed Assessment Model (WAM) was used to evaluate the flow, nitrogen, phosphorus, and biological oxygen demand (BOD) levels throughout the Hillsborough River Basin near Tampa, Florida. WAM was selected because of its comprehensive spatial analysis of land source areas and complex stream network routing techniques. WAM was able to simulate the control protocols for the structures on a major flood bypass system, a branched stream loop, and the in-stream water supply reservoir. Surface and groundwater water supply withdrawals and other point source discharges and withdrawals were well simulated by the model. WAM flow and constituent results matched well with observed data throughout the basin. WAM was also used to provide flow and nutrient/BOD loading information for a seven-year period (nine minute interval) to Water Quality Analysis Simulation Program (WASP) model. WASP is being used to simulate the DO and algal levels within TMDL-listed stream reaches to support EPA TMDL development.

Introduction

The project objective was to simulate water quantity and quality discharges within the Hillsborough River Basin in support of the Florida Department of Environmental Protection's TMDL Program. The basin is located on the west coast of Florida and includes a portion of the City of Tampa. The basin area is approximately 675 square miles and spans portions of three counties – Hillsborough, Pasco, and Polk (see Figure 1).

Figure 1. Hillsborough River Basin Locator Map

Land uses within the basin vary from dense urban to rural and agricultural. The three most dominant land uses are agricultural (31%), urban (28%), and wetlands (21%). The dominant agricultural land use in the basin is pastureland.

Stormwater-runoff and groundwater drain through a network of streams and springs to the Hillsborough River, which discharges into Hillsborough Bay and, ultimately, Tampa Bay and the Gulf of Mexico. There are special hydrologic features within the basin that had to be accommodated for in the model simulations. An old water supply reservoir and flood protection systems constructed in the 1960's are the prominent features. This system includes a widened section of the Hillsborough River and control structures operated to provide flood protection to the southern reaches while maximizing basin storage for consumptive water use. The structures control flow by diverting water to or drawing water from an offsite conveyance system known as the Tampa Bypass Canal.

Groundwater movement is another special feature in this basin that the modeling had to handle. Springs located in and around the basin include contributing areas known as "springsheds" that do not match the surface water flow patterns in much of the basin. Some groundwater within the basin discharges out of the basin and does not enter the river. Conversely, some areas outside of the basin contribute groundwater to the river. Also, large groundwater withdrawals for domestic use had to be simulated to properly handle the water balance.

The primary concern in the basin is nutrients and BOD loads and associated low dissolved oxygen (DO) levels. WAM was used to simulate the flow, BOD, TSS, and nutrients within the streams. These data will be used as input data for the WASP 6.0 model, which is being used to simulate DO. The WASP modeling has not been completed and, therefore, is not presented in this paper.

Modeling Approach

WAM is a Geographic Information System (GIS) based model that allows users to interactively simulate and assess the environmental effects of various land use changes and associated management practices. WAM output will be used directly to provide flow and water quality responses throughout a basin as well as for providing inputs for the WASP model to simulate the DO responses within the basin's (TMDL) listed reaches.

WAM utilizes ESRI™ ArcView 3.2 with Spatial Analyst 2.0 to analyze and display model input and output using grids. Grid datasets, as opposed to polygon datasets, spatially represent geographic data as a collection of raster cells. Each cell contains attributes of the dataset, e.g. land use code numbers that can be overlaid with cells of other grids. The benefits of using grids over polygons include computational speed and output resolution. Output can be displayed by grid cell as opposed to by subbasin polygon. The cell size is dependent on the desired resolution. A grid cell size of 1 ha was chosen with the intent that this would adequately characterize the land use and capture linear features such as highways.

The GIS based processing and user interface in the WAM model allows for a number of user options and features to be provided including:

- Source Cell Mapping of TSS, BOD, and Nutrient Surface and Groundwater Loads
- Tabular Ranking of Land Uses by Constituent Contributions
- Overland, Wetland, and Stream Load Attenuation Mapped Back to Source Cells
- Accommodation of Point Source Information
- Adjustments based on wastewater treatment plant (WWTP) Service Area locations
- Hydrodynamic Stream Routing of Flow and Constituents with Annual, Daily or Hourly Outputs
- User Interface to Run and Edit Land Use and BMP Scenarios

The water quality parameters (impact parameters) simulated within the model include: water quantity, soluble forms of nitrogen (N), including ammonia, nitrate, and soluble organic N, soluble phosphorus (ortho-P), particulate N and P, groundwater N and P, total suspended solids (TSS), and BOD. GIS datasets of land use, soils, WWTP service areas, and rainfall are used to calculate the combined impact of the watershed characteristics for a given grid cell. Once the combined impact for each unique cell within a watershed is determined by various field-scale submodels, the cumulative impact for the entire watershed is determined by routing each cell's surface and groundwater and related constituents with attenuation to the nearest stream and once in the stream hydrodynamically routed through the stream network to the basin outlet. Constituents are attenuated based upon the flow distances (overland to nearest water body, through wetlands or depressions and within streams to the sub-basins and basin outlets), flow rates in each related flow path and the type of wetland or depression encountered. Figure 2 shows the conceptual routing schemes and flow distances that are calculated for each cell. A portion of the flow in each cell is converted to groundwater based on the soil type and amount of imperviousness estimated for each land use. Surface flow that enters depressions is also converted to groundwater. Groundwater is routed to the nearest stream unless directed otherwise.

The hydrologic contaminant transport modeling is accomplished by first simulating all of the unique grid cell combinations of land use, soils, WWTP service areas, and rainfall by using one of several source cell models including GLEAMS (Knisel, 1993), EAAMOD (Bottcher et al., 1998; SWET, 2000), a wetland module, and an urban module. These source cell submodels are called by BUCShell based on the land use of the unique combination (see Figure 2). The time series outputs for each grid cell is then routed and attenuated to the nearest stream and then through the entire stream network of the watershed. Dynamic routing of flows is accomplished within BLASRoute through the use of an algorithm that efficiently solves Manning's equations (Jacobson et al., 1998).

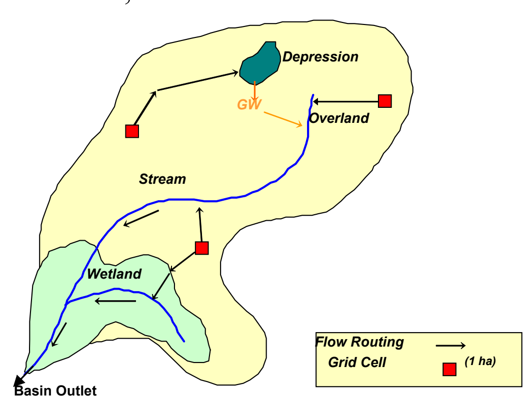

Figure 2. Conceptual Flow Path Routing for WAM

Figure 3 below shows a flow diagram of the overall function of the WAM model.

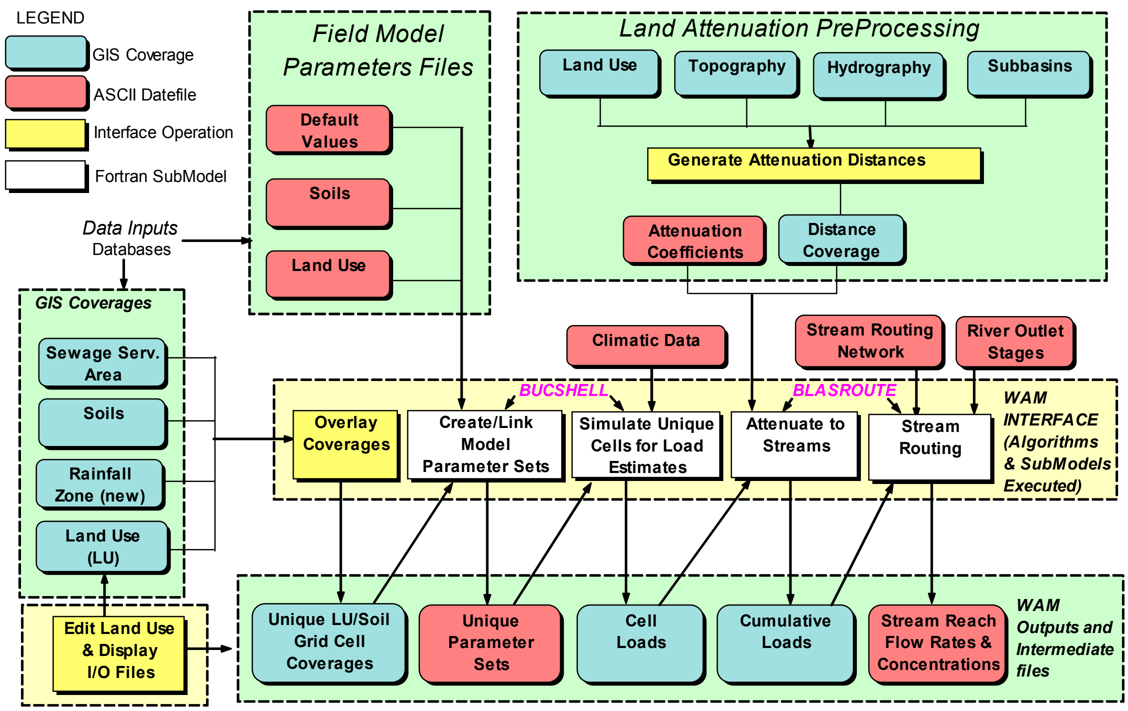

Figure 3. Dynamic Modeling Approach in WAM

WAM Setup for Hillsborough Basin

WAM was setup for the Hillsborough basin using WAM utility programs that assists the user in developing the GIS coverages and datasets needed to run WAM. The setup included clipping and converting the GIS coverages of land use, soils, rainzones, springsheds, and WWTP service areas to grids; developing the stream reach network from hydrographic data and control structure data; development of routing grids; and formatting and verifying weather, point source, surface and groundwater withdrawal data (SWET, 2004).

GIS Datasets

All of the GIS spatial datasets necessary to set up WAM were provided by the Florida Department of Environmental Protection (FDEP) using their custom Albers projection in the HPGN (metric) datum. Most of the datasets were obtained by FDEP and SWET from other sources including Southwest Florida Water Management District (land use), Natural Resource Conservation Service (soils) and United States Geographical Survey (topography and hydrography). The SURGO soils datasets were modified to include abbreviated Compname soil designations in order that these attributes would match WAM soils database established for the State. The land use dataset utilizes the Florida Land Use Code Classification System (FLUCCS), which is also utilized by WAM. Therefore, no modifications were necessary.

National Hydrologic Datasets (NHDs) were used for the hydrologic stream network. Modeled reach types include stream, canal, slough, groundwater and shoreline. The groundwater reach is of particular interest to this study. This type of reach was used to create connectivity between isolated surface reaches and the remainder of the reach network, e.g. Sulphur Springs. Direct surface water is not routed to this type of reach. The groundwater reach was also used to direct groundwater out of the basin to account for groundwater that emerges into offsite streams or springs. Within the stream network are numerous water control structures and multiple flowpaths to a downstream reach, which is called looping. These features are uniquely handled by WAM.

In total, 203 hydrologic model reaches were assigned, but note that 5 of the reaches are boundaries, including a tidal boundary at the river's outlet, and are not physical reaches within the watershed. A sample of these reaches is shown in Figure 4.

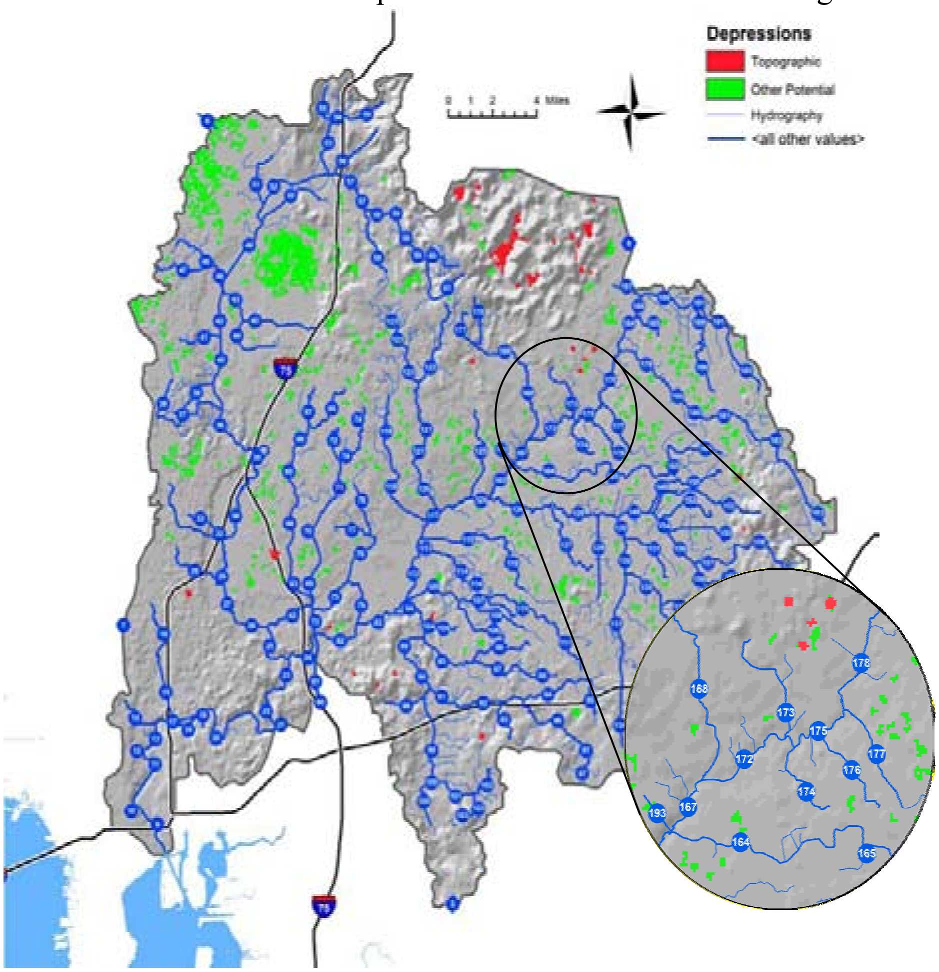

Figure 4. Stream Reach Network for Hillsborough River Basin

Other Input Data

Rainfall datasets were created from monitoring information obtained from the National Weather Service (NWS). A 24 year period between January 1980 and December 2003 was chosen, though it was expected that the model would not utilize this entire period. The model uses a five-year "spin up" period. That is, it throws away the first five years of data in order to ensure that equilibrium with antecedent conditions is reached prior to the period of interest.

Depressions are one of the three geographic features (wetlands, streams and depressions) that to which WAM routes runoff and attenuates constituents. Areas that drain to depressions are attenuated differently than areas that drain via surface water to the streams. Runoff entering depressions converts to groundwater with very little phosphorus and no suspended solids re-emerging in the streams.

WAM attenuates water quality parameters in the runoff based on distances and the type of geographic feature the runoff passes through (upland, wetland, depression and stream. The WAM Primary Basin Setup includes an algorithm to develop a series of ASCII grids for this purpose. ArcView™ Spatial Analyst cost distance functions are employed to both determine distance and attributes of the feature that is found. For example, when the closest wetland is found, the wetland FLUCCS number can also be returned. In addition, a grid of wetland distances to streams can be accessed to return the distance to the next feature.

Major water control structures were entered into the WAM model based on information provided by FDEP. Some of the structures were modeled as other types of facilities to simplify the simulation and to better represent existing conditions. For example, gates could have been modeled as pumps in order that daily recorded flow records could be used as a boundary condition. Table 1 provides a summary of these structures.

Table 1: Water Control Structures

Structure	**Description**
Sulpher Springs Pump	Pump - recorded daily flows used
Reservoir Dam	Fixed weir with crest elevation of 27.0'
S-155	Gates normally open, closed when stages > 28.0' at Fowler Ave
S-159	Gates modeled as weir with crest elevation of 27.0'
S-161	Gates modeled as pump with recorded daily flows, bi-directional
S-163	Gate normally open, closed when stages > 28.0' at Fowler Ave

Water treatment plants, Tampa Water and Crystal Springs, were added to the model to represent the continuous withdrawal of water along the stream system for consumptive use. Model reaches were added at the water treatment plant locations and pump structures were added with daily recorded flow rates for each facility. Point sources contribute pollutants to the stream network on a continual or recurring basis regardless of climate conditions. These sources are important when summing pollutant loads over a long period of time. A database was provided by FDEP for wastewater treatment plants with permitted NPDES outfalls within basin. The recorded monitoring data for these sites were reviewed for content. The review yielded four sites with treated effluent and three stormwater sites discharged to streams that were considered significant enough, in terms of volumes and concentrations, to affect the modeling results. Unfortunately, many records were incomplete or did not include all of the parameters needed in the WAM model. Average values were used where needed. Figure 5 shows locations of water control structures, water treatment plants, and point sources.

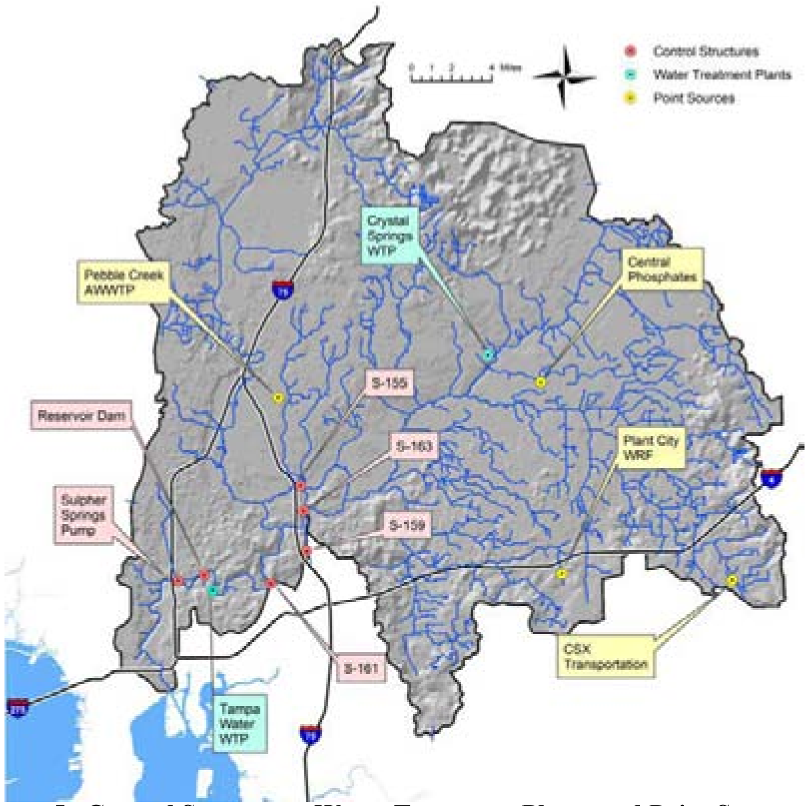

Figure 5. Control Structures, Water Treatment Plants and Point Sources

Potentiometric maps were used to assign the groundwater flow basins. These maps were converted to spatial datasets to create a direction grid and a depth grid (by subtracting it from a grid of topography). Groundwater zones were delineated based on these two datasets. Zones were assigned stream reach numbers based on the first stream encountered, within the direction of groundwater flow that corresponded to a depth to groundwater of 4 meters or less. Areas with depths greater than 4 meters are typically located in the higher, upstream portions of the basin where streams are relatively small and shallow. Groundwater in these areas is assumed to bypass these streams and enter the stream network at another location, typically somewhere further downstream.

Calibration/Validation and Model Runs

Because WAM is a physically based model, very little, if any traditional calibration is required. The primary calibration/validation exercise is to identify watershed characterization data problems by comparing model predictions against observed data. If discrepancies are found, then an investigation is done to find which model inputs are not being properly represented. The hydrologic/hydraulic responses are investigated first and then water quality. The initial data checks for causes of discrepancies are land use type or management misrepresentations and stream network layout errors. For in-stream flow problems, typically flow structure controls or stream profile information have data errors due to data entry problems in the original data sets. Water quality problems are typically caused by land use mapping errors and poor knowledge of land use management activities such as fertilizer usage, wastewater treatment, and use of retention/detention ponds.

Twenty six monitoring sites that contain various amounts of data on flow, stage, and water quality constituents were available for model calibration and validation. The primary data issue that was first identified during this process was that the flow data for Structure S-159 was not available; therefore, the actual flow going out of basin to the Bypass Canal was not available. WAM was forced to simulate the operation of the S-159, which did not have recorded operational data. This was done using a fixed crest weir. Based on observed stages at S-159, the weir crest was adjusted to best represent observed conditions. Another structure of concern is S-161, the second downstream bypass structure, where the quality of the flow data was uncertain. Two separate sources of flow data had different values for the same time period. The higher confidence source, as indicated by FDEP, which also has a longer period of record, was used for flow data at S-161. Errors in S-161 flow data would primarily affect the reservoir.

A very significant water balance issue identified was that the simulated flow at the dam and several upstream sites were higher than observed flows. Visual inspection of the flow data clearly showed that significant ground water withdrawal must be occurring because the trailing limbs of the measured data hydrographs dropped off so quickly. Two possible sources of groundwater extraction that would cause this drop off are natural groundwater gradients out of the watershed and consumptive water use withdrawals by the many wells in the watershed. The groundwater gradients causing flow out of the watershed were accounted for by the springsheds, and the likely cause of the mismatch was domestic groundwater withdrawals. A comparison of the predicted groundwater losses and reported withdrawals (Weber and Perry, 2001) found that there was more than enough groundwater withdrawal occurring within the watershed to account for the approximate 140 MGD loss observed at the reservoir. The model has been adjusted to withdraw groundwater proportionally from all subbasins that have groundwater wells reported in them. Though this withdrawal distribution technique significantly improved watershed scale water balances, it should be noted that individual subbasins may still have flow discrepancies that could only be improved by a comprehensive effort to map groundwater withdrawals by subbasins. It is recommended that this be done in the future if significant discrepancies are found.

A comparison of predicted water quality against observed data found that the nitrogen, phosphorus, TSS, and BOD were in reasonable agreement without calibration. However, it was found that a better representation of the extensive slough along Cypress Creek and

Lake Thonotassasa corrected an over-prediction of nitrogen and phosphorus in these reaches.

Figures 5 and 6 show the results for soluble nitrogen and phosphorus, respectively, at both the source cell and subbasin level, respectively. Figures 7 through 10 show the simulated versus observed flow, BOD, ortho-P, and total N for the Hillsborough Reservoir Dam. As can been seen, reasonable matches were obtained. However, there some short term mismatches of flow, which is speculated to be the result of temporary surface storage in the upper reaches, primarily Cypress Slough. It is suspected that the multitude of small lakes not represented in the hydrographic develop significant storage during dry periods and thus reducing surface flow until they are refilled to an overflow point. The long term balance is correct, but daily flows will be less accurate.

Use of WAM Outputs for WASP Inputs

A primary use of WAM was to provide the hydrodynamic flows and nutrient and BOD inputs for the TMDL listed reaches within the basin. DO in these reaches were also of concern, but WAM does not simulate DO. The WASP model was linked to WAM because of its ability to simulate the algal and DO levels within a stream reach. WASP requires a "hyd" input file to provide flows and water volumes within each reach and a "boundary" file to provide nutrients and BOD loads entering each reach. WAM was modified to provide these files for the specific conditions of the Hillsborough River basin. The calibration of the WASP model has not been completed and therefore will be the subject of a future paper.

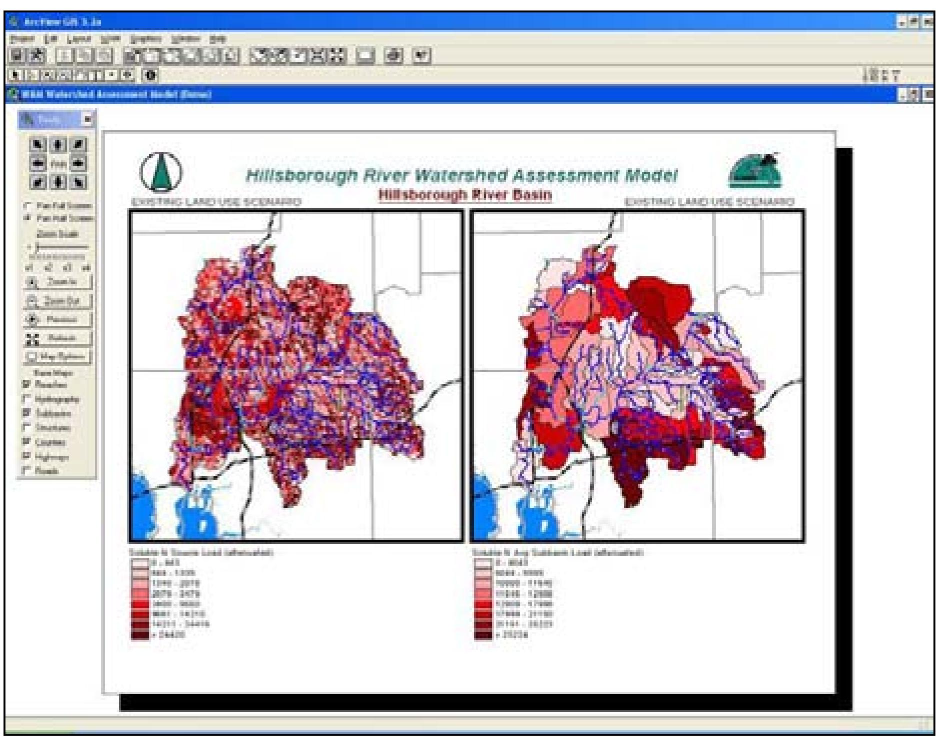

Figure 6. Soluble Nitrogen for Land Source

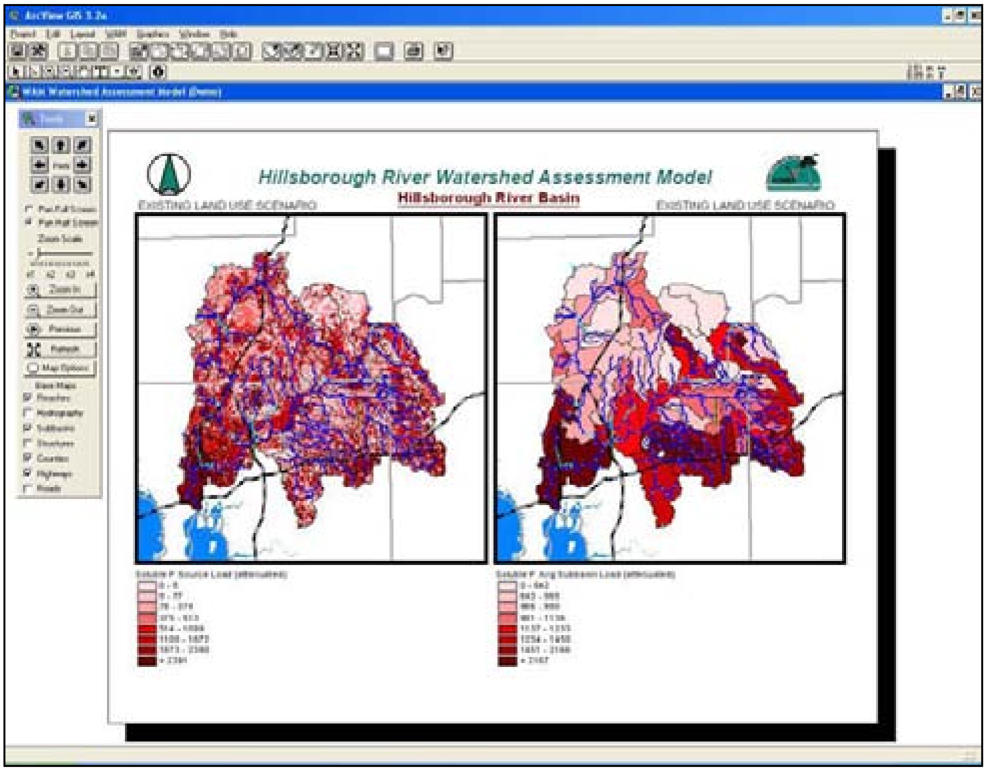

Figure 7. Soluble Phosphorus from Land Sources

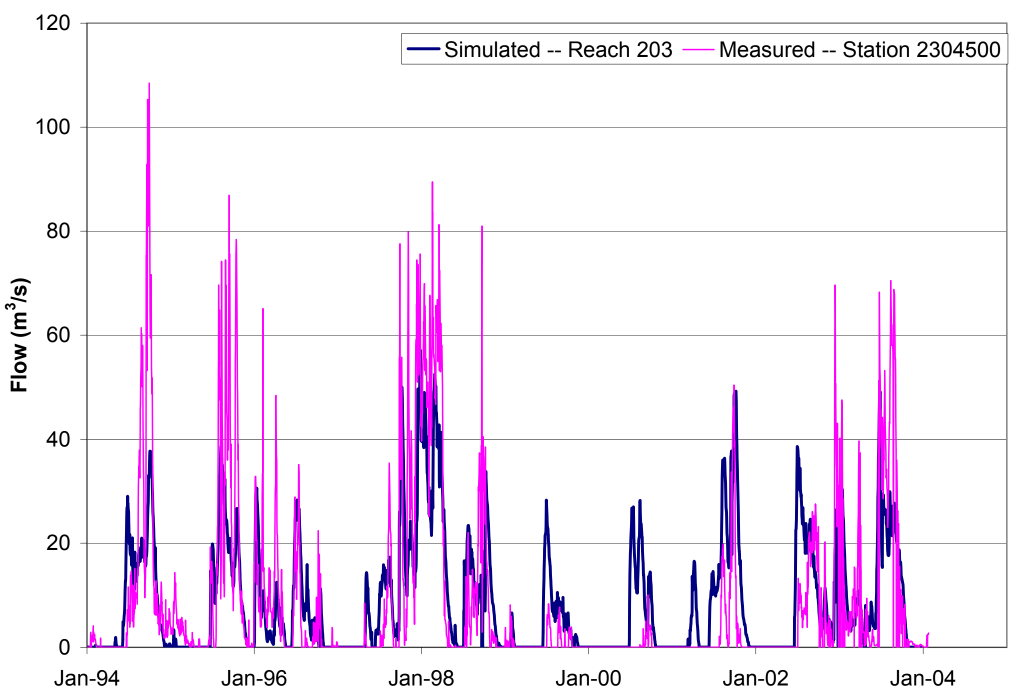

Figure 8. Flow at the Hillsborough River Reservoir Dam

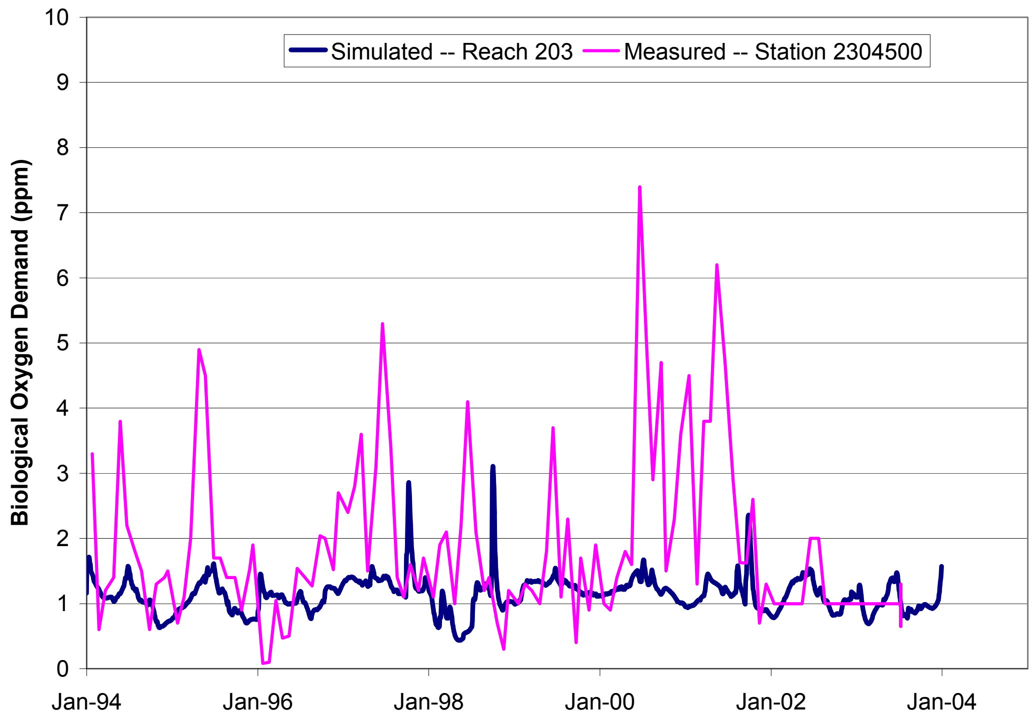

Figure 9. BOD at the Hillsborough River Reservoir Dam

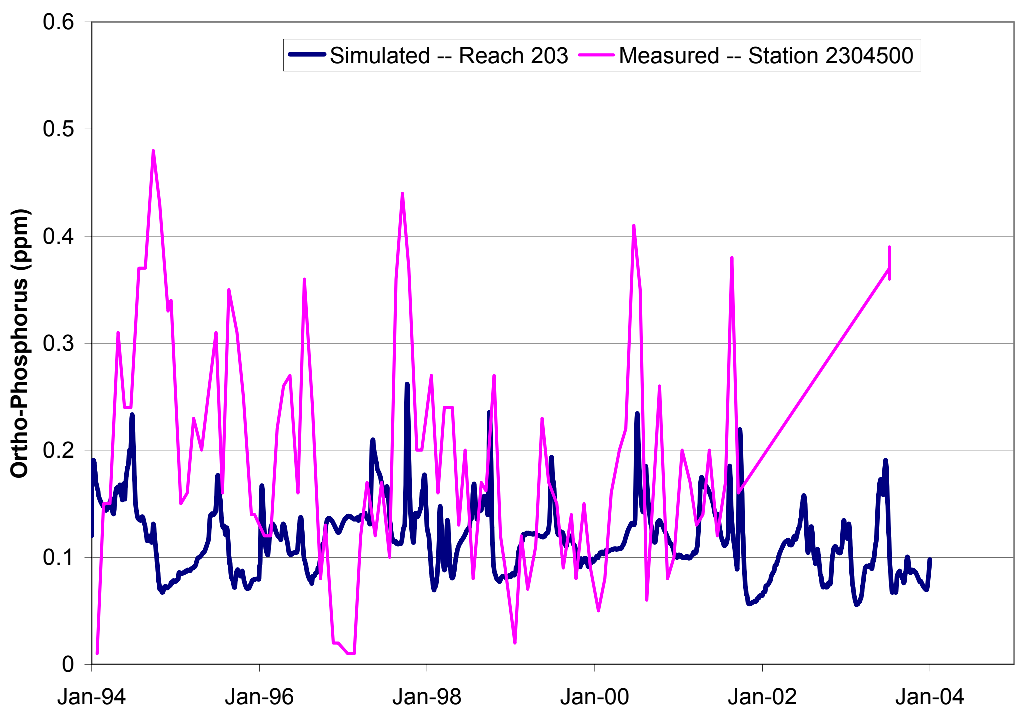

Figure 10. Ortho-P at the Hillsborough River Reservoir Dam

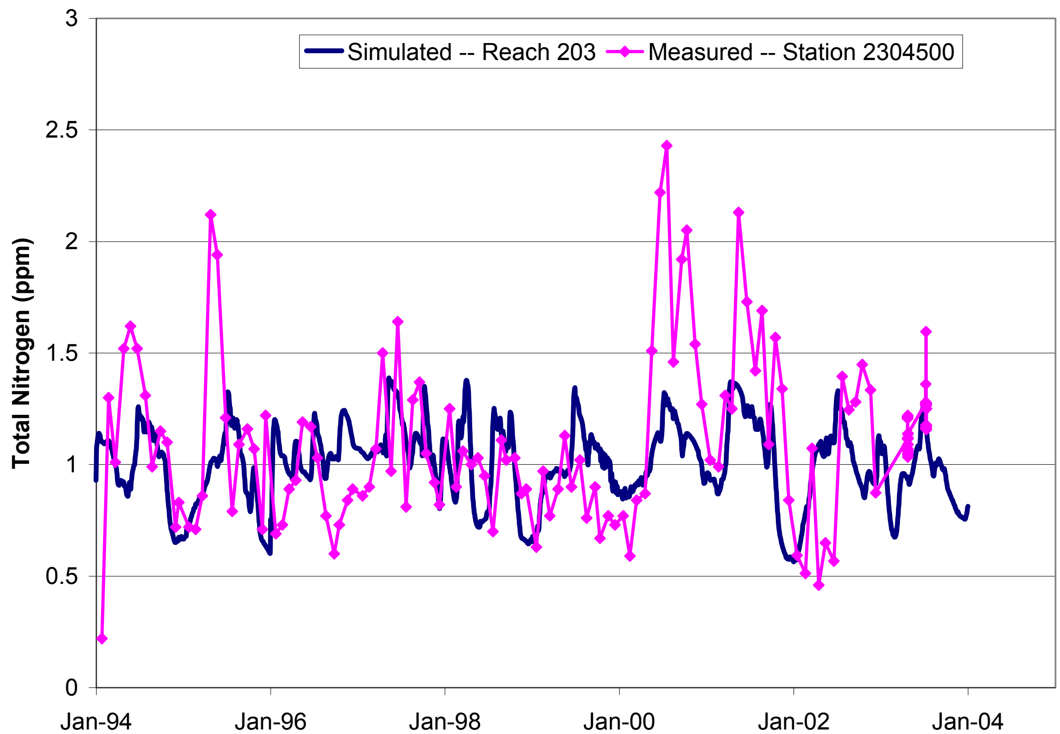

Figure 11. Total N at the Hillsborough River Reservoir Dam

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Volume Control Using Inter-Event Dry Periods

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Abstract

Volume control for stormwater runoff is an option for the reduction of pollutants to meet Total Maximum Daily Load restrictions and an option for the maintenance of a hydrologic balance for watershed and springshed areas. A design approach is presented that requires a specification of sufficient time must be available to achieve a level of effectiveness before the process receives another stormwater event. The design approach uses precipitation data bases and an inter-event dry period. The definition of an inter-event dry period is the time between rainfalls that produce runoff. This approach is favored for situations where there is insufficient transient process performance data, or there is a lack of time related rainfall, evaporation, transpiration and watershed information such as soils and depression storage data to perform a long term simulation.

The results of precipitation analyses using different inter-event dry periods are discussed. The inter-event times used in this paper are those related to water management district specified recovery times and practical infiltration times for shallow ponds. Precipitation from rainfall data stations are illustrated to develop design curves. The design curves that are useful for volume control using off-line (diversion systems) are called **V**olume reduction, **I**nter-event dry period, **V**olume storage (**VIV**) curves. The recovery times used in this paper are 4 and 72 hours. Short one year precipitation data bases are used and compared to longer 15 year data bases. Examples of the use of the **VIV** curves are presented. The implications for development and regulatory rule making are discussed.

Introduction

Why be concerned with volume control? First, the mass of pollutants in stormwater associated with the volume of discharge to ground or surface waters can be reduced if the volume of stormwater can be reduced. Other options to reduce pollution concentrations include the use of structural methods or land use controls. Pollutant concentration reductions may not always be cost effective. Furthermore, the use for recharge or irrigation of excess stormwater can help: (1) save potable waters otherwise used for irrigation, (2) reduce fresh water impacts to estuaries, (3) maintain vegetation cover in an area, (4) maintain micro-climates, (5) reduce salt water intrusion, (6) reduce surface waters that may contribute to flooding, and (7) maintain groundwater recharge. There are

a host of other reasons for the use of stormwater, and direct cost savings is among the most popular, and must be considered.
The common processes that are used for volume control are use of stormwater and infiltration. Typical names given to these processes are irrigation ponds, reuse ponds, exfiltration tanks, infiltration ponds, retention ponds, low impact development (LID) infiltration, rain barrels, pervious pavement, and the like. The infiltration systems above ground are also frequently called off-line retention ponds. The time for recovery of the irrigation pond or infiltration process is related to the time during which there are no rainfall events that produce runoff. The design assumption is frequently stated as one for which the stormwater management process will return to the starting water elevation, control level, or other condition and is called recovery time. The authors of this paper also have experience in the use of stormwater for cooling water purposes whereby the excess water is evaporated (Wanielista, et al, June 2004). In addition, spring flow in the Wekiva basin of Florida has been related to the groundwater levels and in turn these levels have been related to rainfall conditions and the quantity of water recharging an area (Wanielista, et al, January 2005).

Background Information

During the latter part of the 1980s, the promotion of stormwater as a source of irrigation was recognized by the State Department of Environmental Protection and the Saint Johns River Water Management District. Research was complete and published (Wanielista, et al, 1991), and (Wanielista and Yousef, 1991) which provided an understanding of the sizing of on-line ponds and off line detention areas. The Saint Johns District included in its Manual of Practice (as early as 1995), design examples and curves used to the size on-line reuse ponds. These curves are called REV curves, or the **R**ate of irrigation, **E**fficiencies, and **V**olume of pond sizing curves (Saint Johns River, latest version) and specified in their Environmental Resources Permits, Chapter 40C-42 (February, 2005).

With the need to remove treated sewage from direct discharge to surface waters in the late 1980s, priority was given to irrigation of treated sewage, also known as reclaimed water. This practice continues today in spite of the fact that the cost of reclaimed water is higher than the cost of stormwater, as we still have not solved receiving water quality problems. However, as reclaimed water and potable water for irrigation becomes scarcer, and TMDL limitations are enforced, off-line stormwater use systems will become more popular, and their design must be based on basic hydrologic principles and site specific data. Therefore, the efficiency of off-line stormwater systems based on the volume of stormwater generated from a watershed is proposed as one measure to size stormwater facilities to reduce pollution in the discharge and to achieve other societal benefits.

Basic Principles and Data Analyses

Since rainfall is the driving force for rainfall excess or runoff and rainfall events are stochastic in nature, it appears reasonable to describe rainfall and the resulting rainfall excess by probability and statistic descriptors. The basic representation of rainfall showing the inter-event dry period is shown in Figure 1. The inter-event dry period is the minimum dry period between rainfall events that is necessary to return the watershed or the stormwater control to its initial volume condition.

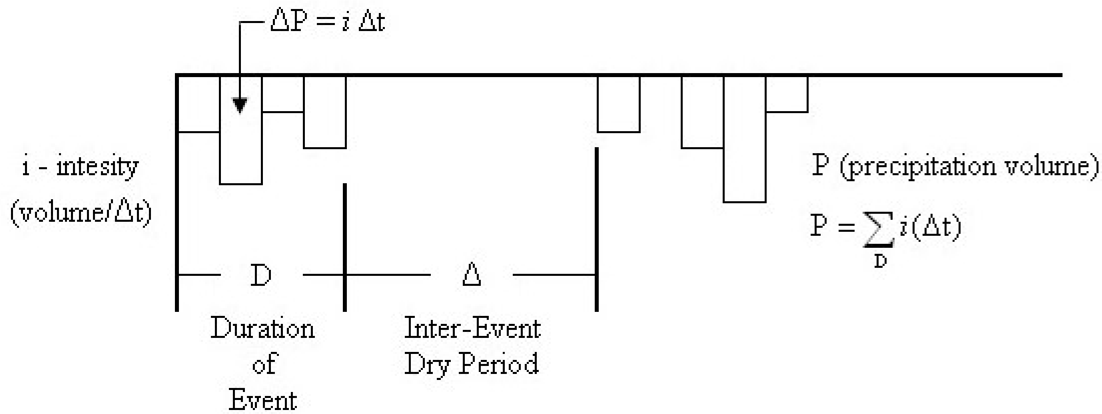


Figure 1. Precipitation Description with Inter-Event Dry Period.

Following the basic principles of Figure 1, an empirical probability distribution for storm event volumes can be developed. Shown in Figure 2 is the histogram of event volumes for 15 years, using an inter-event dry period of 4 hours. This histogram was considered stable in values (within 1%) with a record period of 15 years. After 4 hours, remaining rainwater within a pond or watershed is assumed to either evaporate or infiltrate.

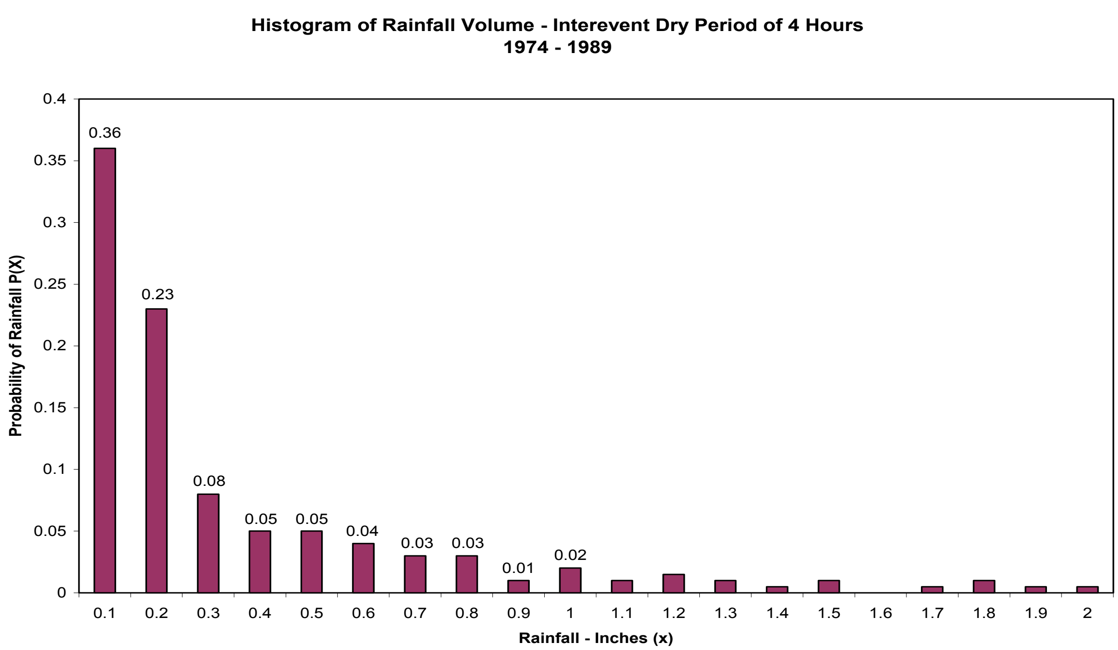


Figure 2. Histogram of Rainfall Volume Given an Inter-Event Dry Period of 4 Hours With number of storms per year = 130.

Specifications and Regulations

Specifications and regulations for volume management should be written based on the precipitation and watershed conditions within an area. Calculation procedures must be developed to estimate the percentage of rainwater that can be diverted and used in the watershed. The basic goals may be to match post equal to pre discharge volume, reduce TMDL, or to recharge a fraction of the rainfall, among other goals. Figure 2 is used to estimate the percentage of rainfall that is not discharged, thus the "credit" for volume control either by abstraction within a watershed or diverted for use by intentional off – line storage.

Watershed Abstraction

How much of the yearly rainfall can be expected in abstraction. Expected values of abstraction can be calculated from empirical or theoretical probability distributions. As an example for the use of an empirical record, using the histogram of Figure 2, the rainfall volumes associated with the rainfalls within the first 0.10 inch is 2.34 inches per year or (.36)(130)(.05). Also, direct summation of all rainfall within the interval over the 15 years of data also produces a similar number (2.33 inches per year).

It is common to store in depression areas within a watershed an abstraction volume of rainfall from each and every rainfall event provided that storage can be recovered before the next event. Low Impact Development (LID) should be guided by this basic principle since many LID methods are based on abstraction. This simple principle of hydrology is applied to two situations, namely the amount of rainfall that does not result in rainfall excess (that volume of runoff not available for surface discharge) and the amount of runoff that can be infiltrated in intentional storage. Thus, the rainfall volume associated with each and every storm event of specific volume can also be stored or infiltrated, not only that volume associated with the event initial watershed abstraction. As an example, the volume abstracted from each and every rainfall event up to and equal to 0.10 inches is calculated as:

$$\text{Volume Abstracted} = \sum_{i}^{\text{AbstractionVol.}} \mathrm{P}(i)_i\, \overline{\mathrm{x}}_i \mathrm{n} + \sum_{i=\text{AbstractionVol.}}^{\infty} \mathrm{P(i)}_i (\text{Abstraction Vol.})(\mathrm{n}) \ \ldots.\text{Equation (1)}$$

> Where the first term is the Expected Value of the abstraction volume up to the abstraction depth, and the second term the abstraction volume for all storm events greater than or equal to the abstraction depth.

And for the example data from Figure 2:

> Volume Abstracted = (.36)(130)(.05) + (1-.36)(130)(.10) = 10.66 inches.

The implication of this calculation for design and rule making indicates that for watersheds with at most 0.10 inches of abstraction, the storage volume in this initial abstraction is at least 20% of the volume of the yearly rainfall of 51 inches (10.66/51). Thus at most 80% of the yearly rainfall on the average for the particular assumptions of abstraction is at least 20% of the volume of the yearly rainfall of 51 inches (10.66/51). Thus at most 80% of the yearly rainfall on the average for the particular assumptions of abstraction and inter-event time will result in rainfall excess. This translation of rainfall to rainfall excess was also calculated by another simulation method as 78.2% by Harper and Baker (2003) for completely impervious watersheds.

Off-Line Retention

Another use of the conditional distributions with inter-event dry periods is to determine the rainfall excess volume reduction (runoff) relative to rainfall when intentional (designed) off-line retention is provided. The initial volume of rainfall that results in rainfall excess is diverted into an area for infiltration or use of the rainfall excess. In this case the target level for pollution control or mass of water reduction may be the controlling design criteria of 1.0 inch or 1.5 inch or other regulated diversion depth. Using Equation (1) and the empirical distribution of Figure 2, the volume of rainfall not discharge associated with a diversion rainfall of 0.5 inches is calculated as:

$$\text{Volume Diverted} = (.36)(130)(.05) + (.23)(130)(.15) + (.08)(130)(.25) + (.05)(130)(.35) + (.05)(130)(.45) + (1-.77)(130)(0.5) = 29.6 \text{ in}$$

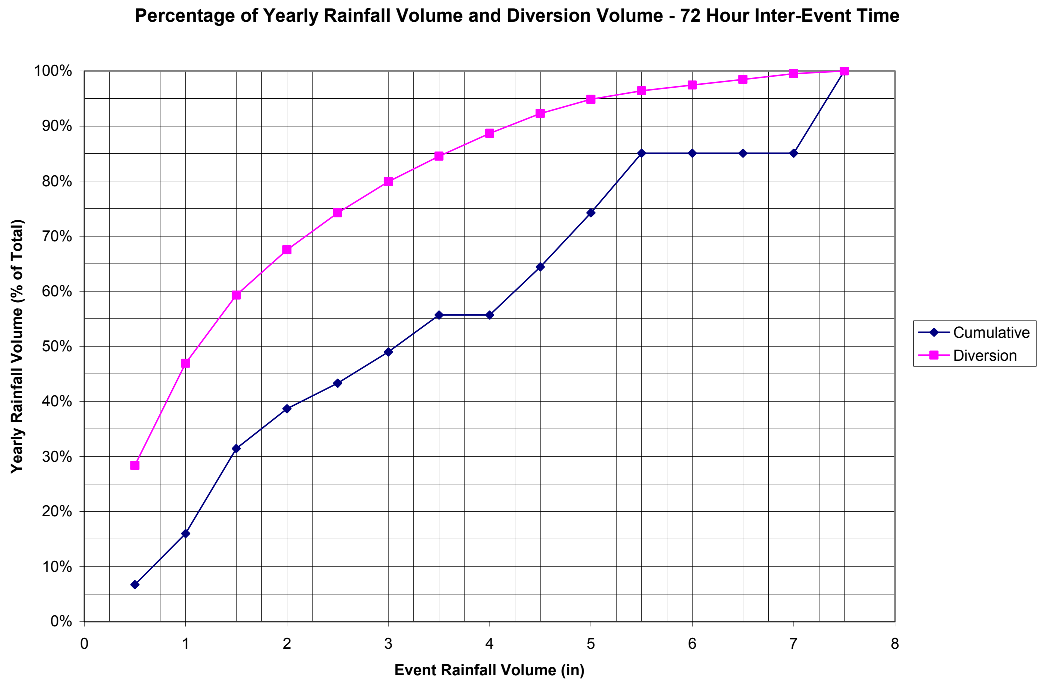
For this example calculation and for a 0.5 inch diversion of each and every rainfall, the percent of rainfall not discharged in an average year is 58% (29.6/51), provided there exists a recovery time of 4 hours.

Similar calculations for 1.0 inch diversion shows the percentage of rainfall per year not discharged is about 80%. For a pollution control objective of 80% reduction on the average, the diversion for off-line retention of the runoff up to 1 inch of each and every storm event results in a mass of water and pollutant reduction of 80% of rainfall in a year using the rainfall which is characterized by the probability distribution of Figure 2. This distribution reflects the long term rainfall and is most likely representative of the average.

Since not all off-line retention systems can recover to their design control elevations in four hours, other conditional probability distributions must be developed. One recovery time which appears in the literature and regulation very frequently is 72 hours. This recovery time can be the time for infiltration in deeper ponds or the time for reuse of the water. For a 72 hour inter-event dry period, and for the central Florida area, a 3 inch diversion event volume is necessary to capture 80% of the annual rainfall.

The Wet Year and VIV Curves

An opportunity to compare **VIV** curves from long term simulations to those using one wet year presented itself with the hurricanes of 2004. Two locations were used, Apopka and Corner Lake with yearly rainfall of 48 inches and 64 inches, respectively. Apopka had a near average rainfall volume but three hurricanes were within the area, resulting in more rainfall for the hurricane days. The **VIV** curves for inter-event dry periods of 72 hours are shown in Figure 3. The Apopka curve with 48 inches of yearly rainfall shows an 80% efficiency occurring at 3 inches while the 64 inch yearly rainfall volume at Michaels Dam produced an 80% efficiency at 4.5 inches and 65% efficiency at 3 inches.



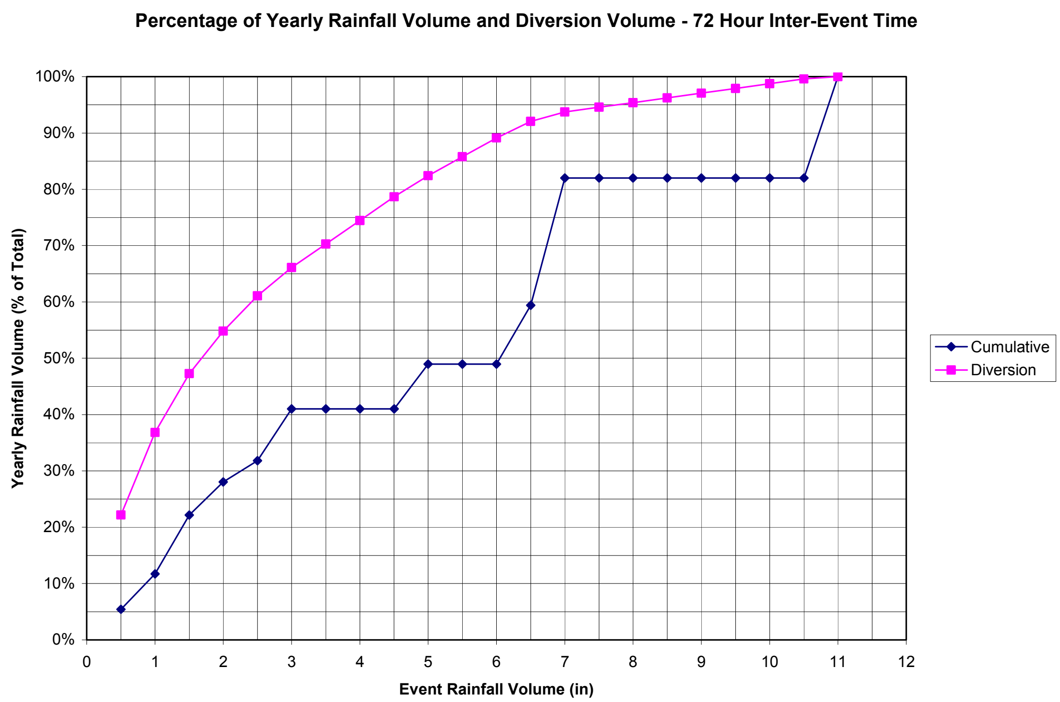


Figure 3. **VIV Curves** Cumulative (No diversion) and Diversion Graphs for a 72 hour Inter-Event Dry Period & Year 2004 in Central Florida (top curve for the Apopka area and the bottom for Corner Lake, both in Orange County, Florida).

Comparisons are also presented for a 4 hour recovery time and an example is shown in Figure 4. Annual rainfall was about 64 inches. The one inch diversion rule removed about 70% or the yearly rainfall. The removal on the "average" is about 80%.

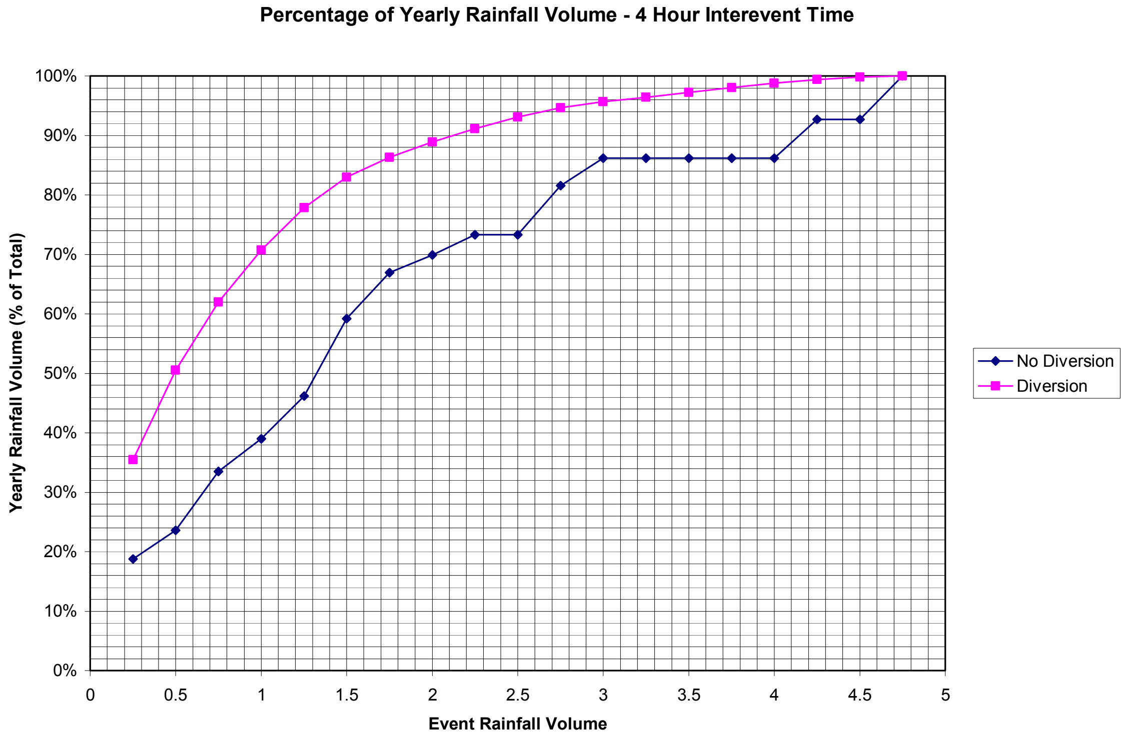
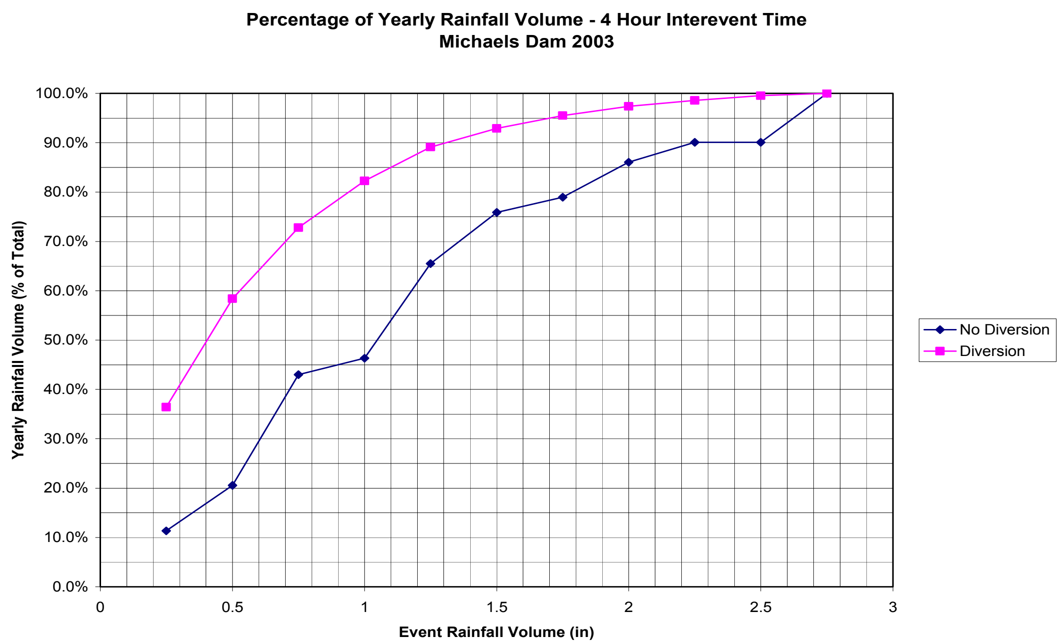


Figure 4. **VIV Curves** or Cumulative (No diversion) and Diversion Graphs for a 4 hour Inter-Event Dry Period & Year 2004 near the University of Central Florida. (data from Orange County Florida Stormwater Management Division)

Using One "Average" Year and VIV Curves

In 2003, hourly data were recorded near the UCF campus at Michaels Dam. The total yearly rainfall was about 53 inches. The long term average for the area is about 50 inches. **VIV** curves were developed for inter-event dry periods of 4 and 72 hours. The data base was reviewed for isolated one hundredth inch rainfalls, which were removed. Visual observations of the "tipping bucket" in "high dew" conditions indicated the isolated 0.01 recordings. The **VIV** results are show in Figure 5. For an inter-event dry period of 4 hours, the one inch diversion criteria will result in at least an 80% reduction in the rainfall excess and is approximately equal to result using the long term or "average" record. However, the inter-event 72 hour **VIV** curve is better approximated by a longer term hourly rainfall data record because of the differences in the event volume associated with short and long term records.



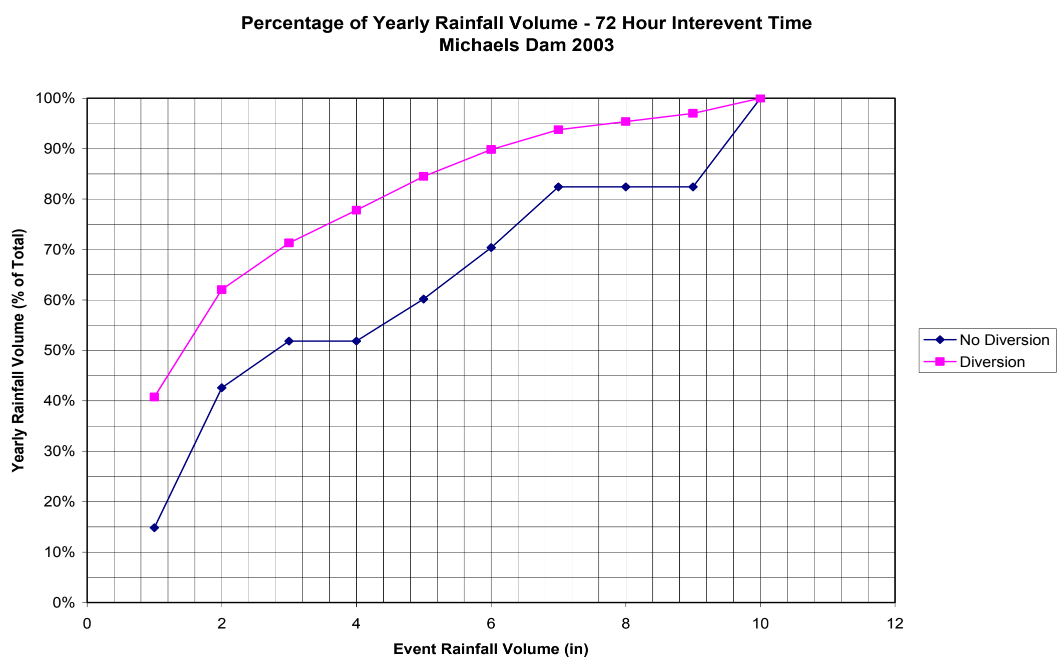


Figure 5. One Year of "Average" Rainfall and **VIV** curves with 4 hour (top) and a 72 hour (bottom) inter-event dry periods. For Michaels Dam near the UCF campus.

Summary and Conclusions

Volume management should be part of the water management regulations for the State of Florida in areas where justified. There are many problems, ranging from pollution control to recharge that can be solved using volume control. The **VIV** curves are useful to size LID infiltration areas, stormwater use ponds, and regional infiltration areas.

A methodology using the frequency distributions for rainfall was presented that was used to assess initial abstraction and diversion efficiencies. It was shown for a .10 inch initial abstraction for rainfalls with an inter-event dry period of 4 hours, the % yearly amount of rainfall not going to rainfall excess can be around 20%.

The development and use of the **VIV** curves to estimate yearly **V**olume of rainfall not discharged, based on an **I**nter-Event dry period, and **V**olume of event rainfall was presented. From previous research, about 15 years of rainfall hourly data are needed to stabilize the results. From historical data for a region, histograms and **VIV** curves are developed using spreadsheets. The current 1 inch regulation for the diversion of stormwater was developed based on **VIV** curves, and a 4 hour inter-event dry period.

The year 2004 was a wet year with 3 hurricanes and a yearly rainfall volume of 64 inches, at Michaels Dam near the campus of the University of Central Florida. While in the same hurricane year, in Apopka the yearly rainfall was about 48 inches. Thus, the hourly data provided an opportunity to examine the effects of a wet year as well as an average year on the **VIV** curves, and to illustrate the changes between a 4 hour and a 72 hour inter-event dry period. There appeared to be no changes in the 80% diversion volume and 4 hour inter-event dry period when the yearly one year rainfall was about the average, while for the wet year of 64 inches, as expected, the efficiency decreased to about 65% for the 72 hour inter-Event time.

To achieve volume balance, previously developed **VIV** curves for a region can be used; however, it is recommended to construct the graphs using the local rainfall record. For regions which do not have long term hourly precipitation records, a precipitation record with an average yearly rainfall close to the long term average yearly values can be used to approximate a **VIV** curve using a 4 hour inter-event period. However, it is still recommended to use long term (15 years or more) records of hourly precipitation where data are available.

Acknowledgements

The financial support of and technical advice from individuals at the State Department of Environmental Regulation, the State Department of Transportation, the State Department of Community Affairs, and the Saint Johns River Water Management District are appreciated. Most recently, the Orange County Florida Stormwater Management Division has provided rainfall data and advice. The continued support of all agencies in the State for this research into the use of stormwater over the past 15 years is acknowledged and is certainly paying dividends to all involved.

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Restoration of Lake Seminole in Pinellas County Florida: the Scientific, Programmatic and Public Policy Aspects of Urban Lake Restoration

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Abstract

Lake Seminole, located at the southwestern tip of the Pinellas peninsula, is a medium sized lake with a highly urbanized contributing watershed. The lake and an associated bypass canal discharge into Long Bayou, a valuable estuary in Boca Ciega/Tampa Bay. The lake was created in the late 1940's by the impoundment of an arm of Long Bayou and the construction of Park Boulevard. The lake was originally brackish; however, in time, the impounded waters were converted to a freshwater system that supported an excellent bass fishery. Though never pristine, Lake Seminole has in recent years degraded from a eutrophic to a hypereutrophic system and now exhibits many of the problems typical of urban lakes including the accumulation of nutrient-rich muck sediments, reduced coverage of beneficial littoral and submerged vegetation, and elevated chlorophyll concentrations.

In 1992, the Southwest Florida Water Management District conducted a lake assessment that highlighted the increasing eutrophication problems in the lake and made numerous recommendations for lake restoration. Based on this study and other concerns, the District and Pinellas County developed a cooperatively funded plan to restore the lake. The first element in this process was the development of the Lake Seminole Watershed Management Plan (LSWMP), developed by Post, Buckley, Schuh and Jernigan (PBS&J) Inc., for Pinellas County and published in its final form in September 2001. The plan recommended structural and non-structural Best Management Practices (BMPs) that are now being implemented. This paper details the scientific, programmatic, and public policy basis for the most comprehensive of these BMPs, the Lake Seminole Stormwater Pollution Reduction Project (LSPR). This project began in March of 2003 with the selection of Environmental Research and Design (ERD) as the project consultant and is in its final design phase.

Introduction

Lake Seminole is the second largest lake in Pinellas County, Florida. It has a total surface area of 2.768 square kilometers (km^2) (684 acres) and a contributing, highly urban drainage area of 14.084 km^2 (3,480 acres). The lake was created in 1940 when an arm of Long Bayou was impounded during the construction of Park Boulevard. At that time, Long Bayou was a brackish section of Boca Ciega Bay, which is one of the five major segments of Tampa Bay. Lake Seminole's original drainage area was 28.7 km^2 (11 square miles); however, in the late 1970s, in a response to heavy flooding in the watershed, Pinellas County constructed a flood control canal that allowed the majority of the Long Bayou watershed drainage to bypass the lake. The construction of the Lake Seminole Bypass Canal (LSBC) significantly altered the flow characteristics of the lake. Prior to the construction of the bypass canal and the rapid urbanization of the lake's watershed, Lake Seminole was a productive freshwater sports fishery. A little over 10 years after the completion of the bypass canal, the lake was exhibiting all the classic signs of eutrophic lakes, declining water clarity, nuisance vegetation and a reduced game fish fishery.

In January 1989, responding to a high level of public concern over the degraded conditions of the lake, the Pinellas County Board of County Commissioners passed Resolution 89-13 that called for a cooperative, long term lake management program for Lake Seminole. Soon after, the Lake Seminole Advisory Committee (LSAC) was formed to begin the process of developing a lake management plan. The LSAC was composed of representatives from Pinellas County, the then Florida Department of Natural Resources, Florida Department of Environmental Regulation, Florida Game and Fresh Water Fish Commissions, the Southwest Florida Water Management District (SWFWMD), the Cities of Largo and Seminole and various home owner and business groups and individuals.

One of the first group initiatives was a cooperative project that produced the *Lake Seminole Diagnostic Feasibility Study Part I* (SWFWMD, 1992) and the corresponding *Seminole Diagnostic Feasibility Study Part II Water Quality Modeling* (Dames and Moore, 1992). The County and SWFWMD accomplished these efforts cooperatively. Based on the study and model, the County and SWFWMD crafted an agreement for short and long term management activities for Lake Seminole. These cooperative efforts resulted in the *Lake Seminole Watershed Management Plan* (PBS&J, 2001), several "short term" restoration projects completed between 1998 and 2002 and four LSWMP implementation projects that were either constructed, planned or at the agreement stage by January of 2005. The LSWMP implementation projects include both structural and non-structural best management practices (BMPs) and recommended actions.

The Lake Seminole example demonstrates some important management issues that may have universal application to urban lake management. These include: (1) need for strong interagency (federal, state, local government) cooperation and commitment to funding; (2) the need for strong public awareness and support; (3) the need for new BMP approaches that are well supported by research and are not heavily land dependent; (4) the ability of project managers and regulators to work together with a full understanding of the overall goals; and, (5) the need to fully understand the long-term nature of urban lake restoration. This paper discusses these elements and how they are being addressed through the implementation of the LSWMP.

The Lake Seminole and Long Bayou Watershed

Lake Seminole is located on the southwest coast of Florida (see Figure 1), in the Long Bayou Sub-watershed of the Tampa Bay watershed. The Long Bayou watershed area is 58.793 km^2 (22.7 square miles) with about half of the watershed (28.7 km^2) draining through the Lake Seminole Bypass Canal or through Lake Seminole to the bayou.

Long Bayou has the land use typical of a densely populated, urbanized watershed. As seen in Table 1, the primary land use (45%) is high density residential and the other primarily urban land use categories (commercial, industrial, institutional, transportation, other residential) compose another 25%. Very little open land (3.5%) remains. The land use reality of the watershed drives the types of restoration and water quality improvement options available.

The primary source of pollution to Lake Seminole and the Bypass Canal (and eventually to Boca Ciega Bay) is non-point source pollution from this highly urbanized watershed. The pollution reduction strategy for the watershed includes the maximum use of open space for stormwater management and the use of chemical treatment where space prevents traditional treatment methods. The Bypass Canal offers an area where water draining from the upper Long Bayou Watershed can be treated prior to passing along the eastern side of Lake Seminole into the Bayou. A portion of the treated water will be used for Lake Seminole restoration and the remaining will pass down the Bypass Canal to Long Bayou.

Table 6 **Land Uses within the Long BayouWatershed** (acres, and percentages of total):

RESIDENTIAL HIGH DENSITY	6536.2622	26,451,311.90	45.07%
COMMERCIAL AND SERVICES	1078.5667	4,364,804.12	7.44%
INDUSTRIAL	991.2254	4,011,346.46	6.84%
BAYS AND ESTUARIES	857.3327	3,469,501.98	5.91%
RECREATIONAL	677.2572	2,740,762.36	4.67%
LAKES	668.6781	2,706,043.98	4.61%
INSTITUTIONAL	578.0629	2,339,337.32	3.99%
OPEN LAND	502.7464	2,034,542.29	3.47%
TRANSPORTATION	415.7077	1,682,309.20	2.87%
RESIDENTIAL MED DENSITY 2->5 DWELLING UNIT	323.9821	1,311,108.91	2.23%
RESERVOIRS	305.6389	1,236,876.62	2.11%
RESIDENTIAL LOW DENSITY < 2 DWELLING UNITS	266.5345	1,078,626.74	1.84%
PINE FLATWOODS	235.9539	954,871.46	1.63%
HARDWOOD CONIFER MIXED	191.6533	775,593.31	1.32%
WETLAND FORESTED MIXED	176.4321	713,995.30	1.22%
UTILITIES	113.8942	460,913.43	0.79%
OTHER	581.3124	2,352,487.57	4.01%
LAND USE TOTAL (ACREAGE AND %)	14501.2407	58,684,432.93	100.00%
			0.00%

The Lake Seminole Stormwater Pollution Removal Approach

The early BMPs used for the project were heavily land dependent and provided a limited amount of pollution reduction. The LSWMP proposed the use of chemical treatment of stormwater as an alternative approach to physical and biological treatment. By adding chemical treatment to the mix of available BMPs, managers were able to develop a combination of approaches that could meet the pollution reduction goals[2]for the lake as well as providing an overall decrease in pollution loads entering Long Bayou.

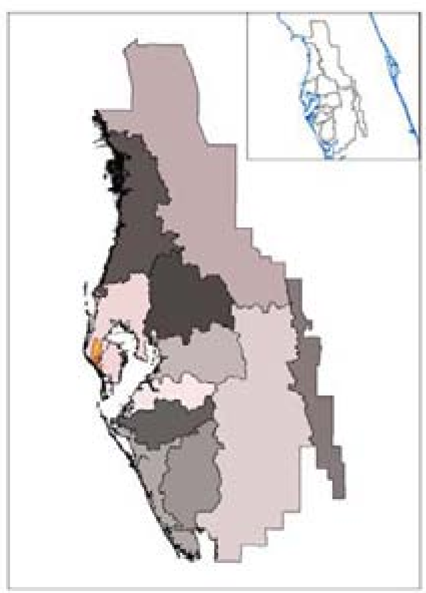

Figure 12. Southwest Florida Comprehensive Watershed view with Long Bayou, a sub-watershed of Tampa Bay and Anclote River Watershed, shown.

The Lake Seminole Stormwater Pollution Removal (LSPR) project is the implementation of Structural Components1[3] and 2[4] in the LSWMP. The general approach was to capture and treat the stormwater from five of the twelve basins that drain to Lake Seminole and to divert and treat a percentage of the flow of the Lake Seminole Bypass Canal (LSBC) for both lake treatment and water quality improvements to Long Bayou. The five basins shown in Figure 2 are responsible for an estimated 74% of the combined pollutant load reaching Lake Seminole. Of the five basins, only Basin 6 receives any significant stormwater treatment. The LSPR approach was based on a goal of a 50% reduction of stormwater phosphorus load and the use of lake flushing by treated water from the LSBC.

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[2] Reduce lake chlorophyll a to below 30 μg/L, stormwater phosphorous load by 50% and lake TSI to below 65.
[3] Construct Enhanced Regional Stormwater Treatment Facilities in priority subbasins.
[4] Divert Seminole Bypass Canal flows to improve lake flushing and dilution.

Both elements in this approach required chemical treatment of stormwater and, as required by Florida Department of Environmental Protection (FDEP) guidelines, the capture of any resulting stormwater residuals due to chemical flocculation. The chemical treatment method chosen for the project was aluminum sulfate injection.

Lake Seminole is a balanced to slightly phosphorus limited lake. Therefore by reducing phosphorus in the lake, the lake can be driven to a phosphorus-limited system that can be further managed through an overall approach that concentrates on phosphorus reduction to limit algal growth. The decision in the later stages of the LSWMP development phase to manage the lake by managing external phosphorus stormwater pollutant loading resulted in final strategy for the lakes restoration. This strategy has three primary elements: (1) reduction of phosphorus loading from stormwater, (2) reduction sources of internal nutrient cycling and (3) reduction of the lake hydrologic residence time. This approach was verified by the lake watershed model (a combined DYNHYD-WASP model and the Linked Watershed-Waterbody model) which indicated that a trophic state index (TSI) goal of 65 or less could be obtained by enhanced stormwater treatment, sediment removal and the diversion of treated Seminole Bypass Canal flows through Lake Seminole. The LSPR addresses the first of these management initiatives.

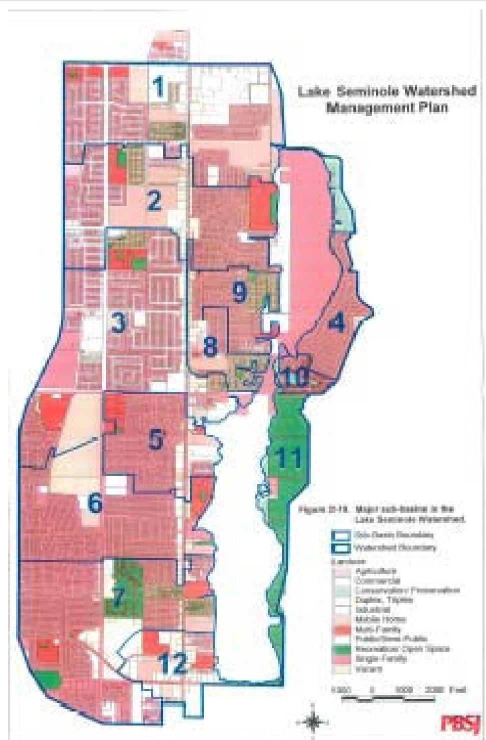

Figure 13 Lake Seminole contributing drainage basins. Basins 1, 2, 3, 5, 6 and 7 contain the majority surface area of the contributing surface area (LSWMP, 2001 p)

The general approach developed by the LSWMP consultant (PBS&J) and approved by the LSAC formed the basis of the initial evaluation effort conducted by ERD in developing the conceptual plan for the LSPR project. Pinellas County, SWFWMD, the Cities of Seminole and Largo, and their consultant, ERD, developed a conceptual design that incorporates the primary LSWMP recommendations for Structural Components 1 and 2. Leading up to the conceptual design, the County and ERD conducted storm event sampling and ERD conducted chemical tests and ran model simulations to determine the system designs for the five Lake Seminole enhanced stormwater regional facilities and the LSPC facility. The conceptual plans designed by ERD for these facilities are shown in Figures 3, 4, 5, 6, 7 and 8.

The Lake Seminole Diagnostic Feasibility Study (SWFWMD, 1992) listed the estimated annual mass total phosphorus load for all sub-basins to be 2,425 kg and the load from the priority sub-basins (sub-basins1,2,3,6 and 7) to be 1,797 kg. To achieve a 50% reduction in load by treating only the priority sub-basins, a BMP treatment efficiency of 90% (typical alum treatment efficiency) and the treatment of 75% of the stormwater flow from the priority sub-basins were necessary. Additionally, a required treatment time of 3 hours at the design peak discharge rate was determined (2 hours settling time plus 1 hour safety factor). The 3-hour treatment time and selected design storm (1.73-inch) established the size of the settling pond required for each facility. This in turn drove the requirements in terms of required land area and type facility for the project.

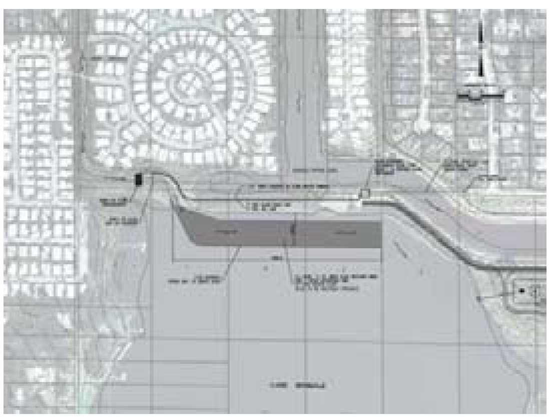

Figure 13. Alum injection and in-lake flocculent settling area for Sub-basin 1.

Figure 13 Alum injection and in-lake flocculent settling area for Sub-basin 2.

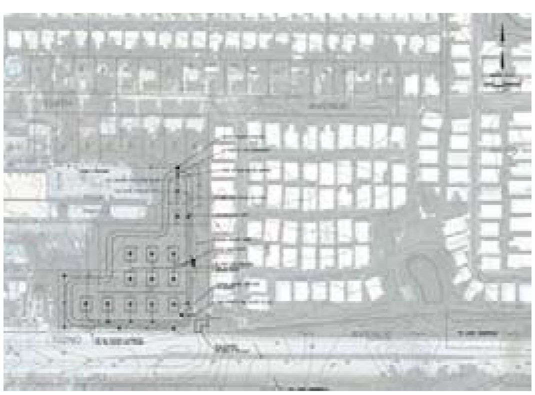

Figure 13. Alum injection and pond flocculent settling area for Sub-basin 3.

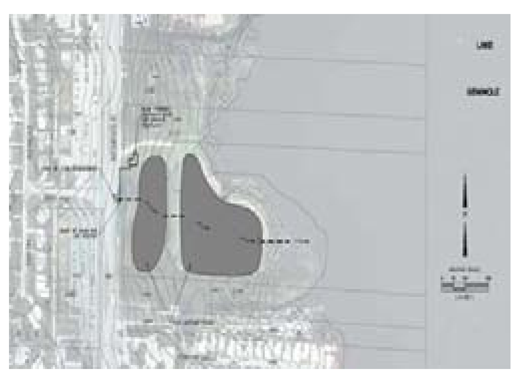

Figure 13 Alum injection and pond flocculent settling area for Sub-basin 6.

The conceptual designs for the chemical stormwater treatment systems included three different approaches. The first is the reasonably well-accepted approach of alum injection in the stormwater stream and flocculent collection in a settling pond with flocculent removal through a series of sumps with feeds to the sanitary sewer system. This is the approach shown in Figures 5 and 6 for sub-basin 3 and 6.

The most radical approach will be employed for sub-basins 1, 2 and 7. The major limiting factor in the design for the LSPR was the availability of land. To overcome this limitation, we decided to consider construction of flocculent removal systems within the lake. These in-lake "ponds" will be dredged or mechanically excavated areas that are enclosed on the lakeside by a corrugated aluminum barrier and open on the end to allow boat traffic. As is seen in Figures 3, 4 and 7, the in-lake ponds are attached to drainage canals. The canals have docks and support boat traffic and must continue to do so. The pond is designed so that flocculent material will be removed prior to the flow reaching the pond opening. This approach has been reviewed by FDEP and, for the urban lake restoration conditions established for Lake Seminole, the approach was approved. The project is presently in the permit approval stage; however, the County was issued a modification to its existing National Pollutant Discharge Elimination System (NPDES) MS4 permit.

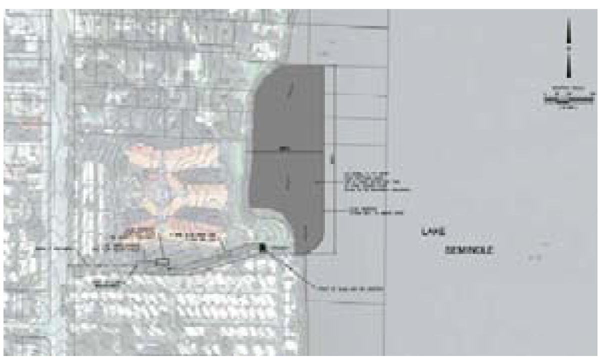

Figure 13 Alum injection and in-lake flocculent settling area for Sub-basin 7.

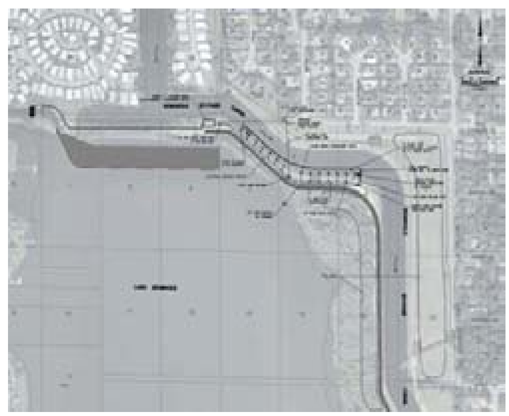

Figure 13 Alum injection and in-canal flocculent settling area for Seminole Bypass Canal.

The final design approach is for the LSBC design. For this design, there is adequate space in the canal itself to treat the flow. The pond is built in the canal by dredging a sump area and constructing a seawall and an outlet structure. Flocculent material is removed in the same manner as for sub-basin 3 and the flow through the system is created by a pump in the control structure that pumps the canal water at an elevation of about 3 feet National Geodetic Vertical Datum (NGVD) to the lake surface elevation of about 5 feet NGVD. A duplex submersible water pump station capable of between 10 and 15 cubic feet per second is specified. The facility will be able to treat and pump water into the lake or back into the canal.

Expected Results

Tables 2, 3 and 4 are based on information supplied in the conceptual design report (ERD, 2004). Table 2 provides the details of the five proposed facilities for the Lake Seminole Regional Stormwater Treatment System. The pond volume, alum required, alum flocculent produced and flocculent trap surface area required are shown for the priority basins. The volume treated, alum requirements, and annual flocculent volumes are shown for the LSBC based on a pump rate of 10.4 cubic feet per second (cfs). The length of the LSBC flocculent capture area is based on the peak pump rate of 15 cfs.

Table 7 Discharge, volume and chemical treatment data for the Lake Seminole and Lake Seminole Bypass Canal Regional Stormwater Treatment Systems

SUB-BASIN	PEAK DISCHARGE (Q_p) for 1.73 inch storm (cfs)	RUNOFF VOLUME TREATED (ac-ft/yr)	ANNUAL ALUM REQUIRED (gal)	ANNUAL FLOC PRODUCED (ft^3)	POND VOLUME 3-hr D_t at Q_p (ac-ft)
1	85	446	26,760	38,900	21.2
2	125	554	33,240	48,300	31.0
3	119	611	36,660	53,300	29.5
6	68	296	17,760	25,800	16.9
7	174	687	41,220	59,900	43.1
TOTAL		**2,594**	**155,640**	**226,200**	
LSBC	**15**	7,544	452,000	657,200	3.7

ERD conducted storm event and base flow measurements at the basin outflow points (canals and pipes entering lake) to determine pollutant concentrations and used a hydrologic model to determine water volumes. To this was applied the efficiency of a chemical (alum) injection stormwater treatment BMP and the percent of the basin treated to determine the pollutant load removal estimate for total phosphorus, nitrogen and suspended solids. The results of this analysis are shown in Table 3.

Table 8 Expected non-point source annual pollutant removal for total phosphorus (TP), total nitrogen (TN), and total suspended solids (TSS) for priority basins.

SUB-BASIN	TOTAL P		TOTAL N		TSS	
	EXISTING LOAD (kg/yr)	REMOVED LOAD (kg/yr)	EXISTING LOAD (kg/yr)	REMOVED LOAD (kg/yr)	EXISTING LOAD (kg/yr)	REMOVED LOAD (kg/yr)
1	113	76	918	241	19599	12494
2	118	80	1208	317	10390	6624
3	290	196	1048	275	13770	8778
6	65	44	625	164	5310	3385
7	177	119	1197	314	29597	18868
TOTAL	763	515	4996	1311	78666	50150

The pollutant load estimate for the LSBC was determined from analysis of Pinellas County ambient sampling data and flow data for the canal (Ulmerton Road site). The load estimate was also confirmed by comparison to NPS pollutant load model results. To this was applied to percent of canal flow diverted and the efficiency of the BMP. The results are shown in Table 4.

Table 9 Expected Non-point source pollutant removal for total phosphorus (TP), total nitrogen (TN), and total suspended solids (TSS) for Lake Seminole Bypass Canal Basin.

BYPASS CANAL	TOTAL P	TOTAL N	TSS
LOAD (kg/yr)	3191	39922	479713
% DIVERTED	31.48	31.48	31.48
LOAD DIVERTED (kg/yr)	1,004.67	12,569.24	151,035.22
% REMOVED	90.00	35.00	85.00
LOAD REMOVED	904.20	4,399.23	128,379.94

Discussion

As mentioned previously, the Lake Seminole example demonstrates important management issues that may have universal application to urban lake management. The first of these, the need for strong interagency cooperation and commitment to funding was demonstrated through the long history of the project and the enduring commitment showed by Pinellas County, the SWFWMD, and the Cities of Largo and Seminole. Although many of the actors changed, the resolve by these agencies continues and the result of their efforts is now shown in the completed restoration plan and the significant financial commitment made by these agencies. As important as the commitment by agencies, is the need for strong public awareness and support. Although some mistakes in this area were made, the current approach being taken by Pinellas County to advise strategic citizen groups of the project and to garner their support will be critical to the success of the project. The selection of ERD by the County and District and the County's, the District's, and ERD's individual commitments of scientists and analytical resources to the project has resulted in a novel but manageable BMP approach that is well supported by research and is not heavily land dependent. Finally, but certainly not the least of concerns is the "permitability" of the approach. Early in the planning effort, the proposed alternatives were discussed with representatives of the FDEP's Department of Water Resource Management. An acceptable approach that would allow the restoration to proceed while protecting the general environment was determined. These discussions will continue as the project moves fully into the permit stage; however, we start with a framework of understanding and, as stated earlier, an approach that includes a modification of the current County MS4 (NPDES) permit that will allow the use of chemical treatment and flocculent recovery as specified in the conceptual plan.

An element of any project the size of the Lake Seminole restoration project requires a long-term vision. The County and Southwest Florida Water Management District's

Pinellas and Anclote Basin Board established this vision in 1993 with the signing of the *Lake Seminole Restoration Conceptual Management Plan* (Master Agreement) agreement. The plan established by this agreement included the commitment of $10,000,000 by the two parties and a series of short term and long term projects. Key to the long-term vision was the requirement for the development of the LSWMP and the inclusion in all responsible local governments and agencies in effecting the plan. The Master Agreement ended in December of 2004; however, the vision is now established by the LSWMP and is being implemented by the LSPR projects and other future planned projects.

Acknowledgements

The authors want to acknowledge Mark Flock, Pinellas County Department of Environmental Management, Mr. Eric Livingston, Mr. Karl Kurkla and Mr. David Worley, Florida Department of Environmental Protection, the Pinellas County Board of County Commissioners, and the Pinellas and Anclote River Basin Board of the Southwest Florida Water Management District, for their support for the project.

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Southwest Florida Water Management District Watershed Management Program – The Data Piece and Tools to Manage

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Abstract

Five major elements make up the District's Watershed Management Program (WMP): Topographic Information, Watershed Evaluation, Watershed Management Plan, Implementation of Best Management Practices (BMPs), and Maintenance of Watershed Parameters and Models. WMP elements are performed as issues arise to address the District's areas of responsibility, water quality, water supply natural systems and flood protection, using a comprehensive watershed approach. As a strategy in the District's Comprehensive Watershed Management (CWM) initiative the WMP provides a method to implementation the CWM plans. The District's Watershed Management Program Geodatabase has been design to provide for data management within the Geographic Information System (GIS). This presentation will discuss the format and tools developed to assist the District in the management and maintenance of the Geodatabase.

Water Quality Friendly Field Operations Yard

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Abstract

Clean Water Services (the District) is the regional stormwater and sanitary sewer agency for Washington County, Oregon. This year, the District constructed a new field operations yard. One of the project goals was to incorporate innovative erosion control and stormwater treatment techniques, both during construction and for the long term. The field operations yard was designed to be a demonstration project so that contractors, engineers, plan review staff, and the public could see these techniques in action.

During construction, innovative erosion control techniques were employed. These included: two large settling basins to receive storm runoff; after initial site grading, immediately covering the site with a layer of gravel and leaving no more than 5% of the area uncovered at any time; a 187m sediment filter dam along the entire downstream side of the property; treatment swales with gravel filter dams; and continuous monitoring of storm runoff. Long term innovative stormwater treatment measures include an 1,063m^2 eco-roof, "scupper gardens" to treat roof runoff, a porous concrete employee parking lot, a "green street" with gently sloping vegetated swales replacing curbs and gutters, vegetated treatment and conveyance swales instead of catch basins and storm pipes, and a 15.2m wide, 187m long water quality facility along the rear.

Introduction

Clean Water Services (the District) is a County Special Service District operating in the urban area of Washington County, Oregon. The District is responsible for the public storm water and sanitary sewer systems, as well as administering the erosion control and water quality programs in the urban areas. There are 12 cities and a large unincorporated area within the District boundary.

In 2002, the Field Operations Division began the construction of a permanent field yard. The site is 5 acres in size with an 2,392 m^2 administration office, 1,993 m^2 of indoor vehicle parking, and 1,727 m^2 of outdoor carport parking. The $9 million facility houses 58 employees and more than 70 pieces of equipment including street sweepers, dump trucks, crew vehicles, sewer cleaning equipment, and excavators.

Since the site was designed to be a demonstration project for contractors, engineers, plan review staff, and the public to see innovative stormwater management techniques in action, it was essential that the construction be done to the highest standards relative to erosion control and water quality. This included techniques used during construction as well as the design of the finished product.

Overview of Field Yard

Another goal was to show that the innovative techniques could be approved through the normal development permitting process. Most of the innovative techniques used at the field yard project were not initially allowed by the local jurisdiction responsible for permitting the project. Though each technique had to go through a special process to be approved, the project demonstrated that through the submittal of proper supporting documentation, innovative techniques can meet the permitting requirements of a local jurisdiction.

Construction Phase

The project site was particularly sensitive because of its location directly adjacent to, and upstream of, the Tualatin Hills Regional Nature Park. All runoff from the project site goes directly to this pristine area, so every effort was made to prevent erosion from leaving the site. Our objectives during construction included:

1. Prevent erosion – The District rules for developers and the District's own erosion control program stress erosion prevention. The local soils are very high in clay which will remain in suspension for days before settling out, and once the soil has eroded it is very difficult to prevent it from leaving the construction site. Therefore, most of the measures used were designed to keep soil in place.

2. Control any sediment erosion that does occur – Some soil will erode despite efforts to keep it in place, so measures are needed to capture the soils before they leave the site. Given the clay soil and the goal of not allowing sediment to leave the site, simple erosion control fences would be inadequate.

3. Continuously monitor performance – The District was fortunate that the corporate philosophy of its prime contractor, Baugh-Skanska, stressed environmental protection during construction. For example, as part of their normal operating procedure, they made checks of their erosion control measures before, during, and after storm events. Additionally, runoff samples were taken during and after storms to monitor performance.

4. Demonstrate innovative techniques

To accomplish these goals, the following techniques were used:

1. Settling Basins - The initial step was to construct two large settling basins to receive all the storm runoff from the site. Each pond was 15.6m x 50m x 1.6m deep. The ponds were hydroseeded to protect the banks. The ponds allowed sediment in suspension time to settle before runoff water left the site.

2. Flocculent - During rainy periods an organic flocculent was added to the ponds to speed the settling process. Storm-Klear Gel-Floc chitosan biopolymer for gravity settling of erosion sediment was used. The local supplier was CSI Geosynthetics, Vancouver, WA (360) 910-4800.

3. Gravel cover - After the initial grading, the entire site was immediately covered with a layer of gravel to protect the soils from eroding. No more than 5% of the site was uncovered at any time.

Settling Basins

Gravel Coverage During Construction

4. Filter dam - A 187m sediment filter dam made of woody debris wrapped in a filter blanket was constructed along the entire downstream side of the property.

5. Swales - Treatment swales, which included gravel filter dams, were installed to transport and filter runoff and direct it to the main treatment swale at the rear of the property. The main treatment swale was hydroseeded and lined with jute matting to prevent erosion.

Filter Dam

Swale and Gravel Filter Dam

6. Erosion membrane - Top soil was removed, stockpiled, and sprayed with an organic erosion prevention membrane.

7. Continuous monitoring - Throughout the construction phase, the quality of the runoff water was monitored by the contractor and construction manager. The contractor was on-site during and following heavy rains to ensure all measures were functioning properly.

Spraying Erosion Membrane

Monitoring Samples

Long Term Stormwater Treatment

The project also incorporated many innovative measures that will provide water quality benefits to the environment for the life of the building. Development of buildings and pavement, because they are generally impervious surfaces, increase the rate of runoff. This can be the most harmful aspect of development for a natural stream system. Development also can add pollutants and increase the temperature of runoff. To solve these problems, thoughtful design can mimic nature to create pavement and buildings that absorb water. These measures are designed to:

1. Slow the rate of runoff
2. Allow percolation into the soil
3. Treat the stormwater before it leaves the site
4. Cool the runoff
5. Demonstrate innovative techniques

The measures used included:

1. Eco-roof - The most striking feature of the project is the 1,063 m^2 eco-roof over the administrative offices at the front of the complex. The soils and plants will absorb and filter rainfall, evapotranspirate 10-100% of the precipitation, and cool and retain most of the runoff during hot weather. The eco-roof is made of 12.7cm of soil held in place with baffles. There is a temporary irrigation system that will be removed after plants are established. Native ground cover plants that are drought and sun resistant were used.

Eco-roof

2. Scupper gardens - The runoff from other roofs is directed to "scupper gardens" that absorb the runoff.

3. Green street - The main access road was constructed as a "green street" with gently sloping landscaped swales replacing curbs and gutters. Native plants were used to vegetate the swale.

Scupper Garden

Green Street

4. Porous concrete - The employee parking lot for 60 vehicles was constructed using porous concrete that allows rain to soak into the ground and stay cool. The lot is made of 30.5cm of 1000 psi porous concrete over 46cm of compacted rock. The rock supports the concrete and is an under-drain system because of poor absorption of the clay soils found at the site.

Porous Concrete Parking Lot

Comparison with Rice Krispie Treat

5. Swales instead of pipes - Treatment and conveyance swales vegetated with native plants were used throughout the site instead of catch basins and storm pipes. There is only one catch basin on the entire site, and that was required by the Plumbing Code to drain the rear parking lot to an oil/water separator.

6. Porous techniques - Porous paver blocks and crushed rock were used instead of paving in parking areas and walkways. Paver blocks were placed over 56cm of sand, crushed rock, ballast rock, and a layer of geotextile fabric.

Vegetated Treatment Swale

Paver Block Construction

7. Buffer area - A 15m wide by 183m long water quality facility was constructed along the rear of the property and provides a buffer between the site and the existing nature park to the south. The buffer area was enhanced to create meanders and woody debris was used to create habitat areas.

8. Gravel storage area - The material storage areas were constructed using reinforced gravel which is a plastic cellular confinement system filled with river rock. The product is made by Geo-Web.

Vegetated Buffer Area

Gravel Storage Area Construction

Conclusions

A number of innovative erosion control and stormwater treatment techniques were employed during the construction of new Clean Water Services field operations yard. Additionally, innovative stormwater treatment designs were incorporated into the new facility. Erosion from the site during construction was significantly minimized, meeting an initial goal of the project. Further study of the new stormwater treatment facilities will be required to determine their overall effectiveness; however, early results are promising.

BMP Research Projects
In Southwest Florida

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Four stormwater research projects are currently underway in Lee County (southwest) Florida ongoing in southwest Florida. These projects are being conducted by Johnson Engineering for The Bonita Bay Group (BBG) and the Florida Department of Environmental Protection (FDEP). *The Green Roof Project* at Shadow Wood Preserve focuses on the water quality effects and the potential economic savings of these systems in southwest Florida. The program is assessing critical issues such as plant species selection, temperature and heat flow data, rainfall and flow monitoring and water quality analysis. *The Pavement Evaluation Project* at Shadow Wood Preserve compares the surface water run-off from a typical asphalt parking area with an adjacent but hydrologically isolated porous concrete pavement area. *The Littoral Plantings Project* at Bonita Bay is assessing the ability of littoral plantings in a typical residential/golf course community wet detention lake to absorb copper sulfate. *The Deep Lake Monitoring Program* at The Brooks is a study initiated in early November 2004 to evaluate the characteristics of deep and shallow wet detention lakes in a typical residential/golf course community. Of primary concern is the dissolved oxygen content present in the lake systems and the effects of having the lakes aerated by bubbler devices. Sophisticated sampling, telemetry and data collection system are used at all sites.

Introduction

Due to the considerable lack of water quality data in southwest Florida, particularly in regards to stormwater runoff into retention ponds, research has been directed towards filling these data gaps for this part of the state. These four projects are an example of a joint effort among an Agency (FDEP), a private developer (Bonita Bay Group), and a private consultant (Johnson Engineering) working at its best. An additional project is now underway through a contract with Lee County Natural Resource Management Division, research pollutant removal efficiencies of three local Best Management Practices (BMP).

Study Sites

The Green Roof Project

The Green Roof Project at Shadow Wood Preserve (a residential/golf course community) focuses on the water quality effects and the potential economic savings of these systems in southwest Florida. Very little data currently exists for green roof systems in this area. The program is addressing critical issues such as plant species selection, temperature and heat flow data, as well as rainfall and flow monitoring with rain event wet chemical analysis. Data collection is ongoing as of February 2005.

The Pavement Evaluation Project

The Pavement Evaluation Project at Shadow Wood Preserve compares the surface water run-off from a typical asphalt parking area with an adjacent but hydrologically isolated porous concrete pavement area. Rainfall, surface water flows, and groundwater levels in both systems are monitored continuously and are recorded on site. The last samples have been collected and data is currently being evaluated and summarized as of February ‘05.

The Littoral Plantings Project

The Littoral Plantings Project at Bonita Bay Lake 62 is to assess the ability of littoral plantings in a typical residential/golf course community wet detention lake to absorb copper sulfate. Rainfall, surface water flows and lake levels are continuously recorded on site, and rain event wet chemical sampling and weekly composite sampling has been occurring for the past year. Also in place in Lake 62 are two continuously recording YSI 6600EDS data sondes, collecting field data at 15 minute intervals to supplement the laboratory analysis being done. One significant finding to date is that the lake system actually discharges to the receiving body only during a very small portion of time on an annual basis.
2

The Deep Lake Monitoring Program – Dissolved Oxygen Study

The Deep Lake Monitoring Program at The Brooks is a study initiated in early November 2004 to research the characteristics of deep and shallow wet detention lakes in a typical residential/golf course community. Of primary concern is the dissolved oxygen content present in the lake systems and the effects of having the lakes aerated by bubbler devices. Data is being collected for four different lakes by utilizing continuously recording YSI 6600EDS data sondes in each lake, anchored at 1 foot off the lake bottom. This data is supplemented by laboratory analysis at the study start and stop points. Two lakes have aerators on and two have aerators off. After a two week data collection period, the aerators will be switched and the study repeated.

Note: Except for the deep lake monitoring program, all sampling is accomplished by using sophisticated automatic programmable refrigerated sampling units made by ISCO which are tied to flow measuring equipment making flow compositing practical. These units have remote communication ability and store rainfall, flow and sampling program data onsite which is then uploaded via telephone communications or to onsite Panasonic TuffBook laptops.

Materials and Methods

The Green Roof Project

The Green Roof Project at the Shadow Wood Preserve project included three test plots, each approximately 800 square feet in area. The green roof system was designed by Charlie Green of Roofscapes, Inc. All of the test plots emphasized good drainage, since the threat posed to the plants by the hot humid summer conditions seemed greater than the winter dry season. The growth media used in each case was designed with a volumetric maximum moisture content of 35%. The grain-size distribution was skewed toward fine sand and course silt size particles in order to increase surface area and moisture-holding properties. The mixture included expanded shale, fine vermiculite and compost. The three plots have varying media mix specifications as described in a July 25, 2003 Memorandum from Charlie Miller to Kevin McKyton of Bonita Bay Group. (2)

The most difficult challenge was to develop a plant list. This effort was headed up primarily by Kim Fikoski, Senior Biologist at Bonita Bay Group. In most previous green roof lists, *Sedum*, a versatile succulent perennial has predominated. These flowering plants spread rapidly and create a uniform ground cover that is self-healing. *Sedum* varieties exist that are adapted to cool temperate climates (with frost tolerance to Zone 3), as well as *Sedums* that are native to the deserts of Mexico.

Plant species	Number of plants	Plant species	Number of plants
Seum oaxacuanum	36	Spartina spartini	100
Sedum album murale	86	Portulaca spp.	150
Sedum sexangulare	82	Portulaca pilosa	200
Sedum rubrotintum Dwarf	67	Tradescantia Wandering Jew	75
Sedum lineare variegatum	4	TYPE III ZONE ONLY	
Maleophora Tequila Sunrise	96	Sedum tetractinum	12
Delosperma cooperii	96	Sedum bohmeri (Orostachys)	12
Delosperma herbeau	4	Sedum pachyphyllum	12
Delosperma nubigenum	91	Sedum muscoideum	1
Euphorbia millii Rosey	100	Sedum album micranthum	12
E. millii Short & Sweet	100	Delosperma floribundem	12
Zephyranthes Rain Lily	50	TYPE II ZONE ONLY	
Aloe vera	33	Agapanthus spp.	30

Table 1. List of Plants for the Shadow Wood Preserve Green Roof Initial Installation

The three roof plots were planted during the week of July 12, 2003. A total of 1433 plants were installed, generally with plants of each variety distributed in groups of 1- to 15 across each of the three roof areas. It quickly became apparent that the *Sedum* varieties could not withstand the hot humid summer conditions. The other plants in the trial, however, thrived. *Portulaca*, in particular seemed to thrive and expanded its presence by seeding itself across the entire roof. The winter and spring dry season, now just ended, presented new challenges to the plants. All of the plants showed signs of drought stress in mid-winter and into spring. In addition, a number of the remaining trial species perished. However, species in the genus of *Delosperma*, *Portulaca*, *Euphorbia* and *Aloe* appear to be good choices. Of these, only *Delosperma* and *Portulaca* are able to create a dense low ground cover which is essential to for green roof design. Once the wet season returns (June), we intend to introduce new species into the trial, in an effort to broaden the list of suitable plants.

One important lesson of this trial has been that the type of media used and the green roof profile structure are secondary to the correct choice of plants. Little difference has been observed between the three test plots, despite their varying moisture management approaches. However, based on our observations we would not recommend un-irrigated green roof with media depths less than six inches. (3)

The other aspect covered by the project is data collection of temperature and heat flow measurements in the green roof system to enable evaluation of the potential economic savings in green roof systems for South Florida installations. To accomplish this, a variety of thermal sensors both on surface black and white bodies, and on the bottom of the vegetation surface are reported to a standard datalogger. The combination of these values represent the thermal gradient through the roof system. This complex thermal process has not been well studied, and this effort attempts to segregate the effects of insulation and heat absorption. (1)

The Pavement Evaluation Project

This project attempts to characterize the potential benefits of parking lots constructed of porous concrete pavement as opposed to typical asphalt based systems. A properly designed and engineered porous system is critical to a successful project as well as an experienced porous concrete contractor to do the installation. When properly done, these systems can be a viable alternate to standard asphalt installations when the volume reduction in stormwater runoff is considered along with the complementary reduction in pollutant loadings entering stormwater systems. Appearance and durability can be a concern and further efforts by the porous pavement industry will improve these factors.

The basins selected for study were in areas that were hydrologically isolated, each having its own outfall catch basin. These catch basins were outfitted with a fiberglass insert box with a specially designed V-notch to facilitate flow measurements. The insert box also held the tube inlet for sample collection. Rainfall would drop onto the pavement, flow to the inlet grate, drop into the fiberglass box, overflow the V-notch, and finally drop into the catch basin bottom and out the concrete stormwater discharge pipe to the wet detention lake system.

Two automated, refrigerated ISCO sampler units were tied to a nearby rain bucket to facilitate storm event detection, and to put the ISCO units into sample mode. Continuous flow measurements were made utilizing bubbler tube technology, allowing flow composited samples to be collected. Events were collected for both wet season and dry season storms, with a 1-inch event being the target sample event.

Data collection for this project is now complete, and lab results, volumes and water levels are being evaluated. Of particular note is the substantial reduction in discharge volumes from the porous system compared to the standard asphalt system. During the initial onset of a storm event, essentially all of the runoff generated in the porous area, percolates through the porous concrete in to the subsurface system and enters the groundwater table. Obviously, this can be overridden by very high rainfall intensities that exceed the infiltration rate of the porous pavement, but under normal to low intensities, it was not unusual to see a 30 minute delay before any water entered the catch basin in the porous area, compared to the standard asphalt area. This can be a significant benefit on its own, for the receiving water body, in addition to any pollutant reduction accomplished by the filtering action of the porous concrete itself.

The Littoral Plantings Project

This project was designed to assess the effects of a littoral planting area on the copper sulfate uptake rate in a typical wet detention system in southwest Florida. The target wet detention lakes are in a typical residential/golf course community and have inflows from rain falling on residential lawns, asphalt roadways, and golf course areas. The primary target lake is set up with 3 sample stations: one at the lake inflow pipe, one at a temporary V-notch controlled outflow, and a third at the final outfall control structure. Outflow from the lake passes over a naturally vegetated area before overflowing the final control structure.

Automated, refrigerated, ISCO samplers are used at all three sample stations to collect both flow composited samples and weekly composite samples taken during periods of limited rain events. These samplers are sophisticated programmable units capable of sampling a variety of protocols, and have telemetry equipment to allow remote observation of the sample units status, flows, rainfall, and other on site data. At event triggers, the telemetry units notify field staff of an ongoing event using standard telephone line communications and beeper technology.

The primary target lake also contains two YSI 6600 EDS Data Sondes, deployed at mid depth to provide ongoing data collection at 15 minute intervals for dissolved oxygen, specific conductance, pH, temperature, oxidation/reduction potential, depth, and turbidity. This high resolution data can be compared with lake water levels collected with Infinities USA pressure transducer dataloggers, as well as with laboratory analysis from the ISCO sample units.

Data collected to date represents the background data collected prior to construction of the littoral shelf. It is anticipated that the littoral plantings will be done during this dry season and monitoring will begin again at the next wet season. Once substantial data has been collected in the post planting phase, datasets can then be compared and copper sulfate uptake rates compared between pre planting and post planting conditions.

The Deep Lake Monitoring Project

This project was designed to obtain field collected data cross checked by wet chemical analysis, to be used in evaluating a group of water quality parameters in typical southwest Florida wet detention lake system. The program assessed these parameters in lake systems of varying depths that were both aerated and non-aerated.

The data collection process will be controlled by establishing a set of target lakes that have similar characteristics of surrounding land use and by selecting monitor location points in each lake that represent typical conditions and not subject to unusual configurations that would restrict the natural movement of water.

Field data was collected by using YSI 6600 EDS Multiparameter Data Sondes. One YSI unit was deployed in each of the 4 target lakes, and anchored 1 foot above the lake bottom. These units were programmed to collect sensor readings at 15 minute intervals, giving a very high resolution dataset. The data sondes collected readings for the following parameters: dissolved oxygen, specific conductance, pH, temperature, oxidation/reduction potential, depth, and turbidity.

Calibration of the YSI 6600 EDS units were performed a total of five times throughout the two phase monitoring program at the program start, completion and in between study phases.

> Sensor calibrations were performed using calibration standards provided by the equipment manufacturer. In addition to this field calibration, five water quality samples were collected (one sample from each of the four monitored lakes) and sent to an approved laboratory for analysis.

Phase 1 Monitoring – 14 days	Phase 2 Monitoring – 14 days
Lake 41 – Aerator Off	Lake 41 – Aerator On
Lake 17 – Aerator On	Lake 17 – Aerator Off
Lake 19A – Aerator Off	Lake 19A – Aerator On
Lake 20 – Aerator Off (Control)	Lake 20 – Aerator Off (Control)

These lakes are all normally aerated during non monitored periods using bubbler type aerators. Trends that develop in the deeper lakes in Phase 1 should be reproducible in Phase 2 after switching the aerator controls. Onsite lake maintenance operation personnel indicated that these bubbler systems were a positive benefit to maintenance operations and general lake aesthetics.

At the conclusion of the program, all aerators designed to be in operation on the deep lakes were switched back on according to local regulatory requirements.

Results

The Green Roof Project
A total of 8 rain events were sampled between 02/05/04 and 09/28/04. The storm events had rainfall totals ranging from 0.8" to 1.98".

The stormwater quality results from Event #1 are shown in Table 2 while the roof temperatures are shown in Figure 1.

Analyte	Result	Units
Cadmium	BDL	mg/l
Chromium	BDL	mg/l
Copper	0.012	mg/l
Zinc	0.015	mg/l
Dissolved Copper	BDL	mg/l
Ammonia as N	BDL	mg/l
NO2+NO3 as N	0.31	mg/l
Orthophosphate as P	0.29	mg/l
Total Phosphorus as P	0.41	mg/l
Total Kjeldahl Nitrogen as N	1.2	mg/l
Nitrogen, Total as N	1.51	mg/l
Total Suspended Solids	1.6I	mg/l

Table 2 Stormwater Quality Data from Rain Event 1

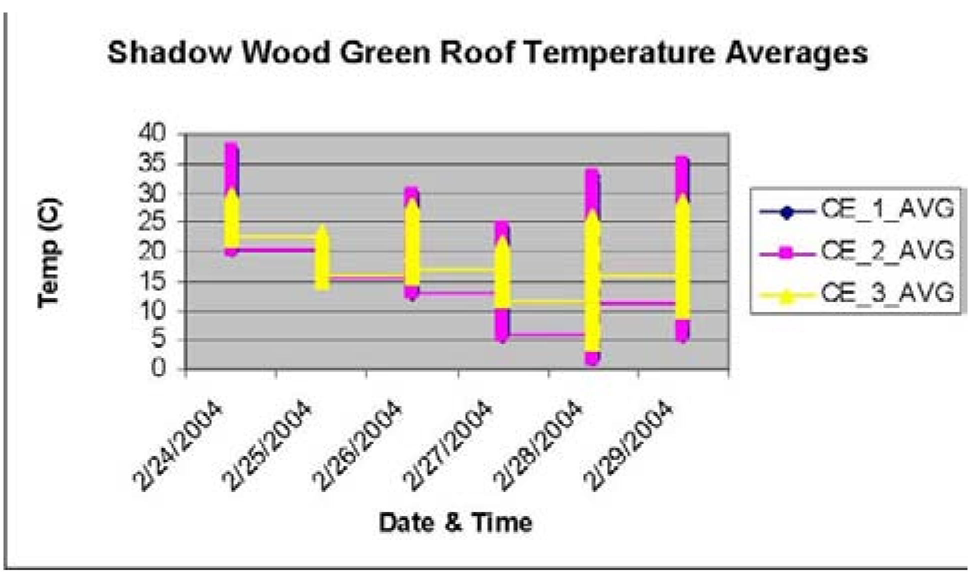

Figure 1 - Green Roof Temperatures

The Pavement Evaluation Project

Ten total storm events have been sampled and analyzed for the Shadow Wood Preserve Project. A typical analyte summary for event 1 is shown in Figure 1.

Analyte	Porous Result	Asphalt Result	Units
Cadmium	BDL	BDL	mg/l
Chromium	BDL	BDL	mg/l
Copper	BDL	BDL	mg/l
Zinc	0.025	BDL	mg/l
Dissolved Copper	BDL	BDL	mg/l
Ammonia as N	BDL	0.023	mg/l
NO2+NO3 as N	BDL	0.17	mg/l
Orthophosphate as P	0.069	0.017	mg/l
Total Phosphorus as P	0.18	0.013	mg/l
Total Kjeldahl Nitrogen as N	0.48	0.45	mg/l
Nitrogen, Total as N	BDL	0.62	mg/l
Total Suspended Solids	23	BDL	mg/l

Figure 1 – Pavement Evaluation Results Event #1

The Littoral Plantings Project

Twenty three water quality sample sets have been collected and laboratory analyzed, representing both wet season and dry season rain events as well as weekly composites taken during dry season periods of limited storm event activity. The sample sets were collected between 02/25/04 and 11/18/04, and represent storm event rainfall totals from 0.28" to 1.88". Typical data is represented in Figure 1 below for Event #1.

Analyte	Result	Result units
Cadmium	BDL	mg/l
Chromium	0.097	mg/l
Copper	BDL	mg/l
Zinc	BDL	mg/l
Dissolved Copper	0.081	mg/l
Ammonia as N	BDL	mg/l
NO2+NO3 as N	BDL	mg/l
Orthophosphate as P	BDL	mg/l
Total Phosphorus as P	0.1	mg/l
Sulfate	56	mg/l
Total Kjeldahl Nitrogen as N	1.4	mg/l
Nitrogen, Total as N	1.4	mg/l
Total Suspended Solids	2.6	mg/l

Figure 1 – Littoral Plantings Event #1

The Deep Lake Monitoring Program

Laboratory analysis has been completed for three samples occurring on 11/01/04, 11/15/04 and on 12/06/04. Each sample set contains the same group of water quality parameters for each of the four study lakes, for a total of 12 total datasets. This is in addition to the data produced by the YSI field units with in place sensors.

Figure 1 below shows the results of the wet chemical analysis for Lake 17 on 11/01/04. Figure 2 shows the corresponding dataset from the in place YSI data sonde closest to the time of samples collected for wet chemical analysis.

Analyte	Result	Units
PH	7.70 Q	UNITS
TURBIDITY	4.9	NTU
AMMONIA NITROGEN	0.032	MG/L
TOTAL KJELDAHL NITROGEN	1.17	MG/L
TOTAL NITROGEN	1.19	MG/L
NITRATE+NITRITE	0.020	MG/L
ORTHO PHOSPHORUS	0.081	MG/L
TOTAL PHOSPHORUS	0.160	MG/L
CHLOROPHYLL A, CORRECTED	21.7	MG/M3
SPECIFIC CONDUCTANCE	650	UMHOS/CM

Figure 1 – Lake 17 on 11/01/04 - Wet Chemical Analysis

DateTime	Time	Temp	SpCond	DO	pH	ORP	Depth	Turbidity
11/1/2004	10:01	26.28	0.65	7.08	7.74	269	3.849	1.2
11/1/2004	10:16	26.29	0.651	7.16	7.74	269	3.85	1.2
11/1/2004	10:31	26.28	0.65	7.04	7.73	270	3.851	1.2
11/1/2004	10:46	26.28	0.648	7.05	7.73	269	3.852	1.3
11/1/2004	11:01	26.29	0.651	7.08	7.73	269	3.851	1.2
11/1/2004	11:16	26.28	0.651	6.87	7.71	269	3.852	1.4
11/1/2004	11:31	26.28	0.651	6.77	7.7	269	3.851	1.4
11/1/2004	11:46	26.29	0.652	6.88	7.72	269	3.849	1.2
11/1/2004	12:01	26.29	0.651	6.97	7.72	269	3.848	1.2
11/1/2004	12:16	26.29	0.651	6.65	7.7	268	3.846	1.4
11/1/2004	12:31	26.29	0.652	6.51	7.68	266	3.845	1.8
11/1/2004	12:46	26.29	0.652	6.53	7.69	266	3.843	1.7

Figure 2 – Lake 17 YSI Typical Data Sonde Results

Preliminary Conclusions and Discussion

The Green Roof Project – Further analysis of the data collected to date is required to assess the energy savings potential of the roof system. Replanting the roof sections with a different mix of plant species as well as irrigation of one or more roof sections is currently being considered.

The Pavement Evaluation Project - Further analysis of the data collected to date is required to evaluate the potential pollutant loading reduction benefit of the porous pavement system. What is obvious at this point is the marked decline in net runoff volume in the porous system compared to the standard asphalt system. Further improvements construction and maintenance techniques will enhance the desirability of these systems as an alternate best management practice technique.

The Littoral Plantings Project – This project is partially complete and data collection will continue after installation of the littoral plantings. This particular lake system does not discharge regularly to an outfall, although it is designed to do so. Discharge only occurs after prolonged periods of steady rainfall followed by a large storm event. During the majority of time throughout the year, the runoff entering this system leaves the lake either by evapo-transpiration or percolating into the groundwater table aquifer. It is not well documented how typical this discharge behavior is in southwest Florida, but it is believed by Johnson Engineering staff to be not uncommon.

Flow measurements into the lake are being accomplished by the use of a Sontek flow meter utilizing the Doppler effect principle, mounted inside a 30” round concrete pipe. This has proved problematic and has made flow compositing difficult. The problems encountered are very low velocities (<0.5 fps), signal format and integration into the ISCO controller module, calibration and signal noise. Efforts are ongoing with the equipment manufacturer to improve this methodology.

The Deep Lake Monitoring Program – The data collected by the study in the form of the in place YSI data sondes is extremely resolute, occurring every 15 minutes. This should be studied further in conjunction with the laboratory analysis of the duplicate samples to evaluate the effects of the aerator bubblers on the lake systems. The current study was completed in dry season conditions. Water levels were below control elevation and the only flows in the system were due to groundwater table aquifer flows. This study is proposed to be performed during the rainy season when flow conditions exist.

Acknowledgements

This study could not have been accomplished without the growing research partnership between the Bonita Bay Group, the Florida Department of Environmental Protection, Lee County Community Development staff, Roofscapes Inc., and Johnson Engineering staff. Johnson Engineering would like to thank the Bonita Bay Group for their assistance and cooperation in the site selection of the Brooks Development and allowing water quality monitoring of shallow and deep wet detention systems.

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A Stormwater Retrofit Incorporating a CDS Unit

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Abstract

A major storm sewer outfall was retrofitted with a Continuous Deflective Separation (CDS) unit and a linear marsh to help treat stormwater discharged from an urban drainage basin in Tampa, Florida. The CDS unit was very effective for removing the gross solids including litter, trash, debris, leaves and sediments larger than 64 microns. These pollutants have not usually been evaluated in stormwater studies that use automated water quality sampler. Most of the gross solids were deposited between February and August 2003. During the first year of this ongoing study, about 300,000 cubic meter of flow passed through the system including both storm and base flow. The CDS unit removed 11.69 m^3 (413 ft^3) of gross solids from the flow stream including toxic levels of Polycyclic Aromatic Hydrocarbons (PAHs).

Introduction

This retrofit project was designed to reduce the amount of pollution discharged to the Hillsborough River and ultimately Tampa Bay by installing a Continuous Deflective Separation (CDS) unit and a constructed linear marsh at a major urban storm sewer outfall. The CDS technology is designed to remove large sized particles such as litter, leaves, twigs, sand and paving residue from storm runoff. The Broadway Outfall retrofit project consists of two phases. Construction of the retrofit (phase I) was completed in November 2001; and the evaluation effort (phase II), was initiated in November 2002. This report presents some of the results from the first year of data collection for phase II.

The monitoring project was designed to measure: 1) how much and what kind of gross solids (>64 microns) are collected by the CDS unit, 2) the concentration of constituents in the flow stream for the suspended and dissolved particles, 3) the accumulation of pollutants in the sediments, 4) the characterization of the macroinvertebrates in the sediments, and 5) the hydrology of the system including storm flow, base flow and rainfall. Space constraints limit the scope of this paper to some summary hydrology and the results of the gross solid data. A complete report is available upon request.

Site Description

The drainage basin that discharges to the Hillsborough River through the Broadway Outfall storm sewer is approximately 53.58 hectare (132.4 acres) in size and includes a 12.3 hectare (30.6-acre) high intensity strip commercial district immediately upstream. The remainder of the watershed includes multi-family and residential land uses as well as a golf course and major urban thoroughfares (Figure 1). As part of the Broadway Outfall Stormwater Retrofit Project a Model PSW100_60 (0.906 cms (32 cfs) capacity) CDS unit was installed in series with an excavated sediment sump followed by a shallow linear marsh system, extending approximately 500 feet downstream from the unit.

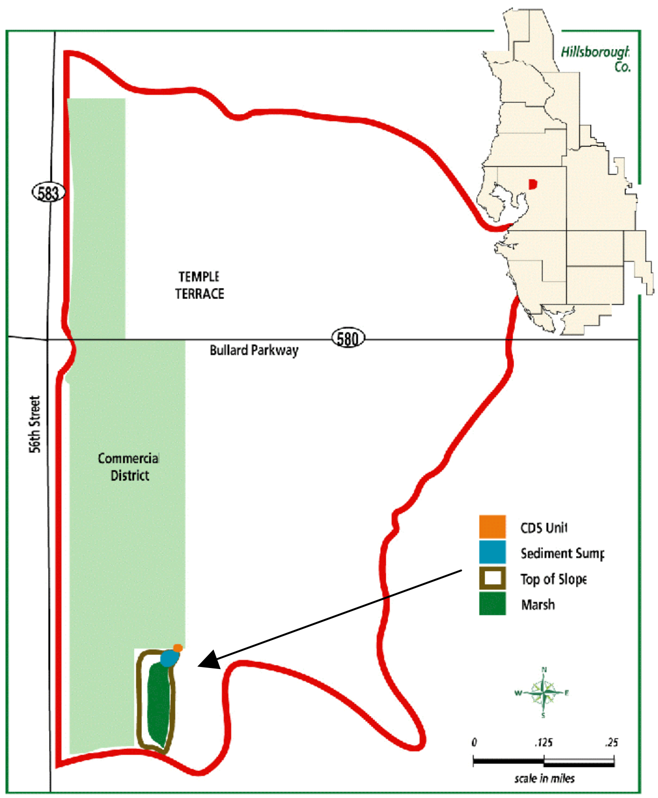

Figure 1. Site plan showing an outline of the drainage basin, the strip commercial area and some of the details of the retrofit with the CDS unit and the constructed marsh at the lower left.

A CDS unit is an underground stormwater treatment method used to capture gross pollutants in urban areas by intercepting storm runoff in the conveyance pipe system. The mechanism by which the unit separates and retains gross solids is by deflecting the inflow and associated pollutants away from the main flow stream into a pollutant separation and containment chamber. A vertical section view (Figure 2) shows the dimensions of the CDS unit.

The chamber is cleaned out with a vactor truck and the gross solids are sent to a landfill or disposed of in some other appropriate manner. Gross solids have not usually been measured in storm water studies since they are not included in the water collected using automated water quality samplers. These samplers generally exclude solid material including trash, litter, debris, leaves and sediments larger than 64 microns. Yet these pollutants degrade aquatic habitat, cause visual blight, smother productive sediments, leach harmful pollutants and can cause unpleasant odors.

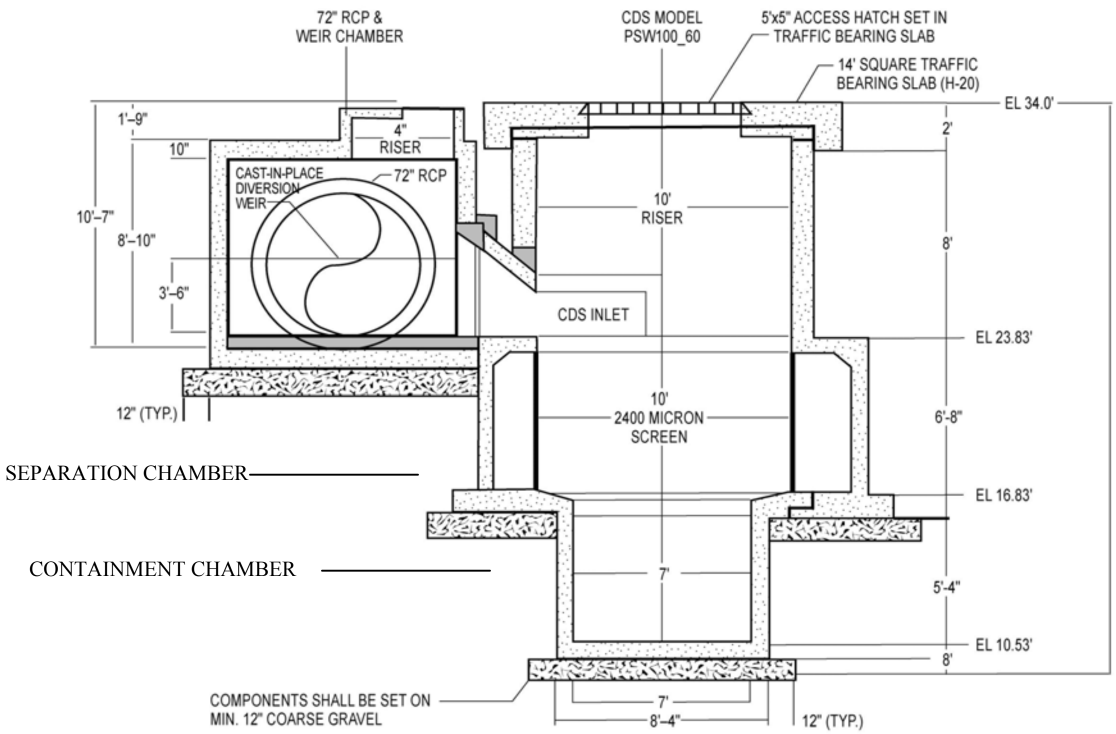

Figure 2. A vertical section view of the Continuous Deflective Separation (CDS) unit shows the dimensions.

Method

Hydrology measurements were calculated using velocity meters, water level sensors, tipping bucket rain gauges and appropriate weirs and formulas. This information was stored in data loggers until retrieved and downloaded into spreadsheets to be processed into tables and figures. Bypass flow over a diversion weir was also measured.

Water quality. Flow weighted samples were taken to measure water quality for both storm flow and base flow using automated samplers. Measurements were made in front of and after the CDS unit and at the outflow of the marsh.

Gross solids were analyzed each time the unit was vacuumed out. The material in the unit, excluding the litter, is measured monthly and cleaned out when the material is about

5 to 6 feet deep. The floatable litter is skimmed off the top each month and dried in meshed bags to be combined with the litter extracted from the mass at the time of cleanout when all the litter is sorted and weighed by category. The rest of the mass is analyzed using methods developed for soil samples. At a minimum a representative sample including different depths and different locations within the unit is collected and sent to a laboratory to be analyzed. For one sampling event, the pollutants were analyzed by particle size and each particle size was evaluated separately. Gross solids are defined in this study as particles larger than 75 micron, which are usually not collected using automatic sampling equipment.

Calculations The ability of the CDS unit to reduce (increase) pollutants was calculated using both the flow weighted water quality samples and the material collected in the CDS unit. Bulk density of the material collected by the CDS was used to convert the material to the same mass units as the water quality loads measured in storm and base flow. Efficiency was calculated for a six-month time period using the following formula:

$$\text{Efficiency } (\%) = ((\text{load in}) - (\text{load out}) / \text{load in}) * 100$$

Results and Discussion

During the first year of the study the CDS unit collected almost all of its gross solids during a six-month period in the spring and summer (Figure 2). The efficiency data in this report represent this six-month period, which extended from February 1st to July 14th, 2003. During this time period, 18 storm samples and 26 base flow samples were analyzed for water quality. This included 57 percent of all the storm event rainfall. Many of the smaller events that were not sampled individually were included in the base flow samples. All of the flow was measured and included in the calculations for mass loading. The CDS unit collected 3.5 cubic meters (336 cubic feet) of material and most of the collected mass was leaves (55 to 75%). Summary data are presented in Table 1.

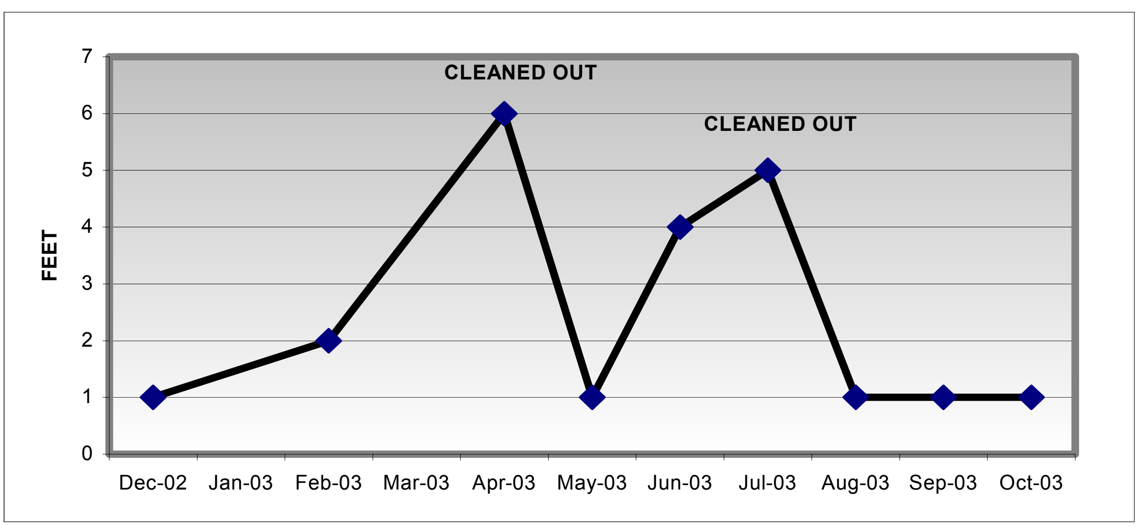

Figure 2. Depth of material measured in the CDS unit shows the two cleanouts.

Table 1. Summary data for water quality and gross solid concentrations and loads for period between Feb.1 and June 14, 2003.1414.

SAMPLE TYPE	FLOW AMOUNT	TOTAL NITROGEN		TOTAL PHOSPHORUS		RECOVERABLE COPPER		RECOVERABLE ZINC		TOTAL* SOLIDS		ORTHO PHOSPHORUS		AMMONIA		NITRATE+ NITRITE		TOTAL KJELDAHL-N	
		BEFORE CDS	AFTER CDS	BEFORE CDS	AFTER CDS	BEFORE CDS	AFTER CDS	BEFORE CDS	AFTER CDS	BEFORE CDS	AFTER CDS	BEFORE CDS	AFTER CDS	BEFORE CDS	AFTER CDS	BEFORE CDS	AFTER CDS	BEFORE CDS	AFTER CDS
WATER QUALITY		(mg/l)	(mg/l)	(mg/l)	(mg/l)	(ug/l)	(ug/l)	(ug/l)	(ug/l)	(mg/l)	(mg/l)	(mg/l)	(mg/l)	(mg/l)	(mg/l)	(mg/l)	(mg/l)	(mg/l)	(mg/l)
STORM SAMPLES																			
# Samples		18	18	18	18	18	18	18	18	17	17	18	18	18	18	18	18	18	18
Mean		1.141	1.217	0.104	0.108	9.5	11.8	51.8	58.7	14.2	14.4	0.027	0.023	0.057	0.036	0.440	0.373	0.701	0.843
Median		1.150	1.115	0.092	0.092	9.4	9.8	46.8	49.2	11.7	12.8	0.024	0.019	0.018	0.018	0.441	0.378	0.631	0.730
St. Dev.		0.291	0.351	0.048	0.049	3.3	5.3	23.7	34.7	11.7	10.0	0.020	0.016	0.071	0.047	0.228	0.186	0.271	0.370
Max		1.740	1.900	0.235	0.205	18.6	25.5	112.0	147.0	50.1	49.4	0.081	0.063	0.205	0.160	1.010	0.819	1.198	1.519
Min		0.591	0.656	0.039	0.046	3.9	3.9	10.4	11.6	1.6	3.3	0.005	0.005	0.006	0.006	0.074	0.065	0.180	0.311
C.V.		0.25	0.29	0.46	0.45	0.3	0.4	0.5	0.6	0.8	0.7	0.74	0.72	1.26	1.29	0.52	0.50	0.39	0.44
BASE FLOW SAMPLES																			
# Samples		26	26	26	26	26	26	26	25	26	25	26	26	26	26	26	26	26	26
Mean		1.488	1.376	0.069	0.074	11.7	11.5	23.1	25.9	5.8	6.8	0.025	0.018	0.022	0.039	0.915	0.705	0.573	0.671
Median		1.440	1.305	0.051	0.065	9.0	8.5	20.4	18.5	3.6	4.4	0.019	0.016	0.016	0.024	1.030	0.699	0.454	0.624
St. Dev.		0.318	0.280	0.044	0.040	13.8	11.6	12.6	19.5	5.4	5.6	0.021	0.012	0.019	0.045	0.383	0.310	0.290	0.287
Max		2.250	2.050	0.169	0.185	74.0	61.2	44.9	67.6	21.5	23.8	0.091	0.046	0.074	0.197	1.940	1.320	1.132	1.443
Min		0.760	0.800	0.016	0.018	1.5	3.2	5.9	5.6	0.6	1.1	0.005	0.005	0.006	0.006	0.269	0.177	0.250	0.282
C.V.		0.21	0.20	0.64	0.54	1.18	1.01	0.54	0.75	0.93	0.82	0.84	0.68	0.89	1.16	0.42	0.44	0.51	0.43
MASS LOADING																			
FLOW***	(cu ft)	(kg)	(kg)	(kg)	(kg)	(grams)	(grams)	(grams)	(grams)	(kg)	(kg)	(kg)	(kg)	(kg)	(kg)	(kg)	(kg)	(kg)	(kg)
MEANS																			
STORM	101,794	116.2	123.8	10.6	11.0	965.1	1198.6	5276.9	5973.6	1443.6	1461.3	2.7	2.3	5.8	3.7	44.8	38.0	71.4	85.9
BYPASS**	46,876	53.5	53.5	4.9	4.9	444.4	444.4	2430.0	2430.0	664.8	664.8	1.3	1.3	2.7	2.7	20.6	20.6	32.9	32.9
BASE	37,351	55.6	51.4	2.6	2.8	436.6	429.7	861.6	968.1	218.2	253.4	0.9	0.7	0.8	1.4	34.2	26.3	21.4	25.0
MEDIANS																			
STORM	101,794	117.1	113.5	9.4	9.4	959.4	1000.6	4764.0	5003.2	1191.0	1303.0	2.4	1.9	4.6	2.6	23.2	19.0	64.2	74.3
BYPASS	46,876	53.9	53.9	4.3	4.3	441.8	441.8	2193.8	2193.8	548.5	548.5	1.1	1.1	0.8	0.8	20.6	20.6	29.6	29.6
BASE	37,351	53.8	48.7	1.9	2.4	334.5	317.3	762.0	691.0	133.3	162.8	0.7	0.6	0.6	0.9	38.5	26.1	16.9	23.3
CAPTURED IN CDS UNIT																			
CONCENTRATIONS				(mg/kg)		(u g/kg)		(u g/kg)		(cu.meter)								(mg/kg)	
		na		770		33		134		9.5		na		na		na		3,977	
LOADS				(kg)		(grams)		(grams)		(kg)								(kg)	
		na		3.96		139.5		688.5		5138.0		na		na		na		20.4	

- Total solids are total suspended solids for water quality and total solids for CDS material.

Pollution Removal Efficiency - One of the purposes of the study was to calculate load reductions for comparison to other storm water studies and that would also be appropriate for determining Total Maximum Daily Loads (TMDL).

Including the sump material in the calculations substantially improves the efficiency of the CDS unit to remove pollutants and also shows that the CDS unit is quite good at reducing some pollutants, but not for removing others. The percent efficiency calculated for loads both with and without the sump material included is shown in Table 2.

Table 2. Load efficiencies including loads for water quality samples only and loads that also include the amount retained in the CDS unit. Negative percentages indicate higher loads were discharged exiting from the CDS unit than entered.

Constituent	Water Quality Only		Includes CDS Loads	
	Sample Means	Sample Medians	Sample Means	Sample Medians
Total Solid *	-2%	-8%	68%	71%
Total Kjeldahl Nitrogen	-15%	-15%	2%	3%
Total Phosphorus	-4%	-3%	15%	18%
Recoverable Copper	-12%	-1%	-4%	6%
Recoverable Zinc	-9%	-2%	-1%	6%
Total Nitrogen	-2%	4%	na	na
Ammonia **	15%	-6%	na	na
Nitrate + Nitrite	15%	20%	na	na
Ortho Phosphorus	13%	13%	na	na

* Water Quality samples include only the suspended solids loads and the CDS loads are total loads.
** The median calculations for ammonia were skewed by six rain events where ammonia was measured below the laboratory limit of detection. When these events are eliminated the efficiency is +29 percent
na = not analyzed in the material collected by the CDS unit.

Total Solids – The unit is quite effective at removing the larger solid material (>75 microns) found in bed loads such as street dirt, leaves and other large size particles. Removal efficiencies for total solids were between 68 and 71 percent. This is not surprising since the units were designed to capture this type of material. If the 32 percent of flow that bypassed the CDS unit is considered (by assuming that the bypassed flow contained the sample proportion of material as was collected by the CDS unit), then the percent reduction is between 56 and 58 percent. The increase in suspended loads in the water column indicates that particles, in particular, fragile material such as leaves, are being broken down into smaller particles as they move around in the CDS unit.

Total Phosphorus – The efficiency measured for total phosphorus (15 to 18%) probably reflects the fact that phosphorus easily attaches to soil particles and organic material and it is expected it would be reduced by attaching to solids. Ortho phosphorus the inorganic portion of total phosphorus is reduced by 13 percent in the water samples indicating it is being transformed to organic phosphorus or attaching to particles.

Total Kjeldahl Nitrogen (TKN) – When the sump material is included in the calculations, TKN removal improves from an increase of 15 percent to a small, but actual removal (2 to 3%). Some of this can be explained by nitrogen transformations. TKN is the combination of organic nitrogen and ammonia. Much of the ammonia and nitrate is converted to organic nitrogen and these soluble nutrients are reduced in the CDS unit as shown for the means of the water quality samples (Table 1). Also, the anaerobic conditions in the CDS unit favor denitrification. The negative efficiency for water quality samples for ammonia was caused by six rain events where concentrations for both the inflow and outflow were below the laboratory detection limit. When these values were removed the ammonia traveling through the CDS was reduced by 29 percent instead of the 6 percent increase calculated for Table 2.

Recoverable Metals – Copper, lead and zinc were measured at low concentrations at the site and probably exhibited no reduction by the CDS unit. Besides the low concentrations, the low pollution removal can possibly be explained by the tendency of metals to attach to the smaller sized particles, where were not collected by the CDS unit. Particle size analysis revealed only a low percentage of particles that measured less than 75 microns were retained by the unit. Although organic material has been found to be an effective sink for metals and over 50 percent of the material collected by the CDS unit was tree leaves, the zinc, lead and copper concentrations retained were quite low and average values were below levels considered toxic to sensitive organisms. Lead was not evaluated for this paper because most concentrations were below the laboratory detection limit.

Captured Pollutant Concentrations – The CDS unit was quite effective at removing two types of pollution that are not usually measured for storm water BMP monitoring studies – PAHs and trash. Since PAHs are rarely measured in water quality sample, these pollutants could not be analyzed for removal efficiency using the method described above. But, they are undoubtedly reduced and the amount retained by the CDS unit was quantified. Toxic levels of PAHs were measured in the material in the CDS unit as well as in soils where the unit discharges into the pond.

Polycyclic Aromatic Hydrocarbons (PAHs) – The total PAH values in the material collected by the CDS unit (150,966 ug/kg) exceeded the level considered toxic (44,792 ug/kg), especially during the April cleanout when concentrations were almost five times higher than the level where effects to wildlife occur. PAHs have been identified as a serious problem in Hillsborough Bay and this data helps explain why elevated concentrations are deposited there.

Metals in the material collected by the CDS unit usually fell in the range between the possibly toxic level and the probably toxic level, during the April cleanout, but fall below the possibly toxic range when analyzed on a yearly cycle.

Sediment Samples - The effect of polluted sediments from gross solid deposition has not always been emphasized in stormwater studies that make their interpretation and pollution reduction on water quality data for individual storm events. But sediments can accumulate pollutants through mechanisms of direct deposition of solids, or through various process where soluble pollutants precipitate/sorb and contaminate the sediments. Scouring of storm conveyance systems and ultimately streams and rivers takes a long time and is difficult to relate to specific storm events. These polluted sediments probably have a greater toxic effect on the biota than the dissolved toxicants in the water column. Many studies have shown the severe detrimental effects of urban runoff on receiving water organisms (Pitt 1995). Other studies have documented that even tolerant species are eliminated when toxic levels of metals and PAHs are measured in the sediments (Rushton et al. 2004).

Sediment samples were taken along the flow path in the pond before the CDS unit was installed (May 2, 200), after construction (August 2002) and two years later (August 2004) (Figure 3).

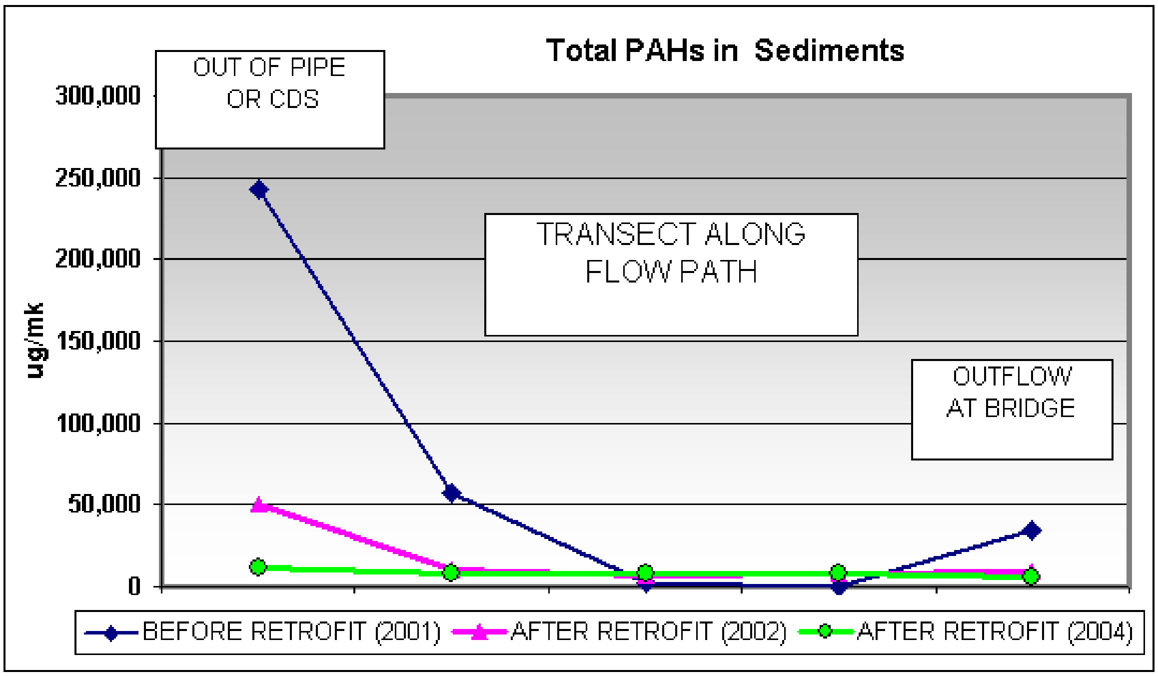

Figure 3. Concentrations of PAHs measured along the flow path in the sediments of the ditch before the retrofit and in the pond after construction of the CDS unit. The X-axis represents distances downstream of the CDS unit. Total distance 152.4 meters (500 feet).

The PAHs were measured at much higher levels in the soils in 2001 before the retrofit. Concentrations were also much higher near the storm sewer where it flowed into the open water ditch. Concentrations tapered off to non-detectible concentrations as water flowed down the ditch until it reached the bridge, where additional storm runoff entered the flow stream. Concentrations in the sediments are much reduced since the installation of the CDS unit. This can be attributed to the CDS unit or to the clean soils uncovered during construction of the pond. The reduction in concentrations at the bridge is a result of that runoff being rerouted beyond the pond after construction.

Litter (trash)

The litter was collected, sorted and weighed for the two-cleanout period (Table 3). The sample included the litter that had been skimmed off each month as well as the litter retrieved from the mass of material removed by the vactor truck at the time of clean out. Although the amount of litter is small 0.90 m^3 (31.62t^3) when compared to the other material collected by the CDS unit 11.69 m^3 (413 ft^3), it is an eye sore and has the potential to impact wildlife as well as leach pollutants. Plastics were measured more often than any other category, but aluminum and Styrofoam were also found in significant quantities.

Table 3. Amount of litter collected in the CDS unit during the first year. It included over 45 kg (100 lbs) of assorted material, which measured a total volume of 0.90 m^3 (31.62 ft^3).

CATEGORY	APRIL 2003 CLEAN OUT				JULY 2003 CLEAN OUT				TOTAL FOR YEAR			
	KG	LB	M^3	FT^3	KG	LB	M^3	FT^3	KG	LB	M^3	FT^3
Plastic	13.98	30.66	0.25	8.79	19.31	42.34	0.33	11.76	33.29	73.00	0.58	20.55
Aluminum	1.65	3.63	0.04	1.52	2.68	5.88	0.05	1.75	4.34	9.51	0.09	3.27
Styrofoam	0.39	0.85	0.08	2.91	0.40	0.89	0.03	0.99	0.79	1.73	0.11	3.90
Miscellaneous	0.94	2.05	0.00	0.11	2.64	5.78	0.01	0.48	3.57	7.83	0.02	0.58
Wood	1.65	3.63	0.04	1.52	2.68	5.88	0.05	1.75	4.34	9.51	0.09	3.27
Paper	0.42	0.91	0.00	0.04	0.00	0.00	0.00	0.00	0.42	0.91	0.00	0.04
Glass	0.05	0.12	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.12	0.00	0.00
Cig. Butts	0.02	0.04	0.00	0.01	0.00	0.00	0.00	0.00	0.02	0.04	0.00	0.01
TOTALS	**19.09**	**41.87**	**0.42**	**14.90**	**27.71**	**60.77**	**0.47**	**16.72**	**46.81**	**102.64**	**0.89**	**31.62**

Conclusions

The CDS unit is effective for removing gross solids, including large size particles (>75 microns), trash and toxic levels of PAHs. It does not remaove significant amounts of suspended pollutants measured in the water column of most stormwater studies. This should be no surprise since storm water is not retained in the unit long enough for much biological treatment or sedimentation of small partices to take place. The CDS unit is an important first step in a treatment train incorporating several storm water treatment techniques. The CDS unit is effective for removing large sized partices, trash or PAHs. The CDS unit would probably have collected more material and shown better removal efficiencies if street sweepers were not used in the basin.

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Characterization of Stormwater System Sediments for Appropriate Disposal

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Abstract

Street sweepings, stormwater pond sediments, and catch basin sediments samples were collected from 20 different locations throughout Florida. The samples were analyzed for the following chemical parameters: volatile organic compounds (VOCs), semi-volatile organic compounds (SVOCs), pesticides, herbicides, metals, and leachable inorganic ions. The analytical methods followed established U.S. Environmental Protection Agency (US EPA) methods and other standardized analytical procedures. Both the total concentrations (mg/kg) and the leachable concentrations (mg/L) were measured. Results were compared to Florida Department of Environmental Protection soil cleanup target levels (SCTLs) and groundwater cleanup target levels (GWCTLs). The results of metal concentrations of more than 300 total samples found that arsenic concentrations in 105 samples exceeded the current residential SCTLs for direct exposure (0.8 mg/kg). All other metals typically fell below the analytical detection limits or were detectable but less than the SCTLs. Metal leaching was evaluated using the synthetic precipitation leaching procedure (SPLP). Metals in the majority of the SPLP extracts were found at concentrations less than GWCTLs. For the most part, the total concentrations of organic compounds were not a prevailing concern in regard to SCTLs for direct exposure. Organic leaching limits were exceeded in only a few samples. Secondary water quality parameters were also examined in several SPLP leachates, and aluminum, iron, and pH were found on occasion to exceed their respective GWCTLs.

Introduction

The management of residuals created by the maintenance of paved roads (street sweepings), stormwater ponds, and catch basins has been raised as an issue in Florida. These materials are collectively referred to here as "residuals." The two management practices most commonly employed for residuals management are direct landfilling and stockpiling for future use or disposal. The large soil content of these materials has prompted the desire to beneficially use them in an application such as clean fill. This objective, coupled with costs associated with landfill disposal, provides incentive to explore reuse options. Prior to reuse via land application, the chemical properties of the residuals must be assessed with regard to the potential environmental impacts when land-applied or reused. The University of Florida's Department of Environmental Engineering Sciences was contracted by the Florida Department of Environmental Protection (FDEP), the Florida Center for Solid and Hazardous Waste Management (FCSHWM), and a consortium of public agencies to perform the chemical characterization of residuals in Florida. This paper summarizes results of this project. For additional details, the reader is referred to the complete report (Townsend et al., 2002) as well as a summary and analysis of the data by the FDEP (FDEP, 2004a).

Material and Methods

Thirteen sampling trips were conducted over fifteen months (January 2001–March 2002) to facilities that produce residuals or to locations where these materials could be collected directly. Twenty different sampling locations were visited. In all cases, street sweepings were collected from piles (or roll-off containers) deposited by individual sweepers or dump trucks containing street sweepings. Pond sediments were collected directly from the stormwater ponds. Catch basin sediments were collected from materials emptied from vacuum collection vehicles; in some cases they were collected from the catch basins themselves. The land use category contributing to the residuals was noted where possible (e.g., residential, industrial).

Total content analyses (mg/kg) for metals (arsenic, barium, cadmium, chromium, copper, lead, mercury, nickel, selenium, silver, and zinc) and organics (volatile organics, semi-volatile organics, herbicides, and pesticides) were performed. When applicable, the results of total content analyses were compared the Florida SCTLs. Target levels are not regulatory standards with respect to land application of solid waste, but represent a set of risk-based goals used in the assessment of waste site cleanup. Further, the levels can be used voluntarily in lieu of a risk assessment by those who want to land-apply solid waste. A synthetic precipitation leaching procedure (SPLP) test was also performed to determine leachability of pollutants such as metals, organics, and secondary water quality parameters. The concentrations of chemicals detected in the SPLP extracts were compared to the Florida GWCTLs to assess potential leaching risks to groundwater. Some SPLP leachates were also analyzed (in addition to heavy metals and organic pollutants) for secondary water quality parameters. Leaching tests were performed on approximately one-half of the collected samples.

Results and Discussion

Results for both total and leaching analyses of street sweepings, stormwater pond sediments, and catch basin sediments are summarized as follows:

1. More than 300 residual samples were collected and analyzed (306 for Ag, 355 for As, 306 for Ba, 354 for Cd, 306 for Cr, 354 for Cu, 303 for Hg, 354 for Ni, 354 for Pb, 354 for Se, and 354 for Zn). The majority of the total sample concentrations (mg/kg) of silver, cadmium, mercury, and selenium fell below the instrument detection limits. Barium, chromium, copper, nickel, lead, and zinc were detected in more than half of the total samples but generally below the SCTLs. In almost half of the total samples analyzed, arsenic (178 amples) was detected, and 105 samples exceeded the arsenic SCTL for residential areas (0.8 mg/kg). Of the arsenic samples detected, 11 samples were above the industrial SCTL of 3.7 mg/kg. Table 1 present the results for the

stormwater pond residuals. Similar tables for the other residuals classes can be found in Townsend et al. (2002).

2. Three hundred and two samples were analyzed for the total concentration (mg/kg) of volatile organic compounds (VOCs). Of 74 VOCs target compounds tested, 12 compounds were detected in a few of the samples. None of the compounds in the samples exceeded the SCTLs for either residential or industrial settings.
3. The total concentrations (mg/kg) of semi-volatile organic compounds (SVOCs) were analyzed for 300 residual samples. Of 116 SVOCs tested, 17 compounds (primarily polycyclic aromatic hydrocarbons [PAHs] and phthalates) were found in a few. Three PAHs (benzo(a)anthracene, benzo(a)pyrene, and benzo(b)fluoranthene) were detected above the SCTLs for residential and industrial limits in two samples (one sample from street sweepings and one from catch basin sediments). The sample from catch basin sediments also contained other PAHs, such as anthracene, benzo(ghi)perylene, benzo(k)fluoranthene, and indeno(1,2,3-cd)pyrene. The concentration of benzo(k)fluoranthene in the sample exceeded the SCTLs for residential areas, and the concentration of indeno(1,2,3-cd)pyrene was found above both residential and industrial SCTLs. No phthalate compounds detected exceeded the respective SCTLs.
4. The total concentrations (mg/kg) of organochlorine pesticides (OCl Pest) were analyzed for 323 samples. Of 43 target pesticide compounds, 14 were detected in a number of samples. Two OCl Pests, 4,4'-DDT and Endosulfan II, were found in 66 and 44 samples, respectively. Neither compound exceeded their respective SCTL. Only one compound, dieldrin, exceeded the SCTLs in four samples; three exceeded the residential SCTL limit of 70 μg/kg, and one exceeded the industrial SCTL limit of 300 μg/kg.
5. No nitrogen-phosphorus pesticides were found above the detection limit (0.25 mg/kg) in any of the 314 total samples.
6. SPLP leaching tests were performed to examine the "leachable" concentration (mg/L) of 11 metals (arsenic, barium, cadmium, copper, chromium, lead, mercury, nickel, selenium, silver and zinc). At a minimum, 150 SPLP leachate samples were analyzed for each metal (150 for Ag, 185 for As, 150 for Ba, 178 for Cd, 150 for Cr, 184 for Cu, 169 for Hg, 184 for Ni, 184 for Pb, 154 for Se, and 184 for Zn). Four metals (arsenic, barium, lead, and zinc) were detected above the respective detection limits in a number of samples (27 for As, 78 for Ba, 50 for Pb, and 44 for Zn). Of 50 samples detected for lead, eight exceeded the GWCTL for lead (0.015 mg/L). None of the other three metals exceeded its respective GWCTL. Cadmium, chromium, copper, and nickel were detected above the detection limits in a few samples (3 for Cd and Cr, 2 for Cu, and 3 for Ni). One out of three detected samples exceeded the GWCTL for cadmium (0.005 mg/L). Of 184 samples, nickel was found in three samples, all of which exceeded the GWCTL limit of 0.1 mg/L.

7. A SPLP tests were also performed to examine leachability of organic compounds (VOCs, SVOCs, OCl Pests, nitrogen-phosphorus pesticides, chlorinated herbicides, and N-methylcarbamates). One hundred and fifty-five SPLP leachates were analyzed for VOCs. Nine VOC compounds were detected in three samples above the detection limit of 5.0 μg/L. Four compounds (1,4-dichlorobenzene, naphthalene, 1,3,5-trimethylbenzene, and o-xylene) were found in two samples above the GWCTLs of their respective analytes. Several solvents used in the SVOC and pesticide analysis were detected in the SPLP leachates, but these were also found in many of the blanks and are thus believed the result of contamination.
8. One hundred and forty-seven SPLP leachates were analyzed for SVOCs. No acid and base/neutral SVOC compounds were detected above the detection limit of 10 μg/L in any of the samples. No nitrogen-phosphorus pesticides and N-methylcarbamates were found in any of the SPLP extracts from 132 samples and 176 samples, respectively.
9. One hundred and sixty-six leaching samples were analyzed for OCl Pests. Out of 43 target OCl Pests, three compounds were detected above the detection limit of 0.05 μg/L in a few samples: 4,4'-DDT in 13 samples, beta-BHC in 7 samples, and Endosulfan II in 1 sample. The concentrations of 4,4'-DDT in all detected samples exceeded the GWCTL of 0.1 μg/L. No GWCTLs are available for the other two detected compounds.

Thirty SPLP leachate samples were analyzed for secondary water quality parameters. The secondary parameters included aluminum, chloride, copper, ethylbenzene, fluoride, iron, manganese, pH, silver, sulfate, toluene, total dissolved solids (TDS), xylenes, and zinc. Aluminum was detected above the detection limit in 20 leaching samples, all of which exceeded the secondary standard for drinking water (0.2 mg/L). Iron concentrations, detected in 8 samples, exceeded the secondary standard concentration of 0.3 mg/L. The concentrations of iron ranged from 0.32 to 2.22 mg/L, with an average concentration of 0.88 mg/L. Results of pH in leaching samples ranged from 7.00 to 9.11, with an average of 7.99. Nine samples showed greater pH than the secondary standard of pH 6.5 to 8.5. No other ions, organics, or other metals exceeded the secondary standard limits for drinking water. Several samples of natural soil were collected and leached using the SPLP. Many of these samples showed concentrations of Al and Fe above their respective GWCTLs. The source of these metals likely was the soil in the residuals.

Conclusion

The analysis of street sweepings, stormwater pond sediments and catch basin sediments in Florida found most contaminants to be below risk-based direct exposure and leaching thresholds established by FDEP. However, a few chemicals were encountered above the risk thresholds at times. The results of this work were used by the FDEP to develop a guidance policy for the management of these residuals (FDEP, 2004b). The guidance document permits disposal of most classes of these residuals in Class III landfills. The guidance document also outlines procedures necessary for beneficial reuse of these materials outside of the landfill.

Table 1 Total Metal Concentrations of Sediment Samples from Stormwater Ponds

Element	No. of Samples	No. of Detects	Ave. Concentration (mg/kg)	Max.	Min.	Standard Deviation	No. of Exceedance (Resid)	No. of Exceedance (Ind.)	Residential SCTLs (mg/kg)	Industrial SCTLs (mg/kg)	Detection Limit (mg/kg)
Ag	68	0	--	--	--	--	0	0	390	9100	0.80
As	74	43	2.4	24.8	0.6	3.90	30	5	0.8	3.7	0.50
Ba	68	61	72.8	1019	8.1	144.6	8	0	110	87000	1.35
Cd	73	1	5.3	5.3	5.3	--	0	0	75	1300	0.37
Cr	68	50	26.6	174.5	5.8	29.88	0	0	210 (Cr VI)	420 (Cr VI)	1.34
Cu	73	73	18.3	90.4	4.5	17.03	0	0	110	76000	1.84
Hg	68	0	--	--	--	--	0	0	3.4	26	0.02 (μg/kg)
Ni	73	73	10.3	40.4	5.4	4.86	0	0	110	28000	1.72
Pb	73	46	46.8	196	5.6	47.78	0	0	400	920	1.43
Se	73	4	10.5	14.1	7.5	2.91	0	0	390	10000	0.25
Zn	73	73	94.9	711	5.4	136.1	0	0	23000	560000	1.35

Note: Average concentration and standard deviation were calculated using only the detected samples.

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Urban Soil Compaction and its Effect on Stormwater Runoff

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Abstract

Inadvertent soil compaction during urban development reduces infiltration rates. Reduced soil infiltration rates cause increased ponding and increased stormwater runoff. This is particularly important when modeling and implementing stormwater management plans for urban areas. The effect of compaction on infiltration rates on sandy soils in North Central Florida was quantified across various levels of compaction. Average non-compacted and compacted infiltration rates were measured for natural forest, for planted forest, and for pasture sites. Although there was a wide variability in infiltration rates across both compacted and non-compacted sites, construction activity or compaction treatments reduced infiltration rates 70-97%. This implies that construction activity in this region increases the potential for runoff and the need for large stormwater conveyance networks, not only due to the increase in impervious area associated with development but also because the compacted pervious area effectively approaches the infiltration behavior of an impervious surface.

Introduction

Soil compaction is associated with urban area development. This compaction can be because of controlled compacting of a site to increase the structural strength of the soil or it can be inadvertently caused by the use of heavy equipment and grading of lots. Soil compaction affects physical properties of soil by increasing its strength and bulk density, decreasing its porosity, and changing the distribution of pore size within the soil. These changes affect the way in which air and water move through the soil and the ability of roots to grow in the soil (NRCS 2000).

Changes to the way that air and water move within the soil can affect infiltration rates. A decrease in infiltration rates will cause increased runoff volumes, greater flooding potential and reduced groundwater recharge within watersheds.

Compaction has a significant influence on such soil hydraulic properties as soil water retention, soil water diffusivity, unsaturated hydraulic conductivity and saturated hydraulic conductivity (Horton et al. 1994). These hydraulic properties in turn govern infiltration rates. Agricultural research has found compaction due to vehicular traffic to be responsible for more than a 75%-reduction in infiltration rates (Li et al. 2001; Sheridan 2003).

Research conducted on the effect of compaction in urban areas has generally consisted of surveys that have measured infiltration rates within urban areas and then compared these data based on methods of land development, land types, or levels of compaction. Research into the effects of soil compaction on infiltration rate has been conducted in Pennsylvania (Felton and Lull 1963; Hamilton and Waddington 1999), Wisconsin (Kelling and Peterson 1975), North Carolina (Kays 1980) and Alabama (Pitt et al. 1999). These studies have shown that soil infiltration rates are negatively affected by the compaction associated with urban development.

The objectives of this research were as follows: 1) quantify the effect of compaction due to construction activities on infiltration rates of typical urban development sites in North Central Florida and 2) determine if the changes in infiltration rate have an effect on the generation of stormwater.

Materials and Methods

Compaction Due to Construction Activities

A natural, mixed wood forest site in the Madera subdivision of Gainesville, Florida was chosen as a research site. Lot 24 was used as an access to a detention pond and for parking heavy construction vehicles. Lot 24 was made up of areas that had been compacted due to construction vehicle traffic and areas that were undisturbed due to the wooded conditions. Lots 2 and 8 of the Madera development were undisturbed lots that had not been cleared or subjected to vehicle traffic. Madera lots 2, 8, and 24 will be referred to as "natural wooded" sites 2, 8, and 24.

Phase eight in the Mentone development of Gainesville, Florida was also chosen as a research site. The predevelopment vegetation was planted slash pine (*Pinus elliottii*), which was at least 10 years old. Compaction testing was carried out on Lot 857 and Lot 818. Lot 818 had been partially cleared to allow access for the construction of one of the detention ponds. Lot 857 had been used to park heavy construction equipment and was used by construction vehicles as a shortcut between adjacent streets. Both lots were made up of areas that had been compacted and areas that were undisturbed. Mentone Lots 818 and 857 will be referred to as "planted forest" sites 818 and 857.

Pre-development and post-development infiltration tests were carried out on wooded site 2 in December 2002 and May 2003. Infiltration rates were measured at four locations on the turf area in the front yard and four locations on the turf area in the backyard. These infiltration tests were carried out using the previously described procedure. Cone index was also recorded near each infiltration test.

Infiltration rates were measured using a constant-head double-ring infiltrometer with ring diameters of 15 and 30 cm. The infiltration tests were conducted for at least 40 min or until the infiltration rate became constant. Infiltration rates were calculated and fitted to the Philip's infiltration equation as follows:

$$I = Kt + St^{1/2} \tag{1}$$

where I is cumulative infiltration depth (mm), K is the saturated hydraulic conductivity (mm/hr), t is time (hr), and S is soil water sorptivity (mm/hr$^{1/2}$). Values of the parameters K and S can be found by regressing the cumulative infiltration data collected in the field to Eqn. 1 (Lal and Vandoren 1990). The parameter K from the Philips infiltration equation can be used as an approximation for the steady state infiltration rate as time increases (Chow et al. 1988).

Soil bulk density and volumetric moisture content were determined using a standard intact core method (ASTM 2002c; Blake and Hartge 1986; ASTM 2002b; Gardner 1986). The cone index (ASAE 2000) was also measured using a Spectrum ™ SC900 Soil Compaction Meter (Spectrum Technologies, Inc., Plainfield, Illinois) which records cone index at depths of 2.5 cm up to 45 cm.

Infiltration, cone index, and bulk density measurements were conducted on "natural wooded" site 24 and the "planted forest" sites 857 and 818. The testing was carried out between February and July 2003. On each lot, 12 locations were selected for testing. These locations were selected so that they could be grouped in pairs with each pair consisting of a location that appeared to be undisturbed and a location with obvious compaction. There was a maximum distance of 2 m between the locations making up the pair. Undisturbed areas were separated from trafficked areas due to trees. On the "planted forest" site 818 the cone index was measured at only eight of the locations due to clearing operations having destroyed four of the sites. A particle size distribution analysis was conducted using the hydrometer method on five soil samples collected randomly on each lot (Gee and Bauder 1986).

Effects of Compaction Level on Infiltration Rates

Controlled Compaction. An existing cattle pasture at the University of Florida Plant Science Research and Education Unit (PSREU) near Citra, Florida was used for a compaction trial. The pasture area had been subjected to traffic particular to this land use for at least 20 years. This site was chosen because it represents pastures in Florida that are being converted to residential subdivisions and will be referred to as the pasture site.

A controlled compaction trial was carried out on the pasture site in February 2004. An area of the pasture approximately 5 m long by 2.5 m wide was cleared of the top 10 cm of grass roots. A mechanical grader was used to clear a 1.2-m width and the rest of the plot was manually cleared with a shovel. This area was then divided into 16 subplots each 0.6 m by 1.2 m; the wheel tracks of the grader were excluded from the sub plots. Four levels of compaction treatment were then applied in a Latin Square experimental design. A Mikasa GX100 (MT-65H) (Mikasa Sangyo Co., Ltd) 'jumping jack' type compactor was used to apply the levels of compaction. The compactor was moved about the subplots in a steady manner to achieve a uniform level of compaction. The four levels of compaction were 0 minutes of compaction (control), 30 seconds of compaction, 3 minutes of compaction, and 10 minutes of compaction. Infiltration rate, bulk density, soil moisture content, and cone index were measured as described previously. Also, a Proctor density test (ASTM 2002a) was conducted on a soil sample from the site.

This experimental procedure was then repeated in an undisturbed area on “natural wooded” site 8. The plot was located in a clearing in a wooded area and the top 10 cm of organic material and soil was manually cleared using a shovel.

Vehicular Compaction. A pasture area at the PSREU was selected and a mechanical grader was used to remove the top 10 cm of grass and soil from three plots each about 18 m long and 1.2 m wide. It took four passes of the grader to remove the grass roots and soil and care was taken to ensure that the grader traveled in the same wheel tracks for each pass, thus ensuring that there was minimal compaction within the plots.

Three vehicles that are commonly used in urban construction were used for the vehicular compaction trial treatments. These vehicles were an all-wheel-drive Caterpillar 416B backhoe weighing 6.3 Mg with a front tire pressure of 206 kPa and a rear tire pressure of 310 kPa; a dump truck with a front axle weight of 6.0 Mg, a total load of 18.4 Mg on the two rear axles and tire pressures of 310 kPa; and a pickup truck with a front axle load of 1.1 Mg, a rear axle load of 0.8 Mg and a tire pressure of 275 kPa. Each vehicle was driven, at walking speed, along a plot with one wheel running down the middle of the plot and the other outside of the plot. Nine passes of each vehicle were made in the plots. Four measurements of infiltration rate, soil bulk density, and volumetric soil moisture content as described previously were taken in each wheel path.

Effect of Compaction on Runoff

To evaluate the effect of compaction on the generation of runoff, a simple infiltration/runoff simulation was undertaken. The simulation used the Philip’s infiltration model to calculate potential infiltration at 1-minute intervals. It was assumed that the soil was homogenous, that the water table was sufficiently below the soil surface to have no effect on the infiltration, and that the soil water content near the surface was constant and near saturation throughout the rainfall event. It was also assumed that any rainfall that did not infiltrate into the soil profile, during a time step, became runoff. A rainfall event (with a magnitude of 124 mm) that had been recorded in Lake County, North Central Florida was used for the simulation.

The Philips parameters used in the simulations were varied to simulate the four levels of compaction as measured on the "natural wooded" site 8.

Results

Compaction Due to Construction Activities

Infiltration tests were conducted across soil moisture ranging from 5-12% by volume and there was no relationship between soil moisture and infiltration rate. Field capacity values for the soils tested in this project were in the 7-10% range and all sites tested were well drained.

Table 1 summarizes the pre-development and post-development infiltration rates measured on the "natural wooded" site 2. The pre-development infiltration rates were measured in approximately the same location as the post-development infiltration rates. The front and back yard measurements for both the pre-development conditions ($t = 3.596$ and $p = 0.037$) and post-development conditions ($t = 4.099$ and $p = 0.026$) had a statistical difference. There were strong significant differences between the infiltration rates for the pre-development and post-development conditions for both the front yard ($t = 7.735$ and $p = 0.004$) and back yard ($t = 6.511$ and $p = 0.007$). Construction activity resulted in an 80% decrease in infiltration rates on the front yard and a 97% decrease in infiltration rates on the back yard.

The pre-development cone index data for the front yard and back yard showed a maximum cone index of 858 kPa and 1104 kPa, respectively. The post-development data for the front and back yard showed a maximum cone index of 4260 kPa and 4382 kPa, respectively. This change in cone index during development of the lot was due to compaction that occurred during the construction process. The maximum cone index in the front yard occurred at 37.5 cm deep while the maximum compaction on the back yard occurred at 27.5 cm deep. The fill that was brought onto the front of the site, for grading purposes, resulted in this 10-cm difference in depth of maximum cone index.

Table 1. Mean pre-development and post-development infiltration rates and CV for the front and back yard on "natural wooded" site 2 where values represent a mean of four observations.

	Infiltration Rate (mm/hr)	
	Pre-development (%)	Post-Development (%)
Front Yard	861 (25)*	175 (48)
Back Yard	590 (31)	8 (41)

*Coefficient of variation in parentheses.

Table 2 summarizes the infiltration rate and bulk density results for the compaction tests carried out on "natural wooded" site 24 and "planted forest" sites 818, and 857. These results show that compaction caused an overall decrease in the infiltration rate, from 733 mm/hr to 178 mm/hr and a corresponding increase in bulk density, from 1.34 g/cm^3 to

1.49 g/cm^3. These overall changes are statistically significant with $p < 0.001$ for the overall infiltration results and $p = 0.001$ for the overall bulk density results. These data support the hypothesis that compaction caused by the vehicular traffic during construction of urban developments results in a significantly increased bulk density and a significantly decreased infiltration rate.

The soil on sites 24, 818, and 857 was classified as sand according to the USDA soil textural classification (Soil Survey Staff 1975). All of the samples analyzed showed a sand classification except for one sample on Lot 24 that was classified as a loamy sand.

The "natural wooded" area and the "planted forest" were different land uses with the wooded area being made up of mixed tree species and the pre-development soil being subjected to very little compaction. The "planted forest" would have been subjected to planting and harvesting activities involving heavy equipment that would have caused compaction. The significant difference ($t = 3.03$, $p = 0.008$) between the mean undisturbed infiltration rates on the "natural wooded" site (908 mm/hr) and the "planted forest" sites (631 mm/hr) was therefore expected; however, there was no significant difference between the undisturbed bulk densities ($t = 1.54$, $p = 0.144$). The lack of a significant difference in bulk densities could be due to the soil core samples being collected in the top 10 cm of the soil profile. The effect of compaction is greatest at depths below 30 cm (Hakansson and Petelkau 1994); therefore, the soil samples collected in the top 10 cm might not show this effect. It is also interesting to note that after compaction there was no statistical difference ($t = 0.33$, $p = 0.746$) in the infiltration rates and bulk densities measured on the "natural wooded" site or those measured on the "planted forest" sites ($t = 0.59$, $p = 0.563$). This indicates that although land use before development on these sites had an effect on infiltration rates, compaction during development resulted in no significant difference in infiltration rates.

Table 2. Average infiltration rates, bulk density, and CV from "natural wooded" site 24, "planted forest" site 818 and 857 (n = 6 for each site and each compaction level).

Lot	Mean Infiltration Rate (mm/hr)		Bulk Density (g/cm^3)	
	Undisturbed (%)	Compacted (%)	Undisturbed (%)	Compacted (%)
818	637 (22.7)	187 (52.4)	1.20 (17.2)	1.48 (5.0)
857	652 (26.9)	160 (52.0)	1.40 (6.5)	1.52 (9.3)
24	908 (23.2)	188 (50.1)	1.42 (4.1)	1.47 (7.1)
Average	733 (28.8)	178 (49.1)	1.34 (12.1)	1.49 (7.1)

Effect of Compaction Level on Infiltration Rates

Controlled Compaction. The mean infiltration rates on non-compacted subplots were significantly higher than the mean infiltration rates on the compacted subplots (Table 3). There was also a significant difference between the non-compacted infiltration rates on the pasture (225 mm/hr) and on the wooded area (487 mm/hr). However, the two locations had the same textural soil classifications (sand) and the same non-compacted mean bulk densities (1.49 g/cm^3).

Statistically significant differences were not found between the mean infiltration rates of 65 mm/hr, 30 mm/hr and 23 mm/hr that occurred after 30 sec, 3 min, and 10 min of compaction on the pasture. This suggests that when describing infiltration rates with respect to compaction, this soil could be classified as either compact or non-compact. A similar trend was observed with the data from the wooded site; however, a statistically significant difference was found between the 30-sec treatment (79 mm/hr) and the 10-min treatment (20 mm/hr). Similarly, for the wooded site, with respect to infiltration rate, the soil could also be generally classified as either compact or non-compact.

The mean bulk densities after 10 min of compaction were significantly different between the pasture and the wooded locations. This can be explained because the maximum Proctor density of 1.89 g/cm^3 on the previously wooded site compared to the maximum Proctor density of 1.83 g/cm^3 for the pasture indicates that the previously wooded site can be compacted to a higher level. The bulk density of the pasture soil after 10 min of compaction was 1.73 g/cm^3. This equates to approximately 95% of the maximum Proctor density, and the bulk density of the soil at the wooded area after 10 min of compaction was 1.79 g/cm^3, which also equates to 95% of the maximum Proctor density.

Table 3. Average infiltration rates and bulk density from "natural wooded" site and pasture site under four levels of compaction (n = 4 for each compaction level).

Treatment	Mean Infiltration Rate (mm/hr)		Bulk Density (g/cm^3)	
	Pasture Site	Natural Wooded Site	Pasture Site	Natural Wooded Site
T0	225	587	1.49	1.49
T0.5	65	79	1.61	1.67
T3	31	52	1.68	1.71
T10	23	20	1.73	1.79

Vehicular Compaction. Table 4 summarizes the mean infiltration rates and bulk density data collected in the wheel ruts created during the vehicular compaction trial. An ANOVA indicated no significant difference between mean infiltration rates in the backhoe tracks and in the pickup tracks, although the backhoe tracks did show a numerically lower mean infiltration rate (59 mm/hr) than the pickup (68 mm/hr). Both the backhoe and pickup resulted in significantly higher mean infiltration rates than the dump truck (23 mm/hr).

There were no significant differences between the mean bulk densities for the three treatments, although the dump truck did result in a numerically higher mean bulk density (1.68 g/cm^3) than the backhoe and pickup (1.61 g/cm^3). The lack of a significant difference between the mean bulk densities may be due to the bulk density being determined from soil samples collected in the top 10 cm of the soil profile. It was also likely that the pasture site was subjected to compaction before these tests, which would have reduced the effect of the tests on bulk density.

Table 4. Mean infiltration rate and bulk density result from tests conducted in the wheel ruts of a dump truck, backhoe, and pickup after nine passes over a graded pasture. Means that were not significantly different were grouped with the same letter (n = 4 for each vehicle).

	K (mm/hr)	CV (%)	Bulk Density (g/cm^3)	CV (%)
Dump truck	23^b	43.9	1.68^a	2.3
Back hoe	59^a	14.1	1.61^a	1.9
Pickup	68^a	23.1	1.61^a	2.5

Effect of Compaction on Runoff

The results of the infiltration/runoff simulation are summarized in Table 5. The level of compaction treatment on "natural wooded" site 8 had a significant effect on the generation of runoff during the simulated storm event. When no compaction was applied to the soil, no runoff was generated and all the rainfall (124 mm) was infiltrated. When the level of compaction was increased to 30 seconds, 3 minutes, and 10 minutes, the runoff increased to 48, 67, and 95 mm respectively. These results show that the reduced infiltration rates caused by compaction do have an effect on infiltration that should be considered.

Table 5. Simulated infiltration and runoff from "natural wooded" site 18, under four levels of soil compaction.

Compaction Level	Infiltration (mm)	Runoff (mm)
T0	124	0
T0.5	76	48
T3	57	67
T10	29	95

Conclusions

Soil compaction was shown to have a negative effect on infiltration rates of soils in North Central Florida. On these sandy soils, the level of compaction was not as important as whether a soil had been compacted or left undisturbed, although it was shown that there could be a significant difference between the effect of compaction caused by relatively light construction equipment (i.e. a backhoe and pickup) and very heavy equipment (i.e. a fully loaded dump truck). Therefore, when classifying the soil infiltration rate it is important that the history of compaction of the soil be taken into account. This classification of the compaction of a soil could have a significant effect on hydrological and stormwater modeling. Accurate infiltration rate information is also important in traditional runoff estimation because undisturbed soil infiltration rates are typically assumed for pervious areas. Overestimation of the soil infiltration rate would result in an underestimation of the runoff from a specified area and a resultant underestimation of a flooding event.

It could also be recommended that maintaining pre-development infiltration rates on a lot could be considered a best management practice that reduces runoff. Demarcating areas of the development to prevent compaction of the soil would help maintain pre-development infiltration rates. Special efforts should also be made to leave natural areas undisturbed as these were shown to have the highest infiltration rates. Reducing the use of very heavy equipment on the lot as much as possible would also help limit the reduction in infiltration rates caused by compaction.

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Enhanced CREP and VEGBACC: Landowner Incentive Programs for Temperature Management

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Abstract

Clean Water Services (CWS) is a wastewater treatment and stormwater utility that serves approximately 500,000 customers in Washington County, Oregon. The Tualatin River, which runs through CWS's service area, is subject to a TMDL that established a temperature waste load allocation for its wastewater treatment facilities. To meet the allocation, CWS found that it would be far less expensive to increase shade along streams than to make capital improvements at its treatment facilities.

CWS then negotiated the nation's first watershed-based NPDES permit with Oregon DEQ. The permit allows CWS to use shade as an offset for the increases in stream temperature attributed to its wastewater treatment facilities. The permit includes a temperature management plan under which Clean Water Services receives temperature reduction credit based on the shade that is expected to exist twenty years after trees and shrubs are planted. An estimated 35 miles of stream shade will be needed for this purpose.

Shade is established using landowner incentive programs. Two of these programs, the Enhanced CREP and VEGBACC, are aimed at agricultural landowners outside CWS's service area. Incentives offered include annual rental payments, bonus payments, free site clearing, planting and maintenance, and the purchase of conservation easements and water rights. The development and implementation of the programs involved several partners, including the USDA, the Oregon Department of Agriculture, the Oregon Department of Forestry, the Tualatin Soil and Water Conservation District, and the Oregon Water Trust.

Introduction

The focus of water quality regulation was once limited to point sources. After large investments in point source control during the 1970's and 1980's, water quality in many of the nation's rivers and streams significantly improved. A large and growing number of water bodies continue to be water quality limited, however, and it is well known that nonpoint source pollution is a major reason. Nonpoint sources are by definition diverse, and this makes control difficult. To improve nonpoint source management, some water quality agencies have adopted a watershed perspective of the problem, and have begun to use nontraditional approaches such as landowner incentive programs and effluent trading. Although their innovative nature makes them a challenge to design, these programs offer the possibility of community consensus, multiple environmental benefits, and low relative cost.

This paper highlights a nontraditional approach that involves two landowner incentive programs, explains how the programs were developed, and how they are designed to meet the temperature requirements of the nation's first watershed-based NPDES permit. These programs may be an indicator of things to come as water quality management increasingly reflects innovative approaches and the watershed frame of reference.

Background

Clean Water Services is a special service district that provides sanitary sewer service and stormwater management to the 500,000 residents of urban Washington County, Oregon. CWS operates two tertiary sewage treatment facilities that discharge effluent to the Tualatin River, a tributary of the Willamette River. The Tualatin River watershed, which lies almost entirely within Washington County, is typical of those located at the urban-rural interface: urban growth has eclipsed much of the once dominant agriculture and forestry sectors, but these remain an important part of the social fabric and local economy. Washington County, which is home to most of Oregon's high technology firms, contains most of Portland's west side suburbs and is the second most populous county in Oregon, yet it continues to rank in the top four in farm income, and contains thousands of farms and forested parcels.

Land area in the watershed is divided roughly into thirds between urban, agricultural, and forest uses. Most watershed streams originate in forested areas, flow through agricultural areas, and finally urban areas before entering larger streams. Local streams support steelhead and cutthroat trout, two cold water fish species that represent the most sensitive beneficial uses. In urban and agricultural areas, much of the shade-producing native vegetation has been removed, resulting in warmer stream temperatures during the summer. In addition, the effluent from Clean Water Services' sewage treatment plants adds 50 million gallons per day of warm water to the Tualatin River.

Regulatory Context

To address the temperature problem, the Oregon Department of Environmental Quality (DEQ) issued a temperature TMDL for several watershed streams in 2001. In response, CWS evaluated various approaches to temperature management, and found that the cost of the traditional approach, which would involve installing refrigeration units at its wastewater treatment facilities, would cost \$60 - \$150 million. In addition, annual electricity costs would be \$2.5 - \$6 million. Replacing lost shade on streams, on the other hand, would cost less than \$10 million. Clearly, the latter alternative was preferable if the details could be worked out.

During 2002, CWS and DEQ began to negotiate a new NPDES permit for CWS's wastewater treatment facilities. From the beginning, it was understood that the desired outcome would be a permit that allowed CWS to use a nontraditional approach to meet temperature requirements. The new permit, which was issued in 2004, allows shade to be used to offset the excess heat load attributed to the wastewater treatment facilities during the summer. It is considered a "watershed-based" permit because CWS will receive credit for shade produced throughout the watershed (even though CWS's service area is limited to the urban portion). The permit leaves the details of how shade credit will be earned to CWS's Temperature Management Plan, which was developed after the permit was issued.

The Temperature Management Plan[5] (TMP) sets up a flexible baseline for measuring the allowed and excess heat loads attributed to CWS's wastewater treatment facilities. The baseline is developed annually and is based on actual river temperature and flow, effluent temperature and flow, weather and other conditions during the warmest part of the summer. The amount of heat load that exceeds the annual baseline must be offset using shade and/or other means.

The TMP quantifies shade using a typical summer solar insolation value of 480 kcal per square foot of stream surface per day. Complete shade would block this amount of energy from reaching the stream surface. In reality, mature vegetation, even on narrow streams, usually blocks 70-90% of shade. Shade values are calculated using the Shade-A-Lator module of the Heat Source temperature computer model.[6] The Shade-A-Lator calculates the percentage of shade over a stream reach using the following data: day of the year, time of day, latitude and longitude, stream aspect, and stream and riparian vegetation characteristics.

One of the challenges in developing the TMP was the long time-frame for shade creation—how can permit compliance be achieved during the five-year permit period if it will take decades for new vegetation to grow? The TMP addresses this issue by giving credit at the time vegetation is planted based on the estimated shade produced by the vegetation in twenty years (i.e., the estimated 20-year vegetation height and density is entered into the Shade-A-Lator module.) To compensate for providing credit today for something that will happen in twenty years, the TMP requires CWS to plant twice as much shade during the permit period as it will take to offset its excess heat load.

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[5] The Temperature Management Plan was awaiting the approval of Oregon DEQ at the time of this writing. If changes will be required, they are expected to be minor.

[6] Heat Source was developed at Oregon State University and is in the public domain.

The flexible baseline creates a challenge for determining permit compliance because the amount of shade needed will vary from year to year. In addressing this issue, the TMP assumes that the *average* annual excess heat load over the five-year permit period will be typical. The TMP also establishes annual benchmarks for increases in shade. The benchmarks are percentage increases in the amount of heat load offset by shade. The benchmarks are as follows:

Shade Benchmarks

Permit Year	Annual Shade Credit Benchmark	Estimated Stream Miles Planted
1	10%	3.5
2	20%	7.0
3	30%	10.5
4	20%	7.0
5	20%	7.0
Total	100%	35

As the table indicates, at the end of the fifth and last year of the permit, the total annual increases in the amount of excess heat load offset by shade must equal 100%. An estimated 35 miles of riparian area will have been planted when this occurs.

Under the TMP, shade serves as a proxy for reduced stream temperatures. Shade and stream temperatures will be monitored during the 20-year shade credit period to determine the actual relationship between the two.

Shade Program Development

To develop landowner incentive programs aimed at creating shade, CWS assembled a technical advisory committee (TAC) that included members from various backgrounds, including farming. The TAC's first task was to review existing riparian restoration programs, none of which had been successful in the Tualatin River Watershed. The TAC then determined that the best course of action would be to create a new program by modifying the USDA's Conservation Reserve Enhanced Program (CREP). CREP had several features that made it an attractive candidate: it could be applied throughout the agricultural part of the watershed; it had plenty of funding, and it could be modified for use at the local level. In addition, it was the subject of two reports that identified reasons why it had not been successful (Viatella and Rhee, 2002). The reports indicated that low program payments and a lack of landowner assistance were major shortcomings.

In developing the modified CREP, which was named "Enhanced CREP," the TAC considered the information contained in the reports and listened to the concerns of the local farm community. This included meetings with farmers and an informal opinion survey. In addition to confirming the validity of the recommendations contained in the reports, the farm community also indicated a preference for flexibility, including various program benefit options. In response, the TAC decided to develop an additional program that would serve as an alternative to Enhanced CREP. This program, called VEGBACC, would provide fewer economic benefits to landowners, but would also place fewer constraints on farm activities.

Enhanced CREP Program Description

Landowners who are enrolled in the current version of CREP receive annual payments, can be eligible to receive bonus payments, and receive assistance with the cost of restoring stream buffer areas. In developing the Enhanced CREP, the TAC determined that the typical local landowner who enrolls in the current CREP might receive annual payments that equal foregone farm income, but would not be compensated for leasing water rights to the State for in-stream use (a CREP requirement), would shoulder at least 25% of the cost of buffer restoration, and would be responsible for virtually all of the cost of maintenance. In addition, at the end of the CREP contract when the landowner would no longer receive CREP benefits, it would be expensive to clear the buffer area again for farm use. Given these limitations, it is not surprising that no landowners in the watershed had enrolled in the program during its five-year history.

In developing the Enhanced CREP, the TAC wanted to reduce or eliminate each of the known problems with CREP. Annual payments for irrigated cropland were increased from $235 per acre to $393 per acre. With the addition of the Oregon Water Trust as a program partner, some water rights holders would receive water rights leasing payments of $20 or $30 per acre per year, depending on the size of the enrolled parcel, and could also permanently transfer their water rights for in-stream use. The price paid for permanent transfers would be as much as $675 per acre. Landowners would also have the option of granting 20-year, 30-year or permanent conservation easements to the Tualatin Soil and Water Conservation District (TSWCD). Payment for permanent easements was set at 30% of the net present value of the highest and longest possible annual payment stream under an Enhanced CREP contract. Payment for 30 and 20-year easements was set at 75% and 50% of the price of a permanent easement respectively. Finally, landowners would have the option of transferring responsibility for buffer restoration and maintenance to TSWCD. If they did this, they would incur no costs for these items.

VEGBACC Program Description

"VEGBACC" is an acronym for Vegetated Buffer Areas for Conservation and Commerce." The idea for VEGBACC was suggested by a local farmer, who indicated that some landowners would prefer a program that provided a high level of management flexibility and lack of constraints even if this meant lower benefits. The TAC designed VEGBACC so that it would appeal to these people. There would be no minimum period that land would have to be in the program, and planting materials, including trees, shrubs, tree and shrub protection devices, mulch and grass seed, would be free of charge. Although landowners would be responsible for site clearing and planting work, they would be provided with a conservation plan and technical assistance. They would also be responsible for maintenance, but would have the option of transferring responsibility for this work during the first five years to TSWCD. If they did this, they would be responsible for 50% of the cost, subject to a $200 per acre per year maximum. The conservation easement and water rights options available under Enhanced CREP would also be available.

As its name indicates, VEGBACC was designed for commerce as well as conservation. Unlike CREP and Enhanced CREP, which prohibited commercial use of restored buffer areas, VEGBACC would allow landowners to make certain commercial uses as long as they did not significantly compromise buffer area ecological functions. The TAC determined that selective tree harvesting, haying, and certain agroforestry practices would qualify, but acknowledged that additional work was needed to determine other allowable uses. The TAC also developed a program element under which landowners who granted conservation easements could sell pole cuttings to TSWCD under multi-year contracts. "Pole cuttings" are branches cut from certain tree and shrub species that can be used to grow new specimens. Eligible varieties include three species of willow, red osier dogwood, pacific ninebark, twinberry, three species of rose, cottonwood, salmonberry and thimbleberry. Pole cuttings are less expensive than potted or bare root plants, but are frequently in short supply at commercial nurseries. A pole cuttings contract would provide TSWCD with a reliable, low cost source of plant material for shade projects, and would provide a significant income source for landowners.

Although VEGBACC contained fewer landowner benefits than Enhanced CREP, landowners who enrolled in it would be allowed to participate in other incentive programs, and thereby further leverage their costs. One such program is the USDA's Environmental Quality Incentives Program (EQIP), which pays a percentage of the cost to restore riparian buffers, and offers incentive payments for certain management practices as well.

Cost Issues

As indicated previously, the primary reason for developing Enhanced CREP and VEGBACC was to manage stream temperature. It was known at the outset that these programs would have enormous cost advantages over the primary technological alternative, which involved installing refrigeration equipment at CWS's wastewater treatment facilities. But how much *should* they cost? Prudent management required that they cost no more than necessary to achieve their goals, but the lack of program precedents made cost estimation difficult. Adding to the problem was the fact that the programs were voluntary, which made enrollment levels hard to predict. The ultimate unknown seemed to be what level of benefits was needed to generate the requisite landowner interest. Again, prudence required that the benefits be no higher than they needed to be. If every eligible landowner in the watershed applied for enrollment, that would be a sign that benefits were too high—neither the available budget nor the amount of shade needed for temperature compliance would support enrollment on such a scale.

The TAC dealt with the benefits issue by looking at several factors, including local land values, land rental rates, crop income, and farm community expectations and attitudes. Data in the ODA-OACD report indicated that Washington County landowners would begin participating in CREP when the annual payments were increased to $400. Higher than the net income produced by many crops that could be grown in riparian areas, this figure reflected the views of many in the local farm community, who tended to view the government with suspicion, weren't comfortable with strangers having access to their land, and didn't like program paperwork. The upshot seemed to be that landowners attached a "hassle" factor to program enrollment, and above-market incentives were needed to overcome it. The TAC also took into account the fact that farms in the watershed tended to be small (most were under 100 acres), and this probably meant that landowners would need to be paid a premium in order to find it worthwhile to enroll what in some cases might be only a few acres.

After careful consideration of the issues noted above, the TAC decided to increase the maximum annual Enhanced CREP payment to $393 per acre. Although this was below the $400 threshold given in the ODA-OACD report, the TAC concluded that other benefits increases would more than offset the difference. These included TSWCD's assumption of the cost of riparian buffer restoration and maintenance, the conservation easement option, and the water rights incentive. The TAC also noted that benefits levels could be re-visited periodically to determine whether they should be adjusted. In fairness to landowners already enrolled in the programs, however, changes in benefits would only affect new enrollees.

The TAC developed preliminary five-year cost estimates for the two programs, which included labor costs for current and new staff, and a number of assumptions concerning land characteristics, eligibility for bonus payments, and the enrollment options selected by program enrollees. The estimated cost of Enhanced CREP was $11,000 per acre, and the estimated cost of VEGBACC was $8,000 per acre. These were acknowledged to be very rough estimates due to the speculative nature of the assumptions.

Selecting the Right Program Manager

The implementation of VEGBACC and Enhanced CREP was designed to be a group effort involving the USDA Farm Service Agency (FSA) and Natural Resources Conservation Service (NRCS), the Oregon Department of Forestry (ODF), TSWCD, the Oregon Water Trust (OWT) and CWS. With this many organizations involved, the TAC was concerned about designing program processes so that they would be as simple as possible for landowners. The TAC was also concerned about maintaining good relationships with the farm community. With these issues in mind, the TAC needed to decide which of the agencies should serve as the on-the-ground program manager and primary contact with the farm community.

Although Clean Water Services would fund a significant portion of the program costs and had played an instrumental role in program development, in the past its focus had been urban, and it had little history with the farm community. FSA, NRCS and ODF had working relationships with the farm community, but since they were large organizations, they would need to make the new programs fit their existing processes and protocols, which could be viewed by some as too bureaucratic. TSWCD, on the other hand, was a small local agency governed by an elected board that currently consisted exclusively of farmers. More than any other agency, TSWCD could be relied upon to establish good working relationships with the farm community. Accordingly, the TAC selected TSWCD for the role of primary contact and on-the-ground program manager.

To minimize paperwork and the number of administrative changes that would need to be made to implement the two new programs, it was decided that no changes should be made to the existing CREP or any of its forms or processes. Instead, Enhanced CREP would exist as a separate program, with its own forms, signup process and accounting system. Enhanced CREP enrollees would need to enroll in both programs, but the Enhanced CREP paperwork would be relatively easy to work with, and this approach would avoid the complexities and delays associated with having changes approved by the USDA's national office.

Pay for Performance

Clean Water Services agreed to fund most of the cost of TSWCD's work. TSWCD is a small agency with no tax base, and would need to hire additional staff to manage the programs. Under the terms of its watershed-based permit, CWS had five years to obtain the amount of shade credit that would be needed to offset its treatment facility heat loads. It was expected that this would require an ambitious effort to create shade. Given the fact that the organization under regulatory pressure was not the organization that would be managing the shade programs, it was appropriate to consider innovative contracting approaches that would stimulate superior performance. Accordingly, CWS and TSWCD decided to negotiate a contract that would contain a number of performance incentives. The incentives would have the potential to double the amount payable under the contract. Although TSWCD would determine how funds received under the contract would be used, it was expected that some of the funds would be distributed to its employees under an incentive-based pay plan.

Current Status of Programs

Enhanced CREP and VEGBACC were recently approved by each of the program partners and made available for enrollment. Interest in the Enhanced CREP, which offers more benefits, is currently running significantly higher than interest in VEGBACC.

Marketing efforts have been occurring since late last summer, and these include mailed brochures, meetings with landowners at the TSWCD office and in the field, and speaking engagements at events attended by landowners. Landowners have expressed great interest, and program staff has been busy fielding questions and inspecting sites to determine eligibility and calculate benefits estimates. At the time of this writing, arrangements were being made to plant the site owned by the first enrollee, and planting at the site of the second enrollee will soon follow. Although it is still too early to determine whether the programs are likely to produce enough shade to meet permit requirements, the high level of landowner interest provides grounds for optimism.

Conclusion

Landowner incentive programs have the potential to provide an alternative means of meeting water quality requirements at greatly reduced cost. The Enhanced CREP and VEGBACC programs do this by providing farmers with incentives to create stream shade in riparian areas. The increased shade will reduce solar heating, and thereby offset the increases in stream temperature attributed to CWS's wastewater treatment facilities.

When developing landowner incentive programs, it is important to consider issues that are important to landowners. These include: level of economic benefits, program flexibility, landowner buy-in, and relationships with the landowner community. Water quality management organizations that lack experience with the landowner community should consider partnering with an organization that does. To minimize program costs, a number of factors should be considered when setting benefit levels, including landowner opportunity costs and attitudes toward enrollment in government programs.

The watershed approach encourages systems thinking and a perspective that recognizes the interconnectedness of nature. Enhanced CREP and VEGBACC may succeed in helping CWS comply with stream temperature requirements, but beyond this lie other potential benefits that are just as important, including flood management, erosion control, water pollution control, air quality improvement, and the conservation of species habitat.

Literature Cited

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Quantifying the Effect of a Vegetated Littoral Zone on Wet Detention Pond Phosphorus Load Reduction

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Abstract

A vegetated littoral fringe is presumed to enhance wet detention pond pollutant uptake and removal. Studies supporting the efficacy of littoral zone vegetation in a wet detention pond are noticeably scarce, however. We simulated storm events in 10 *in-situ* compartments that had pickerelweed, cattail, or were non-vegetated. The compartments effectively simulated wet detention pond hydraulic and pollutant reduction processes. Littoral zone vegetation did not consistently enhance either the phosphorus removal rate or the inter-event phosphorus concentration. Cattail phosphorus removal rates diminished, and phosphorus concentrations increased, in response to herbicide application. Qualitative evidence suggests that littoral zone vegetation reduced the growth of nuisance algae. Our experimental approach and set-up present an opportunity for effectively assessing additional stormwater issues.

Introduction

Wet detention ponds are a common and effective Best Management Practice for reducing stormwater pollutant loads (USEPA 1999a and b, USEPA 2002, Strecker et al. 2004). A vegetated littoral fringe is, explicitly or implicitly, presumed to enhance wet detention pond pollutant uptake and removal (USEPA 1999a and b, Barr Engineering 2001, City of Houston *et al.* 2001, USEPA 2002). To this end, various agencies require or recommend a vegetated littoral fringe (e.g., Maryland Department of the Environment 2000, Barr Engineering 2001, City of Houston *et al.* 2001, USEPA 2002, St. Johns River Water Management District 2003). Guidelines typically include a 3 to 4.5 m wide littoral zone that occupies 20 to 50 percent of the permanent pool water surface area.

Aquatic vegetation is effective at the uptake and removal of pollutants, as evidenced by an extensive literature regarding the use of natural and constructed wetlands for treating stormwater runoff, domestic and industrial wastewater, and acid mine drainage (Hammer 1988, Moshiri 1993, Kent 1994, Kadlec and Knight 1995). However, studies supporting the efficacy of littoral zone vegetation in a wet detention pond are noticeably scarce. Stoker (1997) demonstrated that a single wet detention pond designed with a planted littoral zone exceeded the average pollutant removal efficiency of various structural stormwater control systems. Rushton (1997) suggested that intentionally excluding littoral zone vegetation from wet detention ponds would decrease pollutant removal, stimulate algal blooms, and lead to lowered dissolved oxygen levels. Harper (2002) used a mass balance water quality model to conclude that littoral zone vegetation would provide little direct uptake of pollutants from the water column, although he acknowledged that indirect water quality benefits might accrue.

Varying climate and pollutant load hinder *in-situ* evaluation of the effects of littoral zone vegetation on wet detention pond performance. In Florida storm events occur with an unpredictable frequency, and are especially infrequent from November through May. Stormwater pollutant concentrations vary with the severity of the event, the inter-event period, and activities in the watershed. For example, Rushton (1997) found that nitrate, phosphate, suspended solids, zinc, lead, and copper concentrations in untreated stormwater varied by up to 3000 percent over a four-month period.

One way to overcome the hindrance of variable climate and pollutant load is to simulate storm events in replicated, *in-situ* compartments. We used this approach to evaluate the effects of littoral zone vegetation on wet detention pond pollutant reduction. Ten enclosed compartments were constructed in a Brevard County wet detention pond. Some of the compartments had pickerelweed (*Pontederia cordata*) or cattail (*Typha domingensis*), and others were non-vegetated. Stormwater withdrawn from the pond was spiked with nutrients, metals, and an oxygen demanding substance and pumped into each compartment. Samples were collected over a seven month period from compartment effluents during simulated storm events, and from within the compartments between events.

Our exploratory study had several objectives. First, and most importantly, we wished to determine if littoral zone vegetation enhanced pollutant reduction in a wet detention pond. Second, we wished to determine if any vegetation-induced reduction varied with plant species. Third, we wished to evaluate the impacts of vegetation management, i.e., planting and herbicide application, on pollutant reduction performance. Phosphorus results are described herein, and other pollutants will be discussed in a future publication.

Study Site

The study was conducted in a wet detention pond in Melbourne, Florida (latitude 28° 10.689' N, longitude 080° 40.347' W). Constructed in 1997, the 1 ha site has a 0.36 ha pool (38 x 95 m), a maximum depth of 6m and a littoral bench that extends 1.8 m from the pond edge with a slope of 10:1. Portions of the bench are occupied by pickerelweed and cattail, while other areas are non-vegetated. The study pond receives stormwater from a 0.23 ha wet detention pond located to the south, which in turn receives stormwater from a four lane divided highway. The system is designed to accommodate runoff generated by the 25-year, 24 hour storm, with recovery of the storage volume within 14 days. Water flows out of the study pond to the northeast through a riser and inverted release pipe with aluminum skimmer. Higher flows pass through a trash rack installed on the riser.

Materials and Methods

Ten rectangular compartments were constructed within the pond using geosynthetic floating booms and barriers. Scuba divers placed sand bags along the bottom edge of the barriers to ensure the compartments were hydraulically separate from the pond proper. The compartments were located away from the inflow and outflow structures so as not to interfere with the pond's designed hydraulic function (Figure 1). Each compartment encompassed 3.7 m of shoreline and extended perpendicularly 9 m toward the center of the pond. Maximum compartment depth was 3.4 m, and the volume was 100 m^3.

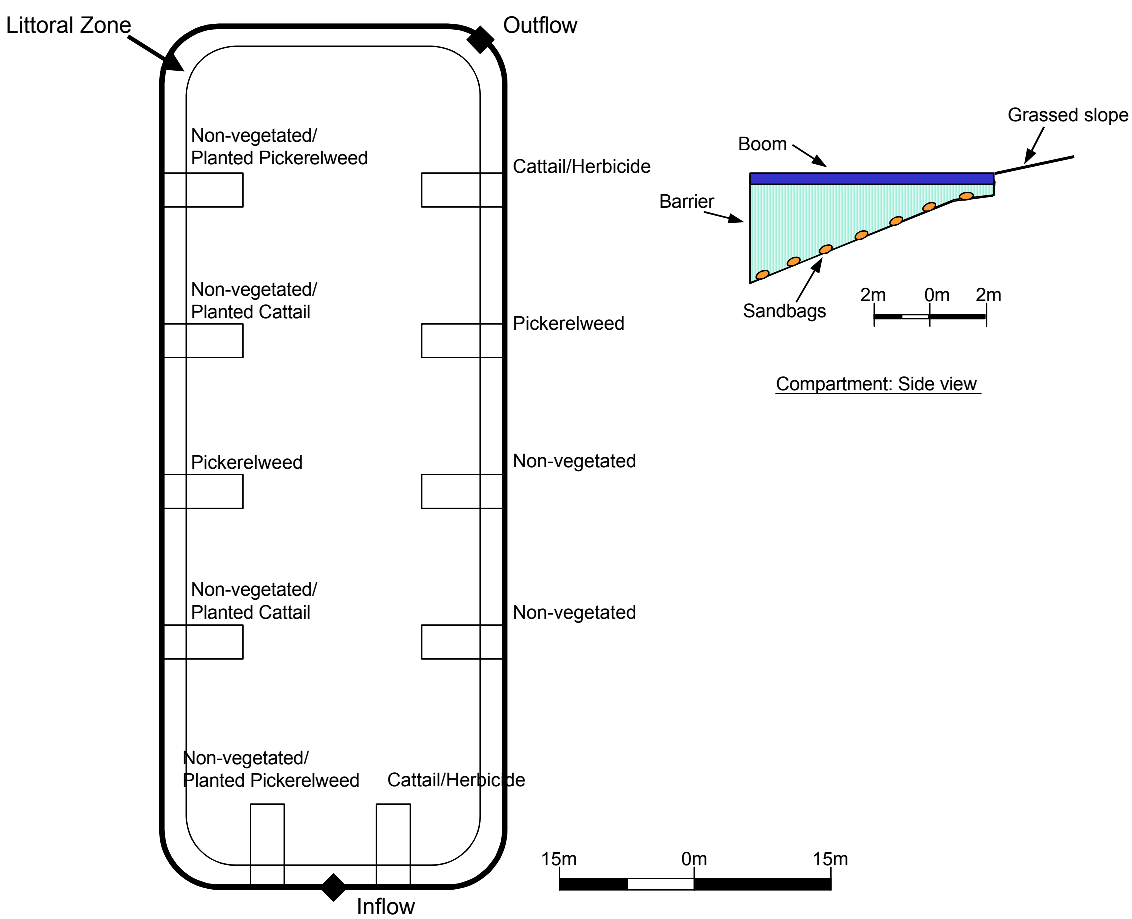

Figure 1. Wet detention pond littoral zone vegetation study configuration.

Outflows were inserted through the barrier 15 cm below the water surface at the deep end of the compartments. The outflows were constructed of 10 cm diameter PVC pipe with a threaded cap, and were open only during simulated storm events.

The compartments were initially constructed to encompass pickerelweed (n =2), cattail (n = 2), or non-vegetated littoral zone (n = 6, Figure 1). After two simulated storm events, two of the non-vegetated compartments were planted with pickerelweed and two were planted with cattail. The two existing cattail compartments were treated with herbicide after the fifth simulated storm event.

A pollutant spike solution mimicking stormwater concentrations (Bingham 1994, Harper 1994, Table 1) was fashioned in a 200 L polyethylene drum containing pond water. The solution then was transferred to two 200 L holding drums fitted with fountain pumps. Venturi force was used to draw the spike solution from the holding drums into a 10 cm trash pump for mixing with pond water and distribution to the compartments. Water flowed from the pond, and from the pump to the compartments, through three 15 cm diameter PVC lines. One line supplied the four compartments to the east, one line the four compartments to the west, and one line to the two compartments to the south. The main lines teed off to a 10 cm diameter PVC line that extended along the inside edge of a boom and terminated above the water surface in the center of compartment. The pumping flow rate was 190 liters per second for the entire system, 3 liters per second to each compartment. Each supply line was calibrated at the start of each simulated pumping event to ensure equal flow. The duration of pumping events was nine hours, which ensured one volume exchange per compartment.

Table 1. Spike solution composition and target concentrations. Solution components were mixed in 200 L of pond water. Lead chloride was dissolved by mixing with deionized water and acidification to pH 2 with muriatic acid to effect dissolution.

	Mass (g)	Target Concentration
Fructose (COD)	2520	20 mg/L
Ammonium nitrate (TN)	720	2 mg/L
Potassium phosphate (P)	219	400 ppb
Copper sulfate (Cu)	48	100 ppb
Lead chloride (Pb)	23	100 ppb

The compartment's littoral zones and open water areas were sampled 5 May 2004. Eight sampling sequences, comprised of simulated storm events and inter-events, were initiated 19 May 2004 and terminated 1 December 2004. Simulated storm event grab samples were collected at 3, 6, and 9 hours at the compartment outflow 15 cm below the water surface. One, seven, and 14-day[7] inter-event grab samples were collected from deep water and littoral zone locations within the compartments. The deepwater samples were a composite of a sample collected 15 cm beneath the surface and 30 cm above the

[7] After the second simulated storm event, the third inter-event samples were collected at 37 days to accommodate pickerelweed and cattail planting and grow-in.

bottom. A single littoral sample was collected midway between the surface and bottom, a depth of about 15 cm. Temperature, dissolved oxygen, and *p*H measurements were made *in situ* coincident with pollutant sampling using a Hach Sension 156 Multi-Parameter Meter. Grab samples were analyzed for nutrients, metals, and COD using EPA-approved methods.

We calculated the first order TP and SRP removal rates (k, day^{-1}):

$$\frac{(C - C^*)}{(C_i - C^*)} = e^{-kt}$$

C is the concentration at time t, and C* and C_i represent event background (lower limit) and initial concentrations, respectively. C* was set at the lowest observed concentrations (29 ppb for TP and 1 ppb for SRP). Initial concentrations were different for each treatment. The equation normalizes each event concentration change to the fraction of total pollutant removed during the event.

Microsoft ® Office Excel 2003 Solver routine was employed to determine k based on minimizing the sum of squared error (SSE).

$$SSE = \sum \left[\frac{(C - C^*)}{(C_i - C^*)} - e^{-kt} \right]$$

TP and SRP removal rates were compared among treatments and within compartments using Wilcoxon Matched Pairs Tests at a significance level of p = 0.05.

In addition, Day 14 total phosphorus (TP) and soluble reactive phosphorus (SRP) concentrations were analyzed as representative of pond inter-event water quality, and expected pond effluent water quality during a subsequent storm event.
Differences among and within treatments were examined graphically, and with Kruskal-Wallis ANOVA and Mann-Whitney Two Sample Tests at a significance level of p = 0.05.

Results

The compartments effectively simulated wet detention pond hydraulic and pollutant reduction processes. Simulated storm events filled the compartments and prompted outflow within ca. 30 minutes. Rhodamine WT injection revealed complete mixing of inflow water and compartment water in ca. 20 minutes. Every compartment, regardless of treatment, exhibited pollutant removal consistent with literature values (Table 2).

Table 2. Littoral zone vegetation study inflow concentrations (mean range), and removal efficiencies compared to literature efficiencies (percent range).

		Removal Efficiencies (percent)	
Parameter	Inflow	Study	Literature[1]
TSS	5 to 8 mg/L	3 to 42	40 to 90
COD	75 to 82 mg/L	12 to 32	20 to 40
TP	375 to 498 ppb	62 to 82	10 to 90
SRP	268 to 371	79 to 96	-
TN	3 to 4 mg/L	48 to 78	10 to 33
Cu	94 to 102 ppb	86 to 94	26 to 90
Pb	107 to 119 ppb	89 to 98	29 to 90

[1] EPA 1999, 2002; Heaney *et al.* 1999, Center for Watershed Protection (no date)

Vegetated treatment phosphorus removal rates, expressed as k values, did not typically exceed the non-vegetated treatment removal rate. Although, some differences in TP and SRP removal rates occurred among treatments. The TP removal rates for the herbicided cattail treatment were less than those of the non-vegetated and existing pickerelweed treatments in both the littoral zone and open water. Also, in the littoral zone, the rate for the planted treatments was less than that of the non-vegetated treatment (Figure 2). Similarly, the SRP herbicided cattail treatment removal rate was less than the planted pickerelweed treatment in the littoral zone, and less than the non-vegetated, existing pickerelweed, and planted pickerelweed treatments in open water. The non-vegetated SRP removal rate was greater than the existing pickerelweed and planted cattail rates in the littoral zone, and greater than the planted cattail rate in the open water.

Within treatments, littoral zone TP and SRP removal rates were almost always greater than open water removal rates (Figure 2). By exception, the existing cattail treatment removal rates exhibited no difference.

Inter-event TP and SRP concentrations did not typically differ among treatments. Open water TP and SRP concentrations, and littoral zone TP concentrations never differed among treatments. By exception, littoral zone existing cattail treatment TP concentration was less than other treatment concentrations in June (Figure 3). So too, herbicided cattail TP concentration was greater than other treatment concentrations in early October, and greater than the non-vegetated treatment concentration in late October.

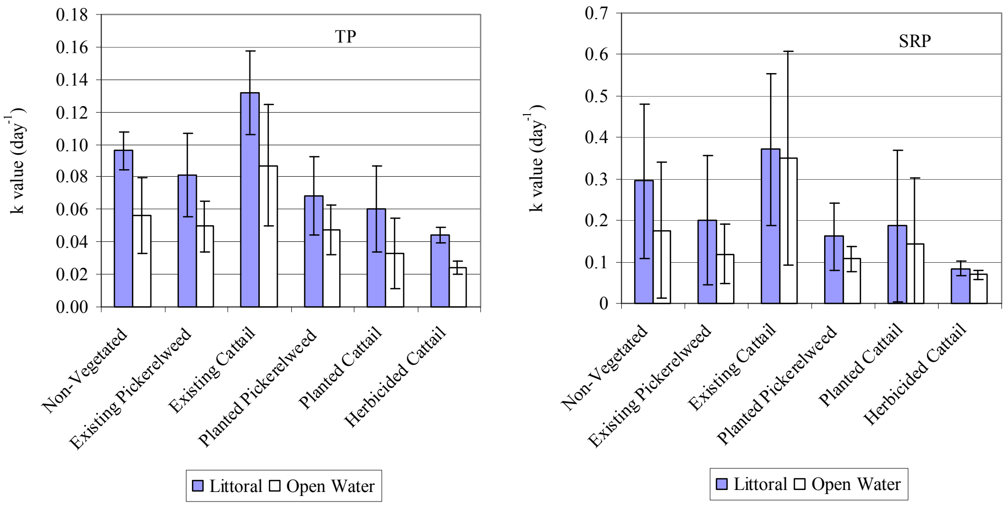

Figure 2. Phosphorus removal rates, expressed as k values, in vegetated and non-vegetated *in-situ* wet detention pond compartments. Higher k values represent better contaminant removal performance.

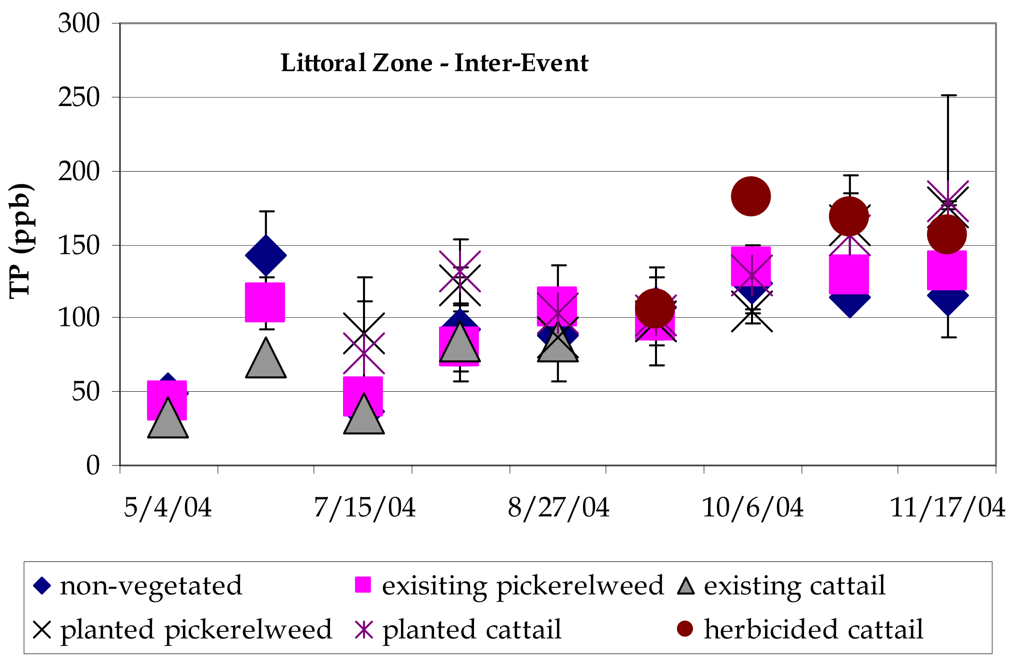

Figure 3. Total phosphorus (TP) concentrations in vegetated and non-vegetated *in-situ* wet detention pond compartments.

Within treatments, there was a tendency for TP concentrations to be lower in the littoral zone than in the open water for all but the herbicided cattail treatment. The tendency diminished and then disappeared in October and November. No tendency was evident for SRP.

Discussion

Our findings do not support the presumption that a vegetated littoral zone enhances wet detention pond phosphorus load reduction (USEPA 1999a and b, Barr Engineering 2001, City of Houston *et al.* 2001, USEPA 2002). TP and SRP removal rates were typically greater in the littoral zone than in open water. However, vegetated littoral zones did not have greater TP and SRP removal rates than non-vegetated littoral zones. In, fact, the converse was sometimes the case. TP concentrations were typically lower in the littoral zone than the open water during the growing season, but did not differ between non-vegetated and vegetated treatments. Taken together, these findings suggest that: 1) the littoral zone effects pollutant removal processes, 2) emergent macrophyte vegetation may not be essential, and 3) littoral zone pollutant removal under these conditions has little effect on overall wet pond pollutant reduction.

Herbicide application appeared to have a local, short-term effect on phosphorus reduction in our study. Phosphorus removal rates were regularly lower in the herbicided cattail treatments than some other treatments, and TP concentration was significantly elevated in the littoral zone soon after herbicide application. Phosphorus concentrations returned to levels consistent with other treatments shortly thereafter. Moreover, herbicided littoral zone and open water phosphorus concentrations tended to be the same, in contrast to other treatments.

Qualitative observations from our study suggest that a vegetated littoral zone reduces the growth of nuisance algae. Visual observation and photograph review revealed that dense, floating mats of algae developed in non-vegetated treatments soon after simulating a storm event. *Cladophora* and *Hydrodictyon* were the dominant algal species in summer and fall, respectively.

Our exploratory *in-situ* experiment represents an effective approach for readily addressing stormwater issues. The compartments functioned as wet detention ponds, and combined with a spiking process permit de-coupling from the vagaries of weather and pollutant load. Consequently, research questions can be addressed more quickly, and with greater certainty, than studies relying on unaltered wet detention ponds. We anticipate that our approach and set-up lend themselves to studies of vegetation/open water ratios, contaminant type and load, vegetation type, algal growth, pond component configuration, and others.

Acknowledgements

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Evaluation of a Floating Wetland for Improving Water Quality in an Urban Lake

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Abstract

Stands of littoral vegetation are thought to improve water quality in ponds and lakes. The effectiveness of fringing vegetation plantings, however, can be constrained by many factors, including steep shorelines that provide limited littoral area, and poor hydraulic exchange between the vegetated littoral zone and the bulk water column. In this study, we evaluated the water treatment effectiveness of a floating wetland, deployed near the center of a 1.6 hectare hypereutrophic urban lake. The wetland vegetation was contained within a floating boom 18 meters in diameter, which was equipped with a flexible fabric skirt that extended from the water's surface to the sediments. This effectively isolated a parcel of water, 262 m^2 and 2.75m deep, from the lake's water column. A solar-powered pump was deployed to provide a semi-continuous water exchange from the lake's water column into the compartment at a rate of approximately 100 m^3/day. At this exchange rate, a volume of water equal to the lake's entire water column passed through the wetland compartment in 10.5 months.

The floating wetland was deployed in August 2003, and performance was monitored from November 2003 through October 2004. The wetland effectively removed particulate matter, reducing total suspended solids and turbidity by 67% and 50%, respectively. Chlorophyll *a* levels were reduced by 65% during passage through the system, suggesting the bulk of the removed particles were phytoplankton.

Because of internal cycling of phosphorus (P) within the wetland compartment, we assumed little net P removal would be achieved by the wetland under steady-state conditions. We therefore injected alum once monthly beneath the floating mat to stabilize P in the accumulating wetland sediments. Based on weekly measurements, P removal in the system from November 2003 through October 2004 averaged 50%, with mean inflow concentrations reduced from 0.168 to 0.084 mg TP/L. Total nitrogen (N) removal in the system averaged 40%. On a mass basis, the system removed 25.6 kg N and 2.81 kg P/yr. The floating wetland was heavily utilized by birds, which probably contributed to an observed net export in coliform bacteria, and also may have reduced system nutrient removal effectiveness.

Data from this, and prior studies, suggest that the floating wetland can be an effective nutrient control technology, particularly for small urban lakes, wet detention ponds and agricultural impoundments with water column TP concentrations in excess of approximately 0.100 mg/L.

Introduction

Emergent macrophytes commonly are planted in the littoral regions of ponds, lakes and wet detention ponds to improve water quality (USEPA 1999a; USEPA 1999b). Not all water bodies, however, are amenable for littoral plantings. In some cases, the system bathymetry is such that there is limited littoral shelf available for emergent macrophyte beds. In other systems, particularly wet detention ponds, water stages can vary markedly, thereby either flooding the littoral vegetation during high stages, or stranding the macrophytes on dry soil during low stages. Even in water bodies with fairly consistent stage conditions, hydraulic exchange between the vegetated littoral region and the bulk water column may be limited. Finally, in some water bodies, shoreline property owners object to littoral plantings for aesthetic reasons, or over concerns that the vegetation could harbor dangerous wildlife.

Because of these potential limitations to the use of littoral macrophytes, we tested an alternative approach to improving water quality, in which we deployed macrophyte vegetation in the center of a small urban lake. This floating wetland was equipped with a solar powered pump to effect water exchange between the wetland and the bulk water column of the lake. Because the wetland was equipped with discrete inflow and outflow ports, it was possible to measure the pollutant removal effectiveness of the wetland based on concentration and mass load reductions. The goals of this study were: to characterize pollutant load reductions by the floating wetland; to compare these load reductions to the estimated external pollutant loads from the lake's watershed; and, to determine under what conditions the floating wetland can be a useful tool for enhancing lake/pond water quality.

Study Site

Lake June is a 1.6 hectare (ha) lake located in the Holden Heights neighborhood, Orange County, Florida. The lake has a 37 ha commercial and residential drainage basin. Bathymetric surveys performed by the Orange County Environmental Protection Division depicted a mean water depth of 2 meters, and maximum muck depth of up to 0.5m. This survey also revealed that the lake has a steep shoreline, with little littoral shelf. Estimated water volume of Lake June is 32,000 m^3. The lake discharges over a weir into a drainage well, which effectively controls maximum stage. Previous water analyses characterized the lake as hypereutrophic, with total nitrogen (N) concentrations of 2.0 mg/L, total phosphorus (P) concentrations of 0.24 mg/L, and chlorophyll *a* values of 92 mg/m^3. At the time of deployment of the floating wetland, the dominant macrophyte vegetation type in the lake was water hyacinth (*Eichhornia crassipes*), which covered approximately 10% of the lake's surface.

Materials and Methods

The floating wetland was contained within a circular floating boom 18 meters in diameter, which was equipped with a weighted, flexible fabric skirt that extended from the water's surface to the sediments, effectively isolating a parcel of water beneath the vegetation from the lake's water column (Fig. 1). This isolated water parcel was 262m^2 in surface area and 2.75 m deep. To initiate development of a floating vegetative mat, we first encircled a portion of the water hyacinths in the lake into the floating boom. Other plants were added within the floating boom to create a more diverse wetland, including plants in the genera *Hydrocotyle*, *Bidens*, *Sagittaria*, and *Pontederia*. The remaining water hyacinths in the lake were killed with a herbicide. No vegetation harvesting from the wetland was performed during the study.

A solar-powered pump was deployed within the floating wetland to provide a semi-continuous water exchange from the lake's water column into the compartment at a rate of approximately 100 m^3/day (Fig. 1). This provided a hydraulic retention time (HRT) within the compartment of 7 days. At this exchange rate, a volume of water equal to the lake's entire water column would pass through the wetland compartment in 10.5 months. Fishing line was deployed to discourage birds from landing on the solar collectors and the edge of the floating boom. No attempt was made to discourage birds from feeding or roosting within the vegetation itself.

Figure 1. The floating wetland deployed in Lake June. The photo on the right depicts a closer view of the solar panels and vegetation.

At the initiation of the study, we assumed that P removal, based solely on the accrual of P in sediments produced by the wetland vegetation, would not be significant relative to the P removal needs of the lake. We therefore enhanced P removal of the floating wetland by chemically stabilizing P in the organic detritus that was deposited in the underlying sediments. This was accomplished by means of a monthly injection of alum beneath the wetland, at a dose of 12.5 mg Al/L as aluminum sulfate (alum). This alum concentration was selected based on results of jar tests, which demonstrated formation of a moderate to rapidly settling floc at this dose. Chemical analyses also revealed that the lake is poorly buffered, so we also injected $NaHCO_3$ immediately before injecting alum. Other than initial jar tests, no further attempts were made to optimize aluminum dose or form of compound during this study.

Wetland inflow and outflow monitoring was performed from November 2003 through October 2004. The wetland inflow sample was collected from the lake (0.2m depth) just outside of the floating wetland barrier. The wetland outflow sample was collected from one of three locations. Prior to March 2004, outflow total P (TP) samples were collected from just inside the wetland enclosure, adjacent to the submerged outflow port. Outflow samples for other analyses (see below) were collected using a long length of polyethylene tube running from the shoreline to the submerged outflow port of the wetland. After several months of collecting samples through this tube, we determined that they were being contaminated with fine particulate matter dislodged by the suction of the sampling pump. In March 2004 we corrected this problem by adjusting the wetland outflow pipe so it discharged above the waters surface. After this time, all outflow samples, for TP and other parameters, were collected as a grab sample from this location.

In addition to TP, soluble reactive P (SRP) and pH were measured weekly, and the following parameters were measured every 4 – 6 weeks: nitrate-nitrogen (N), ammonia-N, total kjeldahl N, chlorophyll *a*, total suspended solids (TSS), turbidity, sulfate, total aluminum (TAl), dissolved oxygen (DO), fecal coliforms (FC) and total coliforms (TC). All laboratory analyses were performed using USEPA-approved procedures, including appropriate quality assurance/quality control protocols.

In order to characterize the floating wetland sediments, during January 2005 we retrieved four 7.6 cm diameter sediment cores from within the wetland enclosure, and an additional four cores from an open water area in the lake. These cores were visually inspected for presence and depth of organic matter and alum floc accretion.

Results

The floating wetland was initially deployed in August 2003, at which time it exhibited approximately 40% vegetation cover. The macrophyte standing crop increased throughout the fall of 2003, and attained 100% cover in March 2004.

Lake nutrient concentrations varied widely during the study, from 0.084 to 0.379 mg /L for TP and 0.76 to 1.25 mg/L for TN (Table 1). We observed no obvious increasing or decreasing trend in lake water TP concentrations during the year-long evaluation: maximum and minimum lake water TP levels were observed in April and August 2004, respectively (Fig. 2). The floating wetland exhibited effective nutrient removal, removing 50% of the inflow TP and 40% of the inflow TN. Despite widely varying lake TP concentrations, the outflow from the floating wetland was relatively consistent, averaging 0.084 mg/L and ranging from 0.054 to 0.130 mg/L (Fig. 2). Neither the wetland inflow (= lake water) nor wetland outflow contained substantial amounts of soluble reactive P (Table 1). Wetland outflow TN concentrations averaged 1.08 mg/L, and ranged from 0.76 to 1.25 mg/L (Table 1).

Table 1. Summary of the water quality treatment performance of the Lake June floating wetland. Total P and soluble reactive P were measured approximately every week for one year. Other constituents were measured every 4 – 6 weeks for six months.

	Wetland Inflow (Lake)		**Wetland Outflow**	
total phosphorus (mg/L)	0.168	(0.084 – 0.379)	0.084	(0.054 – 0.130)
soluble reactive phosphorus (mg/L)	0.006	(<0.002 – 0.027)	0.008	(<0.002 – 0.029)
total nitrogen (mg/L)	1.80	(1.36 – 2.17)	1.08	(0.76 – 1.25)
chlorophyll *a* (mg/m^3)	78	(34 – 123)	26	(15 – 35)
total suspended solids (mg/L)	17	(6 – 26)	6	(2 – 10)
Turbidity (NTU)	12	(8 – 18)	6	(4 – 11)
total aluminum (mg/L)	0.161	(0.057 – 0.260)	0.142	(0.060 – 0.260)
Sulfate (mg/L)	18.1	(10 – 21)	20.9	(12 – 44)
dissolved oxygen (mg/L)	9.6	(6.3 – 15)	1.2	(0.17 – 3.6)
total coliform (CFU)	339	(100 – 840)	3057	(400 – 6800)
fecal coliform (CFU)	193	(20 – 550)	1051	(280 – 1800)

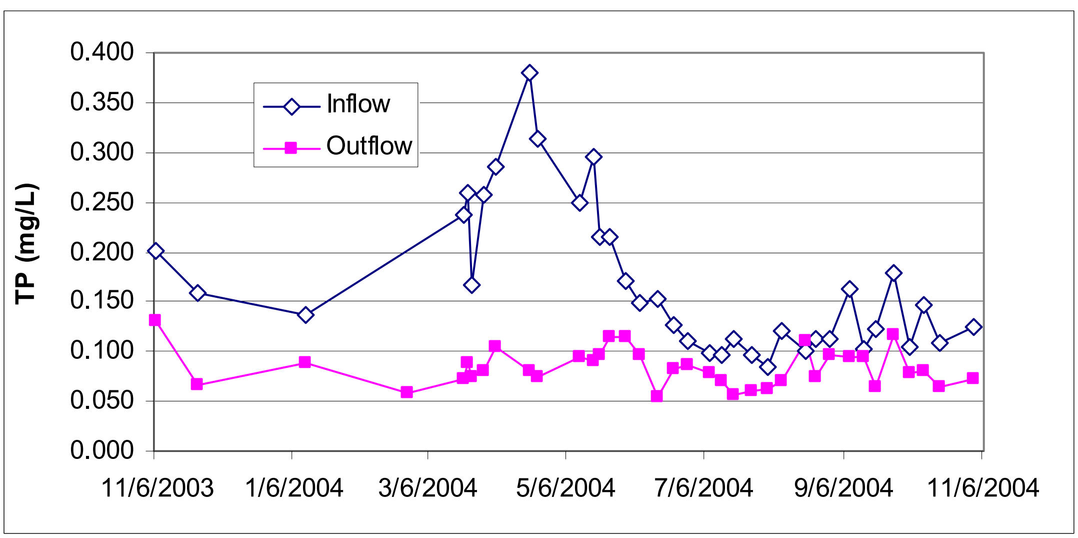

Figure 2. Inflow (= lake water) and outflow TP concentrations from the floating wetland in Lake June.

The floating wetland was effective at removing particulate matter, providing a 65, 50 and 67% reduction of total suspended solids, turbidity and chlorophyll *a*, respectively (Table 1; Fig. 3). Visual inspection of the water samples, coupled with chlorophyll *a* analyses, suggest that phytoplankton comprised the bulk of the particulate matter in the relatively turbid wetland inflow samples (Table 1). By contrast, the outflow from the floating wetland was quite clear. Despite the observed reduction in particles, fecal and total coliform levels in the wetland outflow were approximately an order of magnitude higher than inflow values (Table 1).

Although the monthly injection of alum into the water beneath the floating wetland vegetation undoubtedly enhanced water column pollutant removal, we observed no clear temporal relationship between wetland outflow TP levels and the timing of alum applications. For example, Figure 4 depicts wetland outflow TP concentrations just prior to, and for 22 days following the March 2004 alum addition. Despite the periodic use of alum, mean total aluminum levels in the wetland outflow were slightly lower than those of the influent lake water (Table 1). Outflow sulfate levels, by contrast, were slightly higher in the wetland outflow than in the inflow waters (Table 1).

Daytime wetland inflow (= lake water) DO concentrations typically were high, averaging 9.6 mg/L. Wetland outflow DO levels were markedly lower, with mean values of 1.2 mg/L. Mean wetland inflow and outflow pH levels were 7.1 and 6.3, respectively.

Three hurricanes passed near Lake June during August and September 2004. None of the wetland infrastructure, including the solar-powered pumping system, sustained any damage from the storms. Some of the wetland foliage, however, was shredded by the strong winds, particularly from the August storm (Hurricane Charley).

At each weekly site visit, we noted that the floating wetland was frequented by several bird species (cormorants, herons, egrets, anhingas, gallinules), some of which used the wetland as a perch, and others which fed within the wetland vegetation itself.

The sediment cores collected from the lake and within the wetland enclosure suggested a large, historical accumulation of organic material. The depth of unconsolidated and consolidated floc varied widely both within and outside of the enclosure, ranging from 0.2 to 0.4 m. No evidence of an alum floc was found in any of the cores collected from within the enclosure.

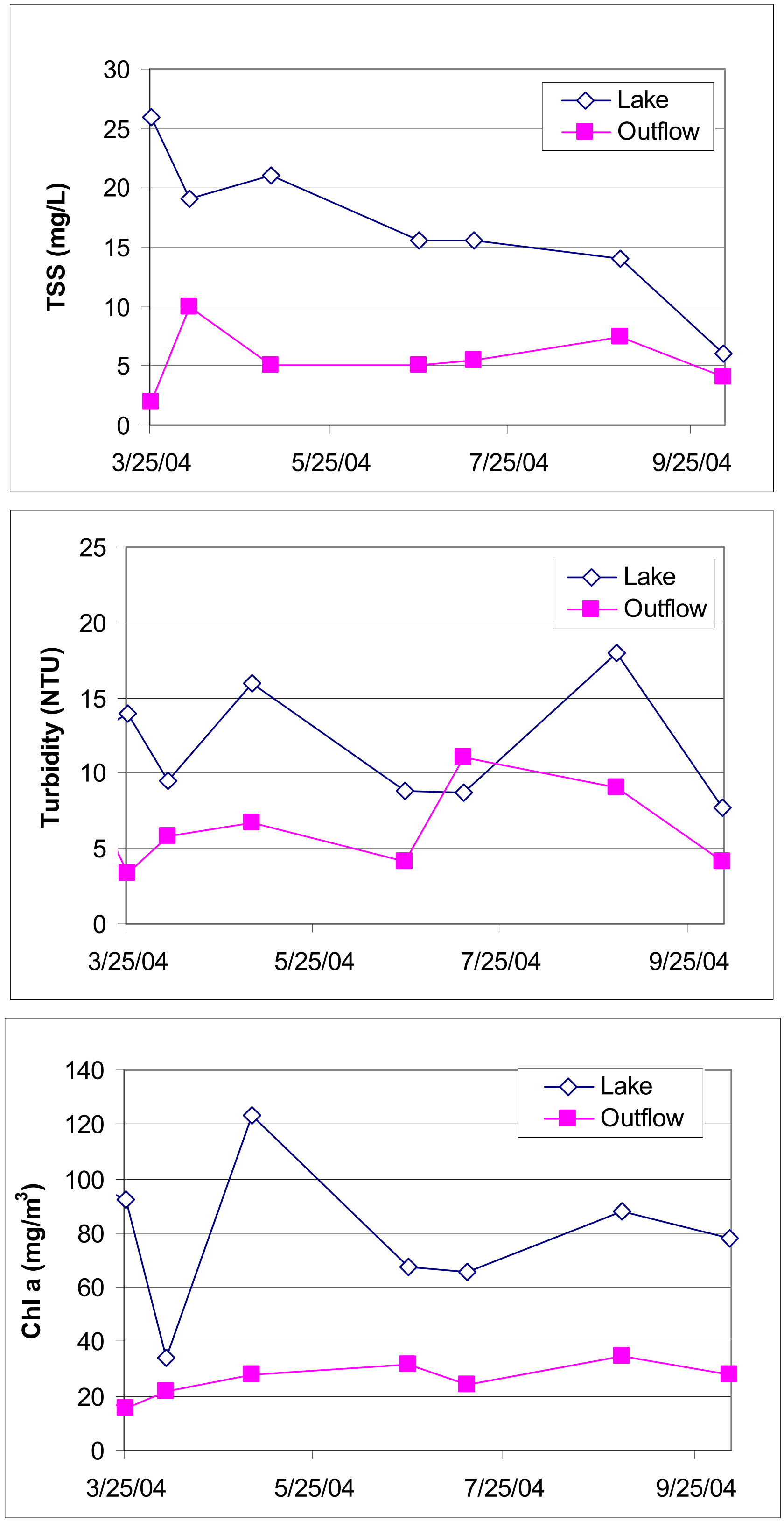

Figure 3. Removal of particulate matter, represented by total suspended solids, turbidity and chlorophyll *a*, in the floating wetland.

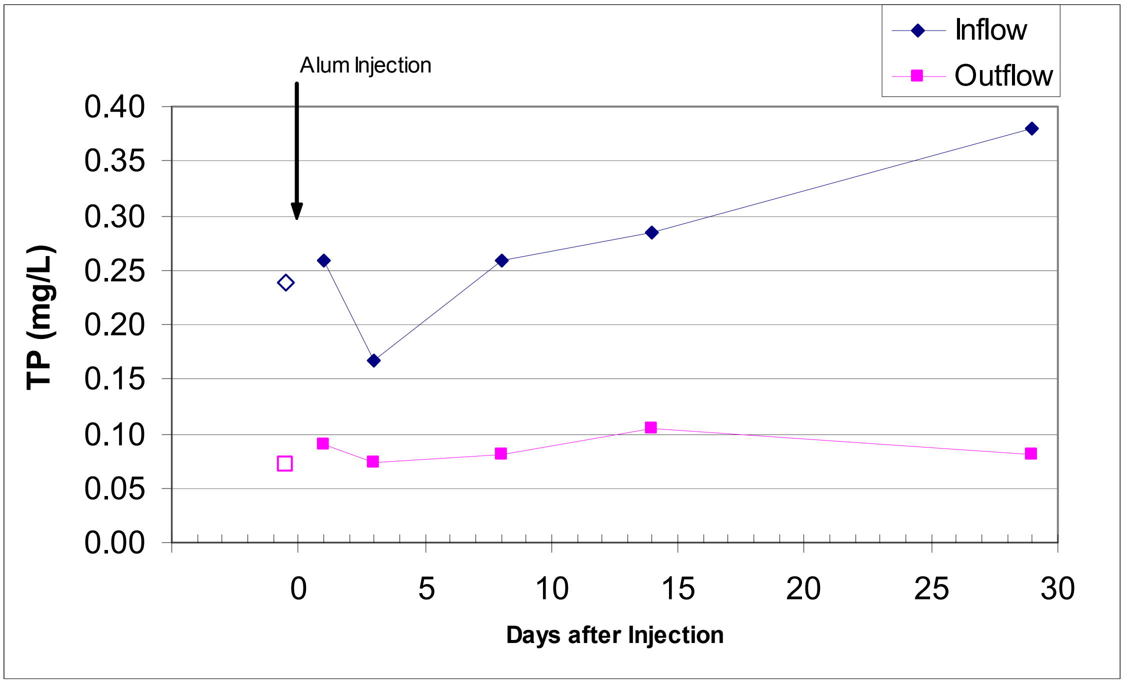

Figure 4. Effect of the April 2004 alum injection on TP removal performance of the floating wetland. Open symbols represent the floating wetland inflow and outflow TP concentrations just prior to the alum injection.

Discussion

The Lake June floating wetland provided effective removal of N, P and particulate matter. Other effects on water quality, such as the low outflow DO levels and slightly acidic pH conditions, are typical of outflows from densely vegetated wetlands (DeBusk and DeBusk 2001). Despite the periodic injection of alum, marked increases in aluminum and sulfate were not observed in the wetland outflow. Similarly, we found no visual evidence of an alum floc in the underlying sediments. This was probably due to two factors. First, the alum floc likely was diluted by a continuous input of organic matter. This material consisted primarily of phytoplankton from the wetland inflow (= lake water) that settled in the dark water column below the mat, as well as detritus produced by the overlying macrophytes. Second, because the lake's original organic floc layer was quite fluid, activity by burrowing organisms could readily have mixed the alum floc with previously deposited sediments.

The order of magnitude increase in coliforms that we observed during the water's passage through the wetland was probably caused by the widespread use of the wetland by birds. Additionally, the birds almost certainly contributed substantial loads of N and P to the wetland. It is probable that the removal effectiveness for most constituents would be improved by utilizing fishing line or netting to discourage all bird activity within the wetland.

Regardless of the inflow (= lake water) TP concentration, which attained levels as high as 0.38 mg/L, the floating wetland generally produced outflow TP levels in the range of 0.05 to 0.10 mg TP/L. It is unknown whether lower TP levels could be achieved in the absence of bird activity. However, without further data, it appears that this system would provide only minimal benefits if deployed in a lake or pond with ambient water column TP levels substantially below 0.100 mg/L.

Our prior experience with this floating wetland concept suggested that a system sized at approximately 2% of the area of the overall water body could significantly reduce the mass of key pollutants that contribute to impaired water quality. In the present study, the floating wetland comprised 1.6% of Lake June's surface area. Based on an average estimated flow rate of 100m^3/day through the wetland, the Lake June floating wetland removed a total mass of 25.6 kg N and 2.81 kg P/yr from the lake water column.

If the above mass removal values are adjusted for the lake's surface area (16,000 m^2), then the floating wetland provided a 0.18 gP/m^2-yr reduction for P, and 1.6 gN/m^2-yr reduction for N. Under certain lake loading conditions, these removal rates can contribute significantly to improving water quality. For example, in a study of north-central Florida lakes, Shannon and Brezonik (1971) noted that eutrophic lakes exhibited an estimated average P supply of 0.30 gP/m^2-yr, and hypereutrophic lakes had an average P supply of about 0.45 gP/m^2-yr. These data suggest that under appropriate loading conditions, the mass load reduction afforded by the floating wetland is adequate to effect an improvement in trophic state.

Many lakes and ponds, by contrast, have loads far in excess of those noted above. Our estimates suggest that Lake June, with a 37.2 ha drainage basin, is one of these. We used the following assumptions to estimate hydraulic, N and P loadings to the lake from the drainage basin. We assumed 80% of the basin was single-family residential, and 20% was a commercial land use. Rainfall runoff from a single-family residential land use is thought to contain 0.43 mg TP/L, with the 28% impervious surface providing a runoff coefficient of 0.373. For commercial land uses, with 98% impervious surface and a runoff coefficient of 0.887, the TP concentration of runoff is estimated to be 0.43 mg TP/L (Harper 1994). Combining these values with the observed 177 cm rainfall depth, a calculated rough estimate of annual external loading to Lake June suggests that the system may have received as much as 310,000 m^3 of runoff, with a nutrient loading of 770 kg N and 105 kg P/yr. If these rates indeed are correct, then the lake volume (32,000m^3) was exchanged approximately 9.5 times during the study with external runoff, and the mass of N and P removed by the floating wetland would need to be markedly higher to improve lake water quality.

For dramatically overloaded systems such as Lake June, it is clear that either multiple floating wetlands, or one larger floating wetland, would need to be deployed in the lake to effect an improvement in lake water quality. In sizing the wetland, it is important to note that floating wetlands also appear effective at treating waters with much higher TP concentrations than those of Lake June. We tested a floating wetland, with a similar operational cycle (ca. 7 day HRT, alum dosing once/monthly), in a pond with ambient TP concentrations of 1.0 mg TP/L. Mass removal rates for this system were eight-fold higher than those of the Lake June system. This is a common phenomenon for both biological (e.g., wetland) and chemical-addition treatment systems, where the pollutant removal effectiveness on a mass basis is usually high at high inflow pollutant concentrations, and then declines as inflow pollutant levels decrease (Kadlec and Knight 1996).

In conclusion, these data suggest that floating wetlands will be most effective, on a mass removal per unit area basis, when deployed in eutrophic and hypereutrophic systems such as golf course ponds, urban lakes with large drainage basins, agricultural impoundments, polluted detention ponds that feed into cleaner water bodies, and even portions or lobes of lakes that receive high external nutrient loads. Discouragement of bird activity will probably enhance nutrient removal, and may even lead to effective treatment of microbiological constituents.

Acknowledgement

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Effects of Residence Time and Depth on Wet Detention System Performance

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Abstract

Wet detention ponds are a commonly used stormwater management technique throughout the State of Florida. Current presumptive design criteria for wet detention ponds vary widely with respect to depth and residence time, ranging from shallow ponds (4-12 feet deep) with short residence times (14 days during wet season) to deep ponds (12-20 feet deep) with long residence times (> 100 days). Existing literature related to wet detention ponds suggest a strong correlation between residence time and removal efficiency for both total phosphorus and total nitrogen in wet ponds, with performance efficiency increasing as residence time increases.

A water quality monitoring program was conducted from 2001-2004 in seven wet detention ponds in southeast Orange County which were constructed to a maximum depth of 20 feet. No significant decreases in dissolved oxygen were observed at the pond outfall, even following rain events in excess of 4 inches. Similarly, no statistically significant differences were observed in mean values of dissolved oxygen, conductivity, ammonia, total nitrogen, SRP, or total phosphorus in samples collected following each rain event.

Engineers should be encouraged to design deep (20 feet) wet detention ponds with long residence times (> 100 days). Construction of deep ponds would not only increase the performance efficiency but also provide a substantially larger storage volume for accumulated sediments in an area where resuspension of the material appears to be unlikely.

Introduction

Both man-made and natural waterbodies have been used for stormwater treatment within the State of Florida for over 100 years. Today, wet detention systems are one of the most popular stormwater management techniques, particularly in areas with high groundwater tables. Pollutant removal processes in wet detention systems occur through a variety of mechanisms, including physical processes such as sedimentation, chemical processes such as precipitation and adsorption, and biological uptake from algae, bacteria, and rooted vegetation. These removal processes are regulated by predictable laws of physics, chemistry, and biology, regardless of whether the waterbody has a natural or man-made origin.

A schematic diagram of a wet detention system is given in Figure 1. A wet detention pond is simply a modified detention facility which is designed to include a permanent pool of water with a depth that varies from approximately 6-30 feet. The water level in a wet detention system is controlled by an orifice located in the outfall structure from the pond. The facility is designed with a required treatment volume based upon a specified depth of runoff over the contributing drainage basin area. The treatment volume represents a relatively small portion of the overall volume of the pond and regulates primarily how rapidly water discharges from the pond following a storm event. Inputs of stormwater runoff equal to or less than the treatment volume exit the pond slowly through an orifice in the outfall structure or through percolation into the surrounding groundwater table. Stormwater inputs into the facility in excess of the treatment volume can exit from the pond directly over a weir included in the pond outfall structure. A littoral zone is typically planted around the perimeter of a wet detention facility to provide additional biological uptake and enhanced biological communities.

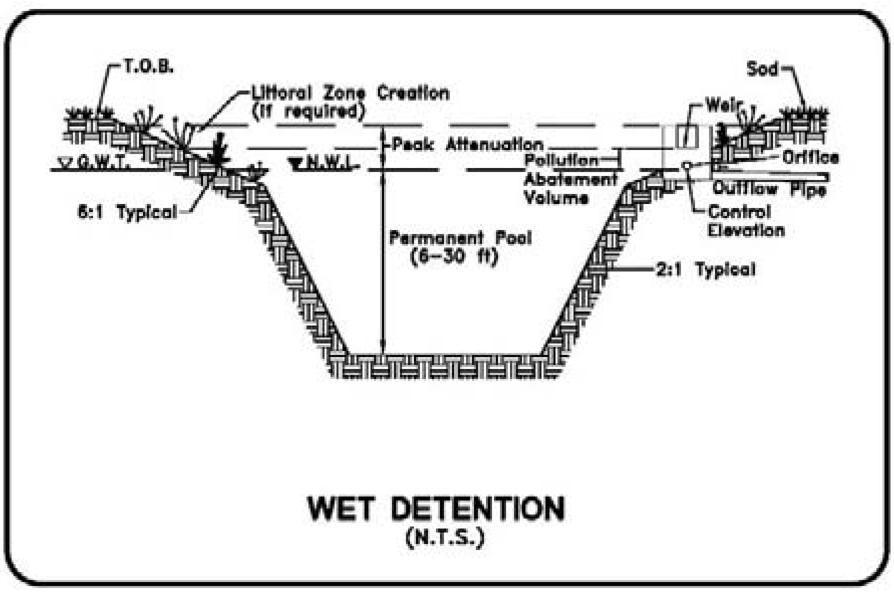

Figure 1. Schematic of a Wet Detention System.

Upon entering a wet detention facility, stormwater inputs mix rapidly with existing water contained in the permanent pool. Physical, chemical, and biological processes begin to rapidly remove pollutant inputs from the water column. Water which leaves through the orifice in the outfall structure is a combination of the mixture of partially treated stormwater and the water contained within the permanent pool. In general, the concentrations of constituents in the permanent pool are typically much less than input concentrations in stormwater runoff, resulting in discharges from the facility which are substantially lower in concentration than found in raw stormwater. As a result, good removal efficiencies are achieved within a wet detention facility for most stormwater constituents. Although the littoral zone can provide enhanced biological uptake, previous research has indicated that a vast majority of removal processes in wet detention facilities occur within the permanent pool volume rather than in the littoral zone vegetation (Harper, 1985; Harper 1988; Harper and Herr, 1993).

Beginning in the early-1980s, design criteria were established for stormwater treatment ponds to ensure a minimum level of pollutant attenuation. The two most significant design criteria for wet detention ponds are the treatment volume, which regulates the pond size and water level fluctuation, and pond depth, which is directly related to the permanent pool volume and residence time of the pond. A summary of current design criteria for wet detention ponds in three primary water management districts in Florida is given in Table 1. The required treatment volume is similar between the three water management districts. However, substantial differences exist with respect to criteria for pond depth and minimum residence time. The St. Johns River Water Management District (SJRWMD) specifies a minimum wet season residence time of 14 days, with a maximum pond depth of 12 feet and a mean depth ranging from 2-8 feet. The Southwest Florida Water Management District (SWFWMD) also requires a minimum 14-day residence time, but places no limitations on pond depth other than the bottom of the pond cannot breech an aquitard. The South Florida Water Management District (SFWMD) has no specific design criteria for either residence time or pond depth.

Table 1

Summary of Design Criteria for Wet Detention Ponds in Florida

PARAMETER	DESIGN CRITERIA		
	SJRWMD	SWFWMD	SFWMD
Treatment Volume	1 inch of runoff	1 inch of runoff	1 inch of runoff
Pond Depth	Maximum: < 12 ft Mean: 2-8 ft	--[1]	--[2]
Minimum Residence Time (Days)	14[3]	14[3]	--[2]

1. Cannot breech aquitard
2. Not specified
3. Minimum wet season residence time

Of the design parameters listed in Table 1, the most important criteria with respect to overall performance of the stormwater management system are pond depth and residence time. However, the significance of residence time on wet detention pond performance has been clearly reported by several researchers. Rushton, et al. (1997) documented a substantial improvement in wet detention pond performance by increasing the mean pond retention time from 2 days to 14 days. Significant increases in removal efficiencies were observed at the higher residence time for TSS, total organic nitrogen, ammonia, NO_x, SRP, total phosphorus, total iron, and total zinc. Toet, et al. (1990) report that settling may be the most significant removal process for constituents in wet detention ponds. Settling efficiency is dependent on the residence time which is related to the permanent pool volume provided. Toet, et al. concluded that increasing the permanent pool volume has a direct impact on removal efficiency of all components. Harper and Herr (1993) documented increases in removal efficiencies for total phosphorus and total nitrogen with increases in detention time from 7-43 days in a wet detention facility receiving a combination of commercial and residential runoff.

Pond depth is also a significant factor impacting the performance efficiency of a wet detention system since pond depth is directly related to permanent pool volume. Unfortunately, virtually no previous research has been performed to quantify the performance characteristics of relatively deep (> 20 feet) wet detention ponds. Current limitations on the allowable depth of wet detention ponds are based primarily on inferences from studies intended for other purposes.

Typical zonation in a pond or lake is illustrated on Figure 2. The upper portions of the water column in a waterbody are typically well mixed, with a relatively uniform temperature. This upper layer, called the epilimnion, is the area in which the majority of algal production occurs. In this zone, photosynthesis exceeds respiration, and near saturation levels of dissolved oxygen are typically maintained. Under certain conditions, lower layers of a deep lake may become isolated from the upper layers as a result of thermal stratification within the waterbody. Penetration of sunlight into these lower layers can be poor, and as a result, little or no algal productivity may occur. In this lower zone, commonly referred to as the hypolimnion, respiration exceeds photosynthesis, and the water column may become void of dissolved oxygen during certain parts of the year.

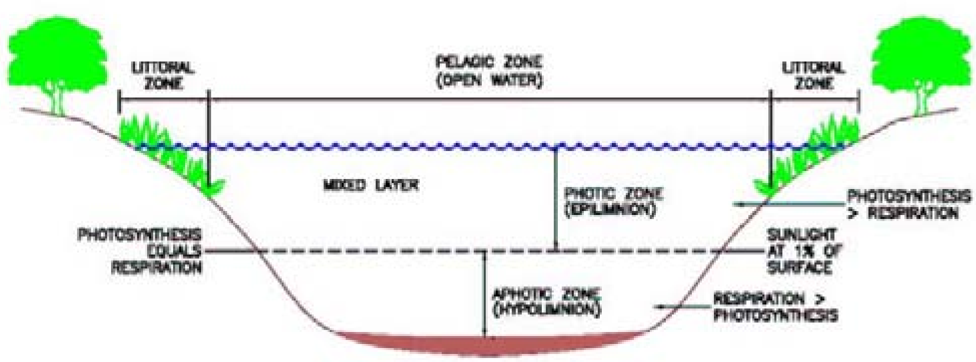

Figure 2. Typical Zonation in a Lake or Pond.

Under stratified conditions, the hypolimnion becomes isolated from oxygen input mechanisms, and anaerobic conditions may develop. Anaerobic conditions, considered to occur when dissolved oxygen concentrations decrease to less than 1 mg/l, may increase the release of ions such as ammonia and orthophosphorus, along with gases such as H_2S and

CO_2, from the bottom sediments into the hypolimnion water. The accumulated constituents in the hypolimnion can then be circulated into the epilimnion as a result of a destratifying event, such as a prolonged windy period or strong storm event, potentially resulting in episodes of reduced water quality and low dissolved oxygen at the pond outfall. However, if penetration of solar radiation is not inhibited, waterbodies as deep as 20 feet or more with low algal production may not experience stratification or anaerobic conditions at deeper water depths.

Impacts of Residence Time on Performance Efficiency

A general literature review was conducted of previous research performed within the State of Florida which quantifies pollutant removal efficiencies for stormwater treatment ponds as a function of residence time. Particular emphasis was given to studies which appear to be scientifically valid, provide a reasonable period of study, include estimates of performance efficiency in terms of mass removal, and provide an estimate of residence time or sufficient information so that a residence time could be calculated. Although studies related to stormwater treatment ponds are relatively common in the literature, very few of these studies provide estimates of performance efficiency calculated on a mass removal basis, and even fewer provide estimates of pond residence time during the period of study. The vast majority of wet detention pond studies simply provide measurements of changes in concentrations during migration through the pond.

A summary of selected stormwater treatment studies identified in the literature is given in Table 2. Thirteen separate studies were selected which provide both mass removal estimates for total nitrogen and total phosphorus and calculated estimates of residence time. The first two ponds identified in Table 2 present the results of stormwater research conducted by Rushton, et al. (1995) and Harper and Herr (1993). Residence times for these ponds range from 2-19 days. As residence times increase, ponds typically become larger in both surface area and volume and are often identified as named waterbodies. The remaining studies summarized in Table 2 reflect studies performed on named waterbodies, utilized primarily for stormwater treatment, as part of a watershed study or water quality improvement project. For each of these studies, the calculated residence time and mass removal efficiencies for total nitrogen and total phosphorus reflect the combined inputs from stormwater runoff, groundwater seepage, and bulk precipitation. Virtually all of these studies reflect urban waterbodies which provide stormwater treatment for large residential and commercial areas. Calculated residence times for the selected studies range from 2-328 days, reflecting a wide range of treatment conditions.

A plot of removal of total phosphorus as a function of residence time in stormwater treatment ponds is given in Figure 3. The "best-fit" equation through these points exhibits a logarithmic shape with an R-square value of 0.720, indicating that residence time explains approximately 72% of the variability in removal efficiency for total phosphorus in stormwater treatment ponds. The "best-fit" curve appears to become asymptotic at a removal efficiency of approximately 90% for total phosphorus at a residence time of 300 days, although removal efficiencies as high as 98% were observed within the data set.

Table 2

Summary of Removal Efficiencies for Selected Stormwater Treatment Ponds in Florida

POND LOCATION	RESIDENCE TIME (days)	MASS REMOVAL (%)		REFERENCE
		TN	TP	
Tampa	2 14	33 61	62 90	Rushton, et al. (1995)
DeBary	19	26	54	Harper and Herr (1993)
Tallahassee (Lake Arrowhead)	49	52	71	Harper, et al. (2000)
Tallahassee (Gilbert Pond)	77	20	60	Harper, et al. (2000)
Tallahassee (Lake McBride)	168	54	76	Harper, et al. (2000)
Tallahassee (Lake Tom John)	114	34	68	Harper, et al. (2000)
Winter Park (Lake Virginia)	220	44	85	ERD (2000)
Winter Park (Lake Osceola)	102	35	71	ERD (2000)
Winter Park (Lake Maitland)	197	35	79	ERD (2000)
Orlando (Lake Lucerne)	105	53	80	Harper and Herr (1991)
St. Petersburg (Mirror Lake)	114	84	92	ERD (1998)
Lakeland (Lake Morton)	328	43	83	Harper, et al. (2002)
Orlando (Lake Eola)	244	89	98	Harper, et al. (1982)

A plot of removal of total nitrogen as a function of residence time in stormwater treatment ponds is given in Figure 4. Removal of total nitrogen as a function of residence time also appears to exhibit a logarithmic shape, although the R-square value of 0.39 is somewhat less than the R-square value observed for total phosphorus. The removal efficiency for total nitrogen appears to become asymptotic at an efficiency of approximately 55%.

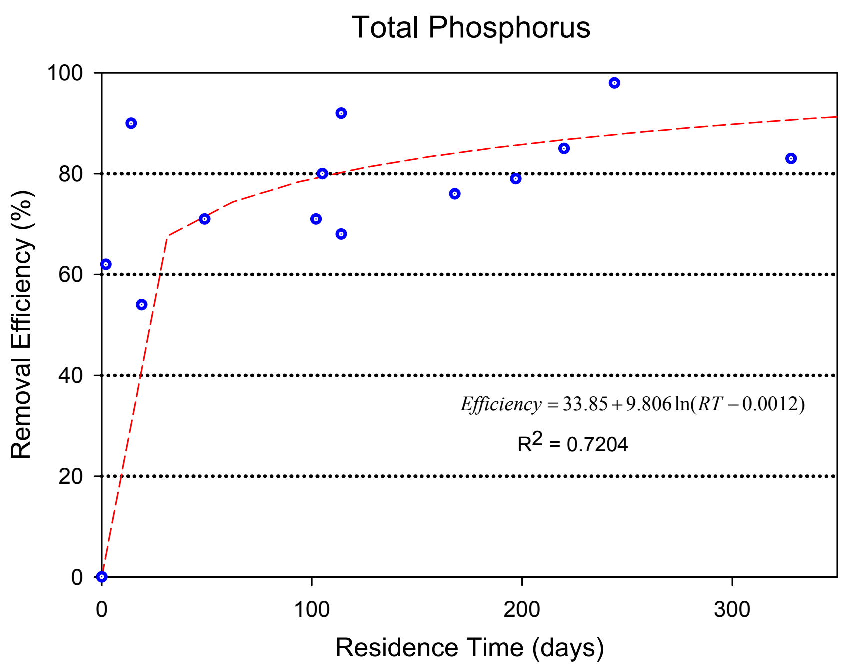

Figure 3. Removal of Total P as a Function of Residence Time.

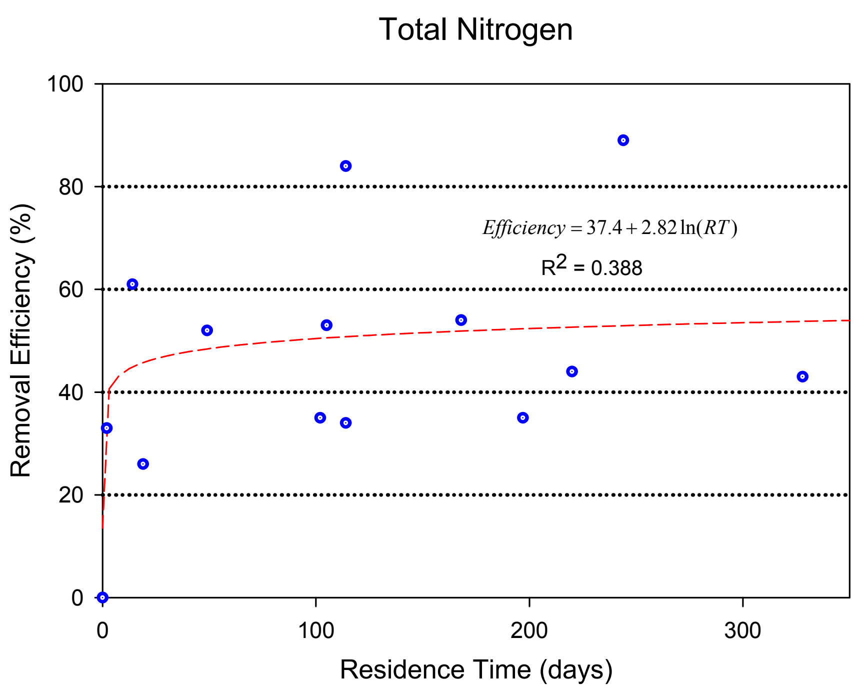

Figure 4. Removal of Total N as a Function of Residence Time

Impacts of Pond Depth on Performance Efficiency

No significant previous studies have been conducted within the State of Florida to evaluate the impact of pond depth on performance efficiency. However, limited water quality monitoring of pond discharges following significant rain events has been required as a permit condition for construction of deep ponds permitted by SJRWMD. A quarterly monitoring program was required by SJRWMD for the Stoneybrook Development, located in southeast Orange County, as part of the permit requirements for construction of seven wet detention ponds to a maximum depth of 20 feet. Characteristics of the constructed deep ponds in the Stoneybrook Development are summarized in Table 3. Pond areas range from 1.23-7.98 acres, with contributing land use consisting of entry road, residential, and golf course areas. Each of these seven ponds was constructed to a maximum depth of 20 feet.

The SJRWMD permit for the project requires water quality monitoring to be performed in each of the seven detention ponds on a quarterly basis after 80% completion of development in each of the watersheds. Water quality samples must be obtained twice daily, at least 6 hours apart, for three days following storm events that produce at least 0.5-inch of rainfall. This monitoring program is designed to detect variability in outfall concentrations of dissolved oxygen, nitrogen species, phosphorus species, and heavy metals following significant rain events which may cause circulation of the entire waterbody to occur. If anaerobic conditions had developed in lower layers of the pond, these conditions would be evidenced by decreases in dissolved oxygen, and increases in species such as ammonia, SRP, and total phosphorus at the pond discharge. The quarterly monitoring program was initiated in 2001 and has continued through 2004, with a total of 11 monitored events.

Table 3

Characteristics of Deep Ponds in the Stoneybrook Development

POND	SURFACE AREA (acres)	MAXIMUM DEPTH (ft)	CONTRIBUTING LAND USE
2-1	1.23	20	Entry road
2-3	2.09	20	Residential
3-1	7.98	20	Residential
5-1	5.51	20	Residential
8-1	3.91	20	Residential
10	4.60	20	Golf Course
11	7.68	20	Golf Course

A statistical comparison of mean variability in discharges from deep ponds in the Stoneybrook Development following storm events is given in Figure 5. Rainfall depths for the monitored storm events range from 0.5-4.4 inches, with a mean rainfall depth of 2.27 inches for the 11 monitored events. As seen in Figure 5, no significant decreases in dissolved oxygen have been observed at the pond outfall even following rain events in excess of 4 inches. None of the monitored outfall events was observed to have dissolved oxygen concentrations less than the Class III criterion of 5 mg/l outlined in Chapter 62-302 FAC. Similarly, no significant increases in specific conductivity, ammonia, total nitrogen, SRP, or total phosphorus were observed in the initial monitoring performed immediately following the storm event which would suggest negative water quality impacts from anaerobic lower layers. No statistically significant differences are present in mean values of dissolved oxygen, conductivity, ammonia, total nitrogen, SRP, or total phosphorus between the six samples collected following each rain event. Based upon the statistical summary presented in Figure 5, the fact that the ponds were constructed to a depth of 20 feet rather than the 12-ft maximum outlined in the SJRWMD regulations does not appear to have negatively impacted discharges from any of the monitored ponds.

A summary of mean characteristics of discharges from deep ponds in the Stoneybrook Development from 2001-2004 is given in Table 4. Discharges from the ponds have been characterized by near-saturation levels of dissolved oxygen, relatively low levels of total nitrogen, and concentrations of total phosphorus similar to those commonly observed in urban lakes. No exceedances of applicable Class III water quality criteria have been observed for lead, zinc, or fecal coliform bacteria.

Table 4

Mean Characteristics of Discharges from Deep Ponds in the Stoneybrook Development from 2001-2004

PARAMETER	UNITS	MEAN VALUE BY POND						
		2-1	2-3	3-1	5-1	8-1	10	11
Diss. Oxygen	mg/l	8.7	7.4	8.3	8.2	7.6	7.9	8.1
Total N	µg/l	1027	807	782	758	801	751	909
SRP	µg/l	2	2	2	2	2	9	7
Total P	µg/l	39	28	23	35	26	33	37
Lead	µg/l	1.3	1.5	1.3	1.3	1.2	1.2	1.2
Zinc	µg/l	8.1	7.8	6.4	5.8	5.8	5.1	3.3
Fecal Coliform	CFU/100 ml	176	174	74	136	181	126	94

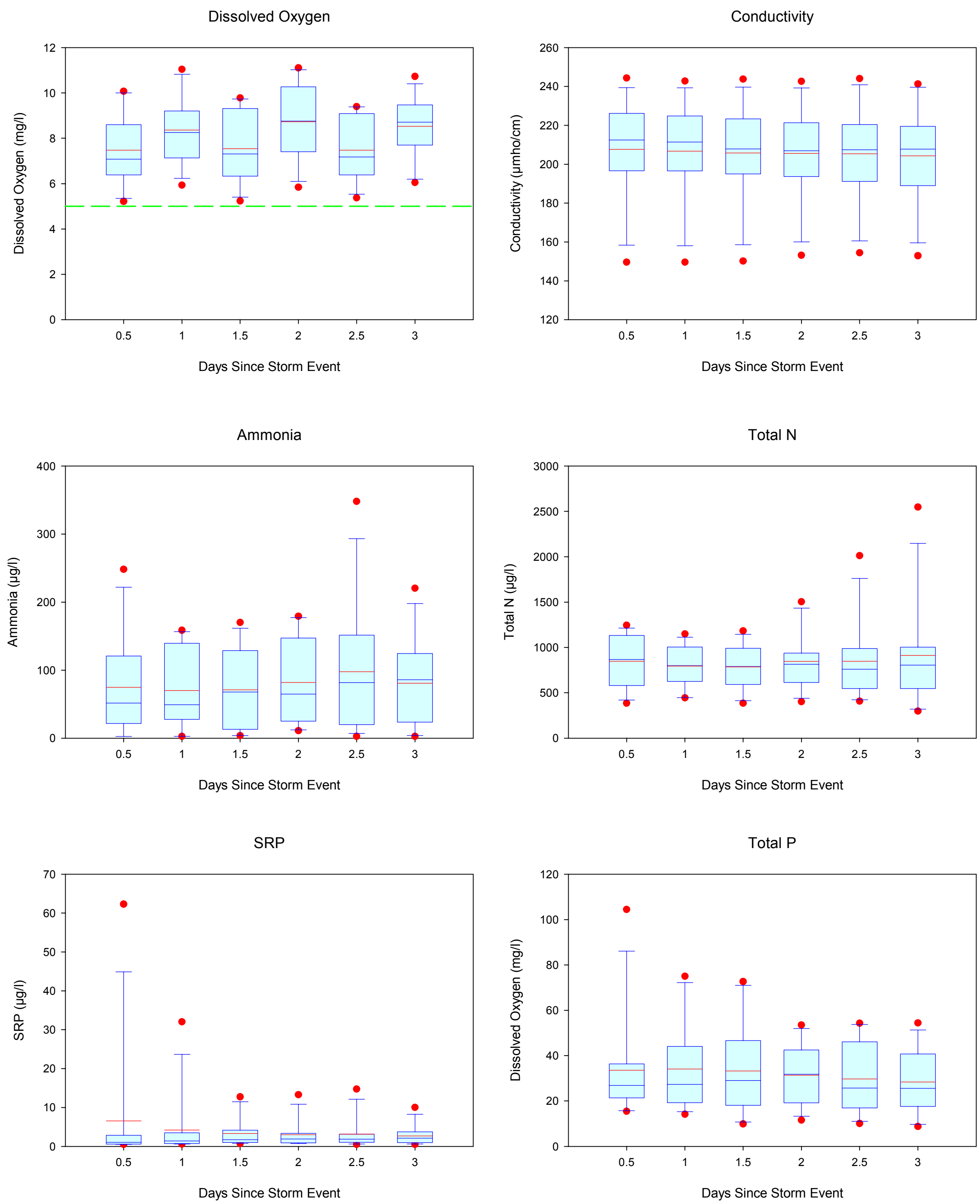

Figure 5. Mean Variability in Discharges from Deep Ponds in the Stoneybrook Development Following Storm Events.

Discussion and Conclusions

Based upon the plots presented in Figures 3 and 4, residence time appears to be significantly correlated with removal efficiencies for both total nitrogen and total phosphorus. Increases in residence time explain approximately 72% of the variability in removal efficiency for total phosphorus, while explaining approximately 39% of the variability in removal efficiency for total nitrogen. The remaining variability in removal efficiencies for total nitrogen is related to differences in the forms of nitrogen present within the pond as well as nutrient limitation dynamics. However, it appears clear that increasing the residence of wet detention ponds can improve removal efficiencies for both total nitrogen and total phosphorus.

Construction of wet detention ponds to a depth of 20 feet does not appear to have a significant negative impact on overall performance of a wet detention system. Circulation of anaerobic water from lower layers of a wet detention pond would be easily observed by substantial variability in concentrations of dissolved oxygen discharging through the pond outfall, particularly during the period immediately following the storm event. The monitoring program performed at the Stoneybrook Development has not revealed any negative water quality impacts associated with ponds constructed to a depth of 20 feet. Further research is recommended to characterize water quality impacts from ponds constructed deeper than 20 feet.

Engineers should be encouraged to design deep wet detention ponds, up to 20 feet in depth, to increase residence time and improve treatment effectiveness. In addition, deeper ponds provide storage for accumulated pollutants in an area of the pond where resuspension is unlikely. The larger permanent pool volume also provides additional protection from shock loads.

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Finding Space for Stormwater in the Urban Environment

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Introduction

Stormwater management in urban areas has always been a challenge due to a lack of space, and new stormwater management programs that require water quality improvement and volume reduction – as well as peak rate reduction – increase this challenge even further. Urban areas often include far more constraints for the stormwater designer than new, lower density development projects.

To manage stormwater effectively in urban areas, the designer needs to find opportunities to incorporate stormwater management into the built environment. This means finding ways to provide stormwater management within the structural elements of the urban environment such as the sidewalks, tree trenches, landscaping features, parking lots, roofs, and even playfields. This "decentralized" approach focuses on managing stormwater where and when it is generated, in both large and small stormwater features that slow down, treat, and reduce the volume of runoff.

In the past, stormwater management has largely consisted of collecting and conveying the stormwater away from the developed area. Requirements for peak rate control, and now for quality and volume management, have often resulted in end-of-pipe designs: detention basins at the lowest, farthest portion of an area, which now may include water quality improvement techniques (such as wetlands) or channel protection techniques (such as extended detention). Conveying stormwater to these elements requires large pipes which become larger and deeper as water is conveyed "away" to an end-of-pipe stormwater element. In older urban areas, runoff may be discharged directly to a stream or combined sewer system with no management of any form.

This paper presents built project examples of a "decentralized" approach to stormwater management, where stormwater is managed for rate, volume, and quality within elements of the built environment. The management of small, frequent storm events is considered as important as the control of large flood events.

It should be noted that term 'urban area' conjures up images of existing towns and cities, however, many of the techniques discussed within this paper are equally applicable to high-density new development projects, and even to low-density new development projects. The concept of managing stormwater where it is generated, and reducing the footprint and disturbance of development is a key component of sustainable design, and recognizing stormwater as part of the water resource is key to meeting future water needs.

Challenges of Stormwater Management in Urban Areas

High-density urban development presents considerable challenges for the stormwater designer, including the following constraints:

1. Urban areas by definition include high densities of imperviousness (50% to 100%) and therefore generate considerable amounts of stormwater. Property values may be high and there may be limited physical space for stormwater management. The developed footprint may include lot-line to lot-line development.
3. Past activities or current construction needs may create compacted soils and areas of fill. Contaminated soils are not uncommon.
4. Underground utilities often cross the project area and may include gas lines, steam tunnels, sanitary sewers, water lines, and telecommunication lines. Concerns about impacting building foundations and basements are also likely.
5. The existing streams may be severely impacted by past activities, and in many cases the original streams have been enclosed or buried in culverts. Streams may be buried beneath a re-development site.
6. Much or all of the original floodplain may also have been eliminated through the placement of fill and structures. Impervious surfaces have been directly connected to storm sewers, resulting in rapid surges of high flow rates, even in moderate rainfall events. In some older areas, stormwater is discharged to combined sanitary-storm sewers, and even small rainfall events trigger discharges to receiving streams.
9. There may be high levels of trash, debris and pet waste being conveyed to stormwater elements and degrading water quality.
10. Urban areas include many stormwater "hot spots": industrial areas, vehicle service areas, public works storage areas, dumpsters, etc. Urban areas often include outdated code regulations and ordinances that may conflict with current BMP design strategies. Local code may require that roof leaders be directly connected to storm sewers.Older urban areas may have limited economic resources and a need to encourage – not discourage – redevelopment. Cost considerations are especially important.

Project Examples

The following projects include examples of urban elements that have also been designed to include stormwater management. These projects include the following project types and examples:

- Inner city urban redevelopment: Penn-Alexander School in Philadelphia and University of Michigan, Ann Arbor

- Campus new development: Pennsylvania State University Visitor Center in State College and Pennsylvania State University - Berks Campus
- Campus retrofits: Villanova University, Swarthmore College, and UNC-Chapel Hill
- Urban area retrofit: Washington National Cathedral in Washington, D.C.
- Residential high density new development: Springbrook Village
- Industrial: Ford Motor Company Rouge Plant

Penn Alexander New School, Philadelphia

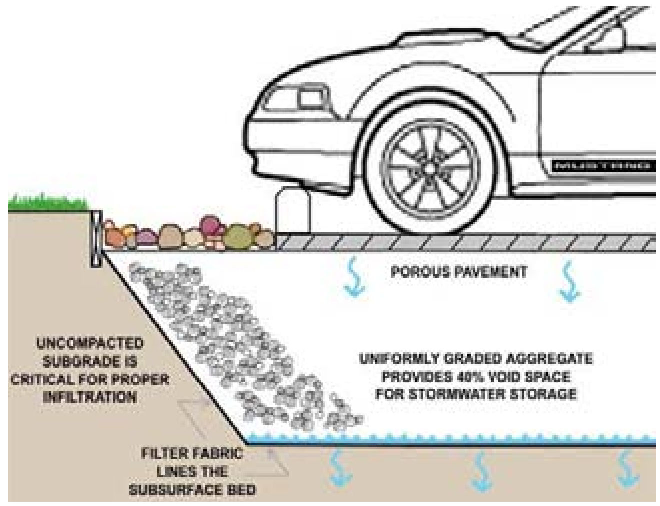
The Penn-Alexander New School is located at 42^{nd} and Locust Streets in Philadelphia, PA in an older area that is served by a combined sewer system. In the 1880's the original Mill Creek was buried in brick culverts that are now some 11 meters (36 feet) below grade. The area has been filled over the years, and was a parking lot prior to the school construction. The stormwater management goals for the project included managing stormwater rate and volume on-site for a 5-cm (2-inch) storm event, and providing educational opportunities for the students. Stormwater elements include a stone storage/infiltration bed located beneath the new athletic field. Roof leaders from approximately one-half of the new building connect directly to this bed, which is comprised of open-graded clean 5-cm (2-inch) stone with 40% void space. A second roof area drains to a Rain Garden that includes a walkway and access platform for student activities. The Rain Garden overflows to a stone storage/infiltration bed located beneath a porous pavement playground. Roof leaders connect directly into the stormwater bed beneath the porous asphalt. The bed includes an overflow structure so that large storm events are able to overflow to the combined sewer system in a controlled manner.

Porous Asphalt Playground

Roof Leaders convey runoff to stormwater bed below playfield.

Challenges to this project included a city plumbing code that required roof leader connection to the sewer, and a detailed investigation of the historic fill conditions, utility locations, and soil conditions was needed to provide design parameters in this urban setting. This project was funded by the PaDEP, the Philadelphia Water Department, and the University of Pennsylvania.

Parking Lot Retrofit, University of Michigan, Ann Arbor



In an effort to reduce urban runoff volume, improve water quality, and reduce flows to the urban storm sewer system, the University of Michigan, Ann Arbor retrofit an existing impervious parking lot with porous asphalt pavement and a sub-surface storage/infiltration bed. Of interest in this project is that the soil investigation uncovered a brick rainwater cistern that had been part of a house located on the site in the 1800's. At that time, all buildings were built with rainwater cisterns for water and fire needs. The concept of holding rainwater in urban areas is an old idea worth remembering.

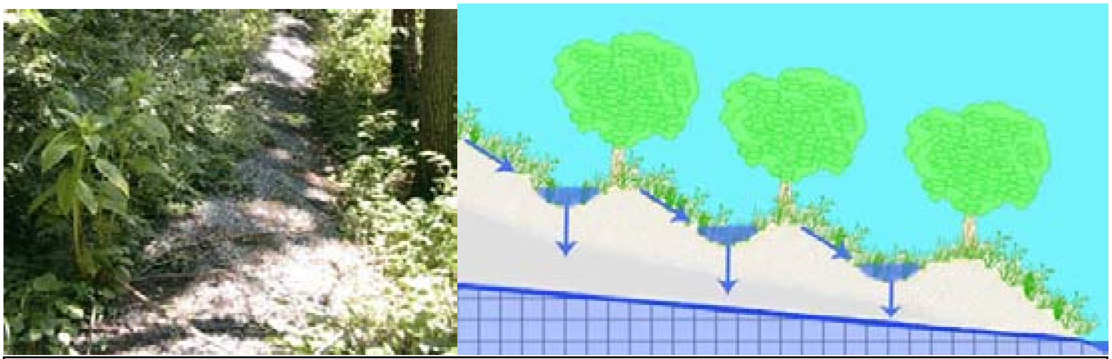
Pennsylvania State University Berks Campus, New Dormitories

The new residential dormitories at the Penn State Berks Campus were built on a wooded hillside, and the intentions were to construct a large 700-car parking lot on the adjoining wooded hillside, removing all the trees and constructing a large detention basin in the valley between. An existing campus detention basin had previously suffered serious sinkhole problems due to the underlying limestone geology.


Existing Woods Protected

The revised "decentralized" approach included a porous pavement parking lot constructed in the valley, leaving the wooded hillside intact. In the dorm area, located on the top of a wooded hill, disturbance was carefully limited to within 3 meters (10 feet) of the new buildings. Infiltration berms on the wooded slopes were used on the downhill side, carefully constructed between the trees and along the contours to receive roof runoff and maintain soil moisture in the existing woods. In the dorm area, roof leaders were connected to stone storage/infiltration beds located beneath standard asphalt paths. To meet the need for fire access, the stone bed is 4.3 meters (14 feet) wide and surfaced with a 3-meter

(10-feet) wide path and 0.6-meters (2-feet) of "Grasspave" on either side. These elements were able to meet all the requirements for rate control such that no detention basin was constructed, and a small existing detention basin was removed.

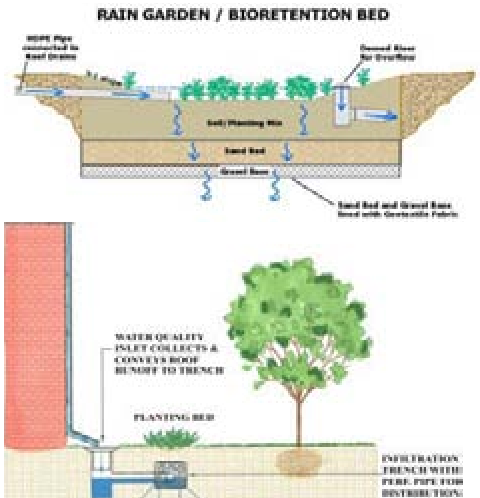


Roof Runoff conveyed to Infiltration Berms in Woods along Contours

Infiltration Beds beneath Asphalt Paths with Grasspave along Edges for Fire Access

Main Campus Visitor Center

At the Penn State Visitor Center in State College, PA, a similar "decentralized" approach was taken that included porous asphalt parking, porous concrete sidewalks, infiltration trenches, and Rain Gardens. With this approach, stormwater was managed within the structural and landscaped elements of the site and the need for a separate detention basin was eliminated. Stormwater management was also provided for an adjacent building by constructing a sub-surface planted infiltration bed. This project was built in a drainage area that is over 60% impervious, and up stream of an intersection that was experiencing flooding due to surcharged storm sewers. The Penn State Civil engineering Department has researched the performance of the system.



Infiltration Trenches and Rain Gardens

Swarthmore College, Villanova University, and UNC Chapel Hill

Urban campus retrofit stormwater techniques have been incorporated at a number of campuses including Swarthmore College, which has been replacing existing walkways with porous asphalt walkways. The campus also has two green roof projects (designed by others). At Villanova University, an existing parking lot between dorms was rebuilt as a plaza with sub-surface storage/infiltration and porous concrete. A proprietary porous concrete mix was initially used in this project and failed, and has been replaced with porous concrete based on a standard specification.

Porous Concrete Sidewalks and Porous Asphalt Paths

Plaza w/ Porous Concrete Strip at Villanova University (right side: during rain)

At UNC Chapel Hill, the Rams Head project includes a new three-story parking garage, with a recreation facility building, dining building, and grassed plaza built on the roof of the new garage. The plaza has been constructed as a vegetated roof system. Cisterns located beneath the brick walkways on the plaza receive roof runoff from the dining and recreational buildings for irrigation needs. Adjacent to the new building, a new athletic field has is being constructed with a sub-surface storage/infiltration bed, and previously buried springs are "daylighted" into a new stream channel. This project is part of compliance with a new town stormwater ordinance for volume management, and part of a program to manage stormwater where it is generated and avoid enlarging existing storm sewers. The campus has also built large porous pavement parking lots. A campus-wide program to reduce runoff has been developed that includes landscape changes (less lawn, more planting beds), green roofs, cisterns, and other measures.

The Village at Springbrook Farms – New Residential Development

Springbrook Village consists of 242 townhouse and quad units and 17 single-family houses constructed on a "closed depression" drainage area underlain by limestone and subject to sinkhole formation. Detention basins and deep, large storm sewers have been avoided by using a "decentralized" design of rain gardens, sub-surface storage/infiltration beds beneath planting areas, stormwater management beneath driveways, and infiltration trenches. Curbs have been eliminated on most streets, and a system of vegetated swales with check dams are used for stormwater storage, conveyance and infiltration.

Washington National Cathedral

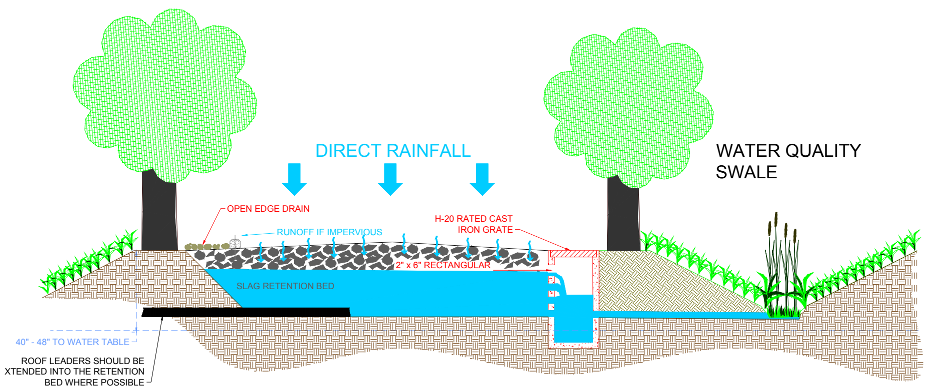
The Washington National Cathedral was built on the top of a wooded hillside where paths through the woods were designed by Olmsted and are part of the spiritual experience of the cathedral. However, the large cathedral, surrounding buildings, and parking areas generate significant volumes of stormwater runoff, creating eroded ravines through the wooded hillside. Because infiltration was prevented by the large impervious areas on the hilltop, the moisture in the woods was significantly reduced and the mature woods were dying. The cathedral had installed a well and was pumping groundwater in an effort to water the trees in this unsustainable cycle.

A "decentralized" approach to restore the water balance to the site included infiltration trenches, including infiltration beneath existing roadways. A large infiltration system was built above the woods using "Rainstore" elements. Checkdams were used to restore the ravines once the amount and rate of flow was reduced. Currently, the cathedral is incorporating infiltration trenches into the design of a new outdoor ampitheater.


Infiltration Trench in Roadway and "Rainstore" Infiltration Bed at Washington National Cathedral

Ford Rouge Plant

At the Ford Rouge Plant in Dearborn Michigan, porous pavement areas with vegetated swales are used to both reduce the amount of runoff and slow the rate of runoff while improving water quality. The plant also includes a green roof (designed by others) as part of an overall stormwater plan to mange stormwater where it is generated.



Porous Pavement and Water Quality Swales at Ford Rouge

An Evaluation of an Effluent Filtration Stormwater Pond System

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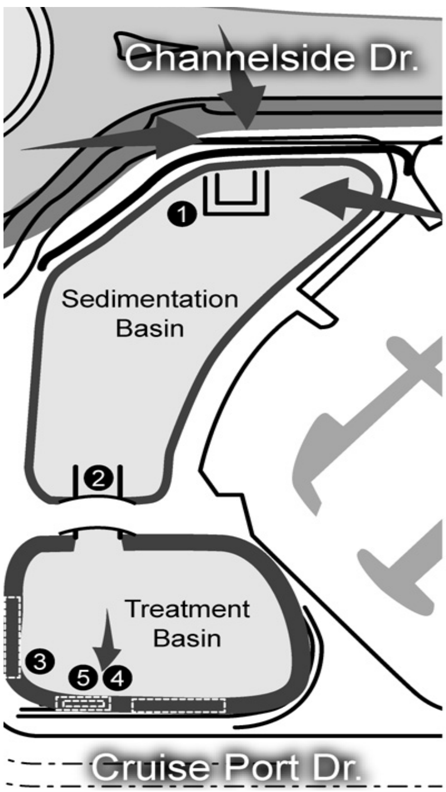
Abstract

The stormwater pond is located at the Florida Aquarium in Tampa and is designed to treat 4.2 hectares (10.4 acres) of street and urban runoff. It is an effluent filtration system that incorporates artificial side bank filters packed in aggregate to slowly release storm runoff after rain events. The pond was monitored throughout a two-year period. Year 1 includes data from November 2000 through August 2001. During this drought year 16 rain events were monitored. Monitoring was discontinued for eight months during the construction of a cruise ship terminal. Year 2 includes data from June 2002 through October 2003. The longer time period and above average rainfall resulted in water quality samples for 38 rain events. The under drains were also sampled for both flow and water quality. Load efficiency calculations were made on a monthly basis, since the rain, inflow and bypass outflow occurred only during rain events, but the under drains flowed continuously. The results indicate that the stormwater system is not effective for removing dissolved nutrients and may be dewatering the ground water. Ten percent more water left the site compared to storm event flows into the pond. Efficiency calculations on a yearly basis showed that dissolved nutrient loads increased considerably. For example, an increase of 84 percent was measured for ammonia and 64 percent for ortho-phosphorous when the entire system was evaluated. Other nutrient species were only moderately reduced (14 to 17 percent). In contrast, total suspended solids and most metal loads were reduced by a significant amount (79 to 89 percent). Only a few samples at the bypass outflow did not meet the State of Florida Class III water quality standards in Year one (12% for lead), while a much greater number failed to meet standards in year two (27% for copper, 23% for lead and 4% for zinc). Although higher concentrations were measured in year two, the greater number of non-compliant samples was also caused by the greater amount of rain, which made the pond water much softer and the standard more stringent. A more complete report is available from the author by request.

Introduction

The problems associated with the increased volume of runoff and pollution loads caused by urbanization has resulted in regulations to treat storm discharge water before it enters the nation's lakes, streams, rivers and bays. In addition, new rules to develop total maximum daily loads (TMDLs) and data required by computer models have emphasized the need for reliable information about individual stormwater treatment techniques. The evaluation of an effluent filtration stormwater system summarized in this report was designed to fill one of those data needs. The results presented here include: 1) estimating a monthly water budget, 2) measuring the reduction (or increase) of pollutants treated, and 3) comparing pollutant buildup in the sediments over time. This paper reports the results from two years of data collection. Sampling began in November of 2000 and concluded in October 2003. Sampling was halted from August 2001 to June 2002 during the construction of a cruise ship terminal. This break in sampling splits data into Year 1 and year 2.

The study site is located at The Florida Aquarium in downtown Tampa (Figure 1).



The pond was designed to treat 4.2 hectare (10.4 acres) of street and urban stormwater runoff. It is an effluent filtration system that incorporates artificial side bank filters packed in aggregate. According to the Southwest Florida Water Management (SWFWMD) rules, detention ponds with effluent filtration systems must treat one-half inch of runoff for drainage areas less than 40.47 hectares (100 acres). The filter medium of such systems must meet the Florida Department of Transportation road and bridge specifications, be of effective grain size and contained in a way that they do not move. Stormwater must pass through at least 0.61 meters (2 feet) of filter before entering the under drain and not be held longer than 36 hours in the filtration system. The pond actually consists of two ponds connected in the middle by an equalizer pipe. The first pond is designed to act as a sedimentation basin, and the second pond includes the side bank filtration system, which discharges to the outflow drop box (Figure 2).

Figure 1. Site map showing the pond system in year two: 1) inflow, 2) equalizer pipe, 3) south under drain, 4) north under drain, and 5) bypass weir. Dotted outlines delineate the underground filters and the bypass weir.

Methods

A summary of the methods used to evaluate the site is presented below. A more complete description is available (Teague et al. 2004).

Hydrology. Rainfall was recorded with a tipping bucket rain gauge, inflow was measured with a velocity meter in a full pipe, bypass outflow was measured using water level and weir formulas, and the under drain flow was estimated from water level and Thel-Mar™

weirs installed in the pipes. Under drain measurements were only a best estimate, since flows were small and erratic. Other estimates included: an additional inflow pipe that drained 10 percent of the drainage basin and was included as 10 percent additional inflow. Evapotransporation was estimated using data from a previous study. The hydrology data was used to calculate a monthly water budget with the standard formula:

Inflow – Outflow = Change in storage.

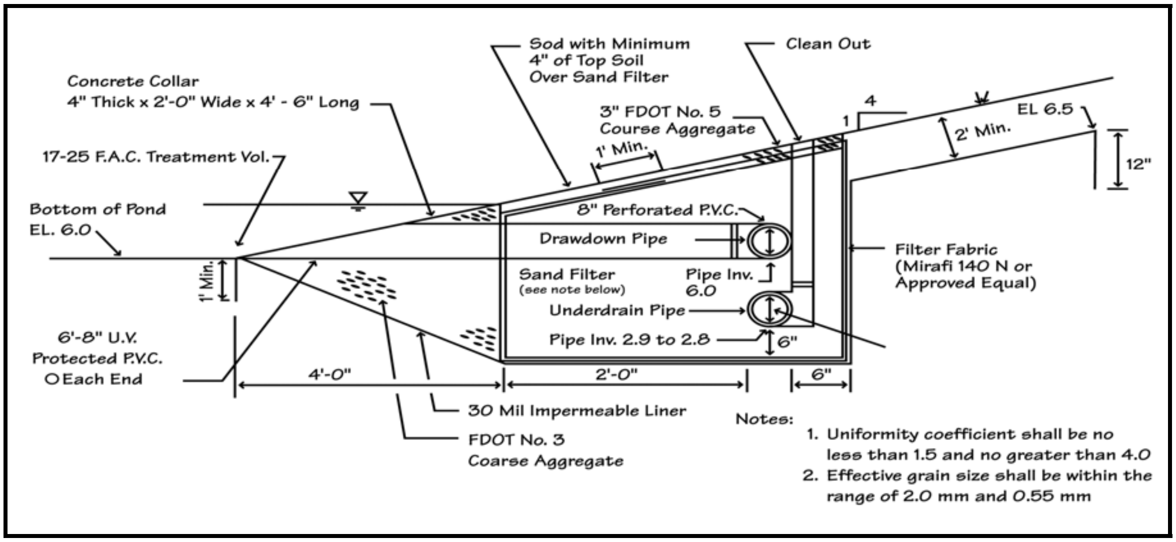


Figure 2. Cross section of the south side bank filter (effluent filtration) system.

Water Quality. Flow weighted water quality samples for rainfall, inflow and bypass flow were taken with automated samplers during storm events. Since the under drains flowed continuously, they were samples twice a month by taking flow weighted samples over several days. Samples were analyzed in the SWFWMD laboratory using standard methods.

Sediments: Sediment samples were taken at three locations in the pond using coring devices. Two strata were analyzed separately the top inch and the level four to five inches below the surface, but only the results for the top inch are presented here.

Results and Discussion

<u>**Hydrology**</u>. Year one and year two represent two different hydrologic regimes. During the first year of the study (ten months), the area was experiencing a drought and rainfall was considerably below normal 66 cm (26 inches) compared to the long-term average of 111.8 cm (44 inches). During the seventeen months of data collected during the second year, El Nino conditions increased rainfall at the site to above average levels 254 cm (100 inches) compared to the long-term average of 223.5 cm (88 inches).

Although a water budget was calculated on a monthly basis for the entire second year, only the months used to calculate pollutant loads (to be discussed later) are shown in Table 1. The effect of seepage from groundwater into the under drain pipes complicated calculating the water budget, which is evident by the high error values for groundwater seepage, which also included the error term. Hydrographs constructed from the under drain data varied with pond water levels and indicated the filters were operational (Teague et al. 2004). The graphs also showed that when the pond level was below the elevation of the under drain intake pipes that considerable flow continued and this flow was attributed to groundwater.

Table 1. Water budget calculated during Year 2 (July 2002 to June 2003). Negative values denote flow out of the system or decreased level of the pond.

MONTH	RAIN	RAIN ON POND	INFLOW	10% EST. N.PIPE	OVER WEIR	UNDER DRAIN SOUTH	UNDER DRAIN NORTH	ET (est)	CHANG E IN ST'G	GROUND WATER & error
	cm	$meter^3$	$meter^3$	$meter^3$	$meter^3$	$meter^3$	$meter^3$	$meter^3$	$meter^3$	$meter^3$
Jul-02	3.7	124.8	1474.4	147.4	-5.9	-563.4	-563.4	-199.6	-333.8	748.3
Aug-02	8.2	279.2	3497.1	349.7	-1041.1	-1611.8	-1961.5	-178.6	285.0	-951.9
Sep-02	9.0	305.0	4614.5	461.5	-1619.4	-3052.5	-3114.1	-142.5	-105.8	-2441.7
Oct-02	7.0	93.3	606.7	60.7	0.0	-929.2	-1000.9	-126.0	32.6	-1328.0
Nov-02	5.8	77.7	782.2	78.2	-80.9	-1742.8	-1424.2	-112.0	-203.6	-2218.2
Dec-02	28.7	382.7	5883.7	588.4	-4557.9	-1737.7	-2076.4	-63.0	171.0	-1751.3
Jan-03	9.8	131.3	1978.7	197.9	-1408.6	-703.6	-801.4	-42.0	-346.0	-301.7
Feb-03	7.2	96.7	2949.6	295.0	-10.8	-1537.6	-1373.4	-56.9	651.4	-288.9
Mar-03	11.9	158.8	3327.3	332.7	-769.4	-2089.8	-2099.8	-105.0	-346.0	-899.1
Apr-03	10.4	139.1	4082.9	408.3	-478.9	-1273.0	-1685.0	-142.5	-77.3	1128.1
May-03	4.7	62.4	3975.6	397.6	0.0	-836.4	-1346.0	-168.1	-256.5	2341.6
Jun-03	27.0	360.3	5363.2	536.3	-1808.6	-1618.1	-2345.1	-183.2	529.2	-224.3
TOTAL	**133.4**	**2,211**	**38,536**	**3,854**	**-11,781**	**-17,696**	**-19,791**	**-1,519**	**0**	**-6,187**

** Two days of data were omitted in June due to equipment malfunction.

Water Quality. Flow-weighted water quality samples were collected for surface water during storm events and for the under drains on a regular schedule to determine the ability of the pond system to remove pollutants (Figure 3).

Nutrients. The ambient water quality criteria recommendations for Ecoregion 12 (US EPA 2000) for rivers and streams are 0.02 mg/l for nitrate, 0.90 mg/l for total nitrogen and 0.04 mg/l for total phosphorus. For year 1, the median levels of nitrogen met criteria recommendations at both the outflow weir and the under drains. For nitrate levels, median concentrations at the outflow for Year 1 (0.02 mg/l) also met criteria, but the median concentrations in the under drain pipes (0.09 mg/l) failed to meet the recommendations. Total phosphorus failed to meet the criteria at the bypass outflow weir for both years (0.08 mg/l and 0.06 mg/l) and in the under drains for both years (0.12 mg/l). Also the concentrations of nutrients, especially in the soluble form, were almost always higher in the under drains than in the pond (Figure 3). Other studies have also found higher levels of inorganic nitrogen and phosphorus in the under drain pipes of effluent filtration systems (Harper and Herr 1993). Trapped organic particles of nitrogen and phosphorous on the filter media were listed as probable causes.

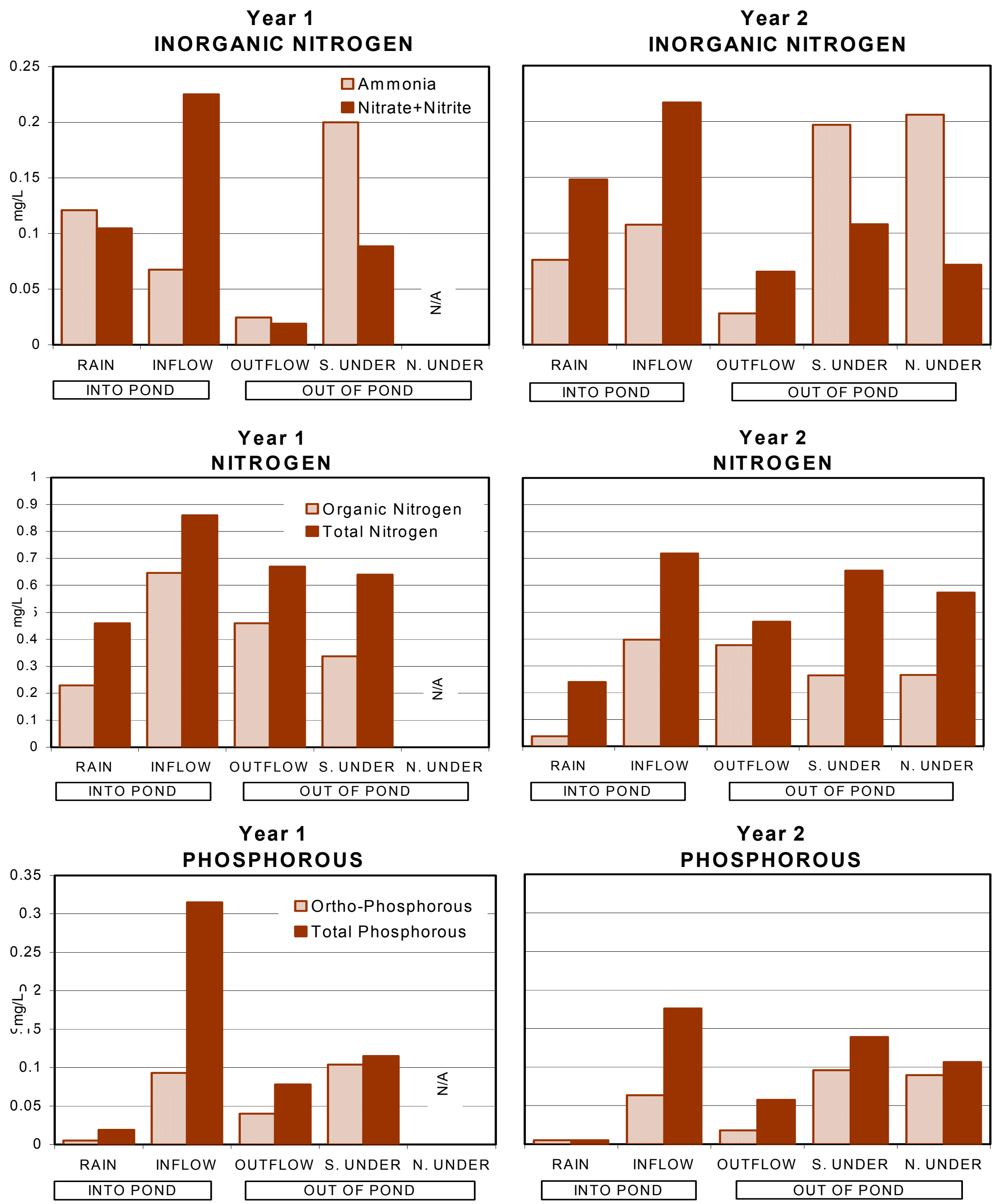


Figure 3. Comparison of average water quality concentrations measured in rainfall, surface discharge, and under drain discharge.

The fact that the soluble forms of nutrients (ammonia, nitrate and ortho-phosphorus) were measured at concentrations over twice as high as concentrations discharged over the bypass weir is of even greater concern when one considers that 77 percent of the stormwater measured exits the system by way of the under drain pipes (see water budget section). To investigate this further an analysis of the data were made, which divided the under drain discharge into storm flow and flow discharged when the water level in the pond was below the level of the draw down pipes (Figure 4). The concentrations of the soluble nutrients were significantly higher when the level of the pond was below the level of the draw down pipes indicating that higher nutrient concentrations are being discharged between storm events and must be coming from groundwater input.

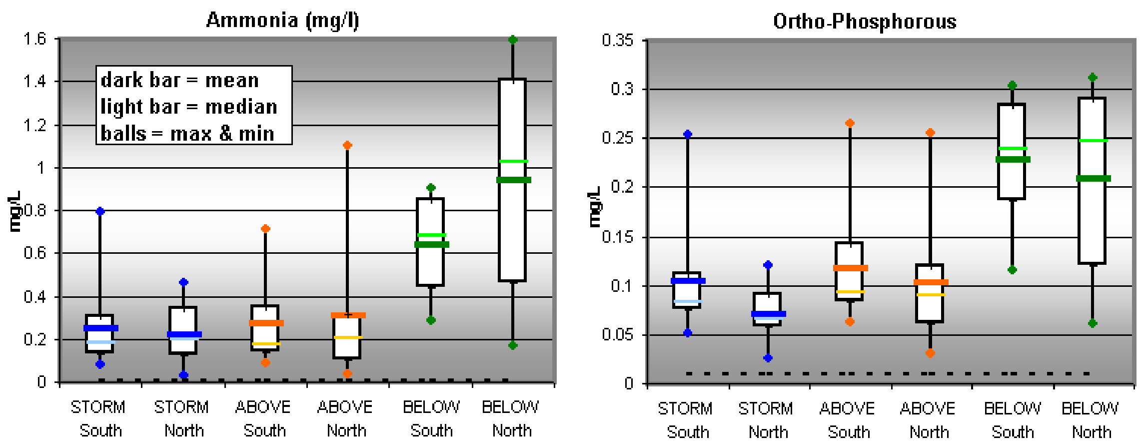


Figure. 4. Box plots of under drain flow for ammonia and ortho-phosphorous during different pond levels. Key: storm event input (STORM), above the bottom of the under drain pipe (ABOVE), and below the bottom of the under drain intake pipes (BELOW).

Metals. Metals are a concern in urban runoff and the State of Florida has developed standards to protect fish and wildlife in receiving waters. The metal concentrations discharged from the under drain pipes were low and always met standards, but the discharge over the bypass weir failed to meet standards on several occasions (Table 2).

Table 2. Summary data comparing metal concentrations measured at the outflow to metal class III standards (FDEP 2003). All values are ug/l. Exceedance (%) represent the percent of samples that failed to meet state standards.

YEAR 1	TOTAL COPPER	STD. COPPER	TOTAL LEAD	STD. LEAD	TOTAL ZINC	STD. ZINC
Number	16	16	16	16	15	16
Average	4.0	9.9	1.4	2.5	12.6	89.0
Median	3.0	9.5	0.8	2.3	7.5	85.5
Exceedance (%)	0.0%		12.5%		0.0%	
YEAR 2						
Number	26	26	26	26	26	26
Average	8.4	7.2	4.0	1.5	28.9	64.4
Median	4.0	6.8	2.0	1.4	20.0	61.0
Exceedance (%)	26.9%		23.1%		3.8%	

Since concentrations of metals in soft water are more harmful to wildlife, the criteria require a unique standard for each water quality sample based on water hardness. The standard was almost always met in year 1, a drought year, but was often exceeded during a year with more rainfall. Although concentrations of metals increased in the second year, the rainfall also increased water softness, which make the standard more stringent.

Pollutant Loads. An effort was made to estimate pollutant loads for the effluent filtration pond (Table 2), but these results depend on some estimated under drain data. Load calculations were difficult to make for several reasons. 1) The under drains flowed continuously and much of this was believed to be groundwater since flow continued even when the level of the pond was far below the intake pipes. 2) Some base flow probably entered the pond, but the flow was too weak to be measured with the velocity sensor. 3) The small eight-inch diameter under drain pipes were equipped with weir structures, but the results depended on measurements of an inch or two while the accuracy of the level sensor was less than half an inch. In addition, these level measurements drifted and required constant adjustments. 4) The amount of flow out of the under drain pipes was greater than could be explained by the reduction in pond levels.

The load calculations were made on a monthly basis since the rain, inflow and weir outflow occurred only during rain events, but the under drains flowed continuously. The under drains were not necessarily sampled during rain events, but were sampled twice a month with composite samples covering several days. The results indicate that the stormwater system is not effective for removing dissolved nutrients and may be dewatering the ground water (Table 3). Ten percent more water was estimated leaving the system compared to storm event flows into the pond. Dissolved nutrient loads increased considerable in the system. For example, an increase of 84 percent was measured for ammonia and 64 percent for ortho-phosphate; and other nutrient species were only moderately reduced (14 to 17 percent). In contrast, total suspended solids and most metal loads were reduced by a significant amount (79 to 89 percent).

Sediment Samples. Sediment samples were taken in October 1997, November 2000, and December 2003. Between the 2000 and 2003 sampling events, many disturbances occurred which affected the quality of the pond sediments. A parking garage was built in the drainage basin. In addition the cruise ship terminal was constructed at the outflow and a trolley line was built next to the inflow. During this interval, the sedimentation pond was altered and the north under drain was added to the pond. Not only was the entire pond system altered, but the existing sediment column was disturbed in the treatment filtration basin when it was reshaped.

Nutrients. When the final sediment samples were taken in 2003 there was a tremendous increase in both nitrogen and phosphorous. This increase is attributed to the frequent cover of duckweed and associated floating macrophytes that often covered the pond, but would then die, sink to the bottom of the pond, release nutrients into the water column and start the process over again. There was also a trend for nutrients to be measured at higher concentrations at the outflow.

Table 3. Monthly loads measured entering and leaving the pond (Eff% positive=reduction and negative=increase loads).

HYDROLOGY cubic meters	RAIN	INFLOW	OUTFLOW	S.UNDER	N.UNDER
Jul-02	124.8	1,621.7	-5.9	-563.3	-563.3
Aug-02	279.2	3,846.4	-1,041.0	-1,611.6	-1,961.3
Sep-02	305.0	5,075.5	-1,619.3	-3,052.2	-3,113.8
Oct-02	93.3	667.3	0.0	-929.1	-1,000.8
Nov-02	77.7	860.3	-80.9	-1,742.7	-1,424.1
Dec-02	382.6	6,471.4	-4,557.5	-1,737.6	-2,076.2
Jan-03	131.3	2,176.4	-1,408.4	-703.6	-801.3
Feb-03	96.7	3,244.2	-10.8	-1,537.5	-1,373.3
Mar-03	158.8	3,659.7	-769.3	-2,089.6	-2,099.6
Apr-03	139.1	4,490.7	-478.9	-1,272.9	-1,684.9
May-03	62.4	4,372.7	0.0	-836.4	-1,345.8
Jun-03	360.3	5,899.0	-1,808.4	-1,617.9	-2,344.9
TOTAL	2,211	42,386	-11,780	-17,694	-19,789
IN/OUT		44,597			-49,264
EFF%					-10%

AMMONIA grams	RAIN	INFLOW	OUTFLOW	S.UNDER	N.UNDER
Jul-02	29.4	1,346.0	-0.8	-684.6	-415.1
Aug-02	237.1	373.1	-21.9	-665.7	-382.5
Sep-02	38.5	253.8	-13.8	-520.7	-634.1
Oct-02	60.3	115.4	0.0	-240.2	-449.9
Nov-02	23.7	287.4	0.0	-716.7	-758.4
Dec-02	50.0	637.5	-109.4	-237.2	-326.0
Jan-03	0.0	0.0	0.0	-474.3	-849.5
Feb-03	16.0	19.5	-0.2	-291.4	-405.9
Mar-03	146.2	324.5	-21.5	-584.1	-873.5
Apr-03	295.9	1,944.7	-63.7	-493.9	-1,043.6
May-03	2.9	708.5	0.0	-463.7	-772.1
Jun-03	133.0	438.5	-40.7	-540.3	-682.7
TOTAL	1,033	6,449	-272	-5,913	-7,593
IN/OUT		7,482			-13,778
EFF%					-84%

NITRATE + NITRITE grams 0.01	RAIN	INFLOW	OUTFLOW	S.UNDER	N.UNDER
Jul-02	16.8	622.8	-0.4	-78.2	-62.4
Aug-02	68.4	1,092.5	-75.0	-328.8	-365.0
Sep-02	28.8	609.1	-18.2	-298.9	-207.3
Oct-02	14.9	200.2	0.0	-161.7	-87.1
Nov-02	13.6	242.6	0.0	-283.6	-184.4
Dec-02	23.9	763.7	-330.5	-682.9	-535.7
Jan-03	0.0	0.0	0.0	-60.5	-69.7
Feb-03	10.2	509.4	-0.1	-232.2	-46.0
Mar-03	17.3	575.9	-51.9	-324.4	-119.0
Apr-03	33.5	1,562.9	-144.2	-322.5	-264.6
May-03	4.9	760.9	0.0	-205.8	-117.1
Jun-03	70.4	1,249.7	-122.5	-215.0	-133.1
TOTAL	303	8,190	-743	-3,195	-2,191
IN/OUT		8,492			-6,129
EFF%					28%

ORGANIC NITROGEN grams 0.06	RAIN	INFLOW	OUTFLOW	S.UNDER	N.UNDER
Jul-02	4.2	573.3	-5.2	-130.6	-197.7
Aug-02	0.0	1,400.2	-254.8	-353.6	-391.0
Sep-02	11.4	1,167.5	-550.6	-736.3	-1,186.3
Oct-02	2.8	338.3	0.0	-216.0	-293.8
Nov-02	0.0	304.6	0.0	-406.9	-470.7
Dec-02	22.2	1,349.4	-1,451.7	-391.9	-487.9
Jan-03	0.0	0.0	0.0	-218.1	-138.6
Feb-03	2.4	1,807.2	-5.1	-262.2	-362.6
Mar-03	24.2	2,074.0	-284.7	-413.3	-498.7
Apr-03	7.7	2,645.3	-314.2	-332.3	-502.1
May-03	5.2	1,661.8	0.0	-237.8	-457.6
Jun-03	16.2	2,901.6	-862.7	-466.0	-721.0
TOTAL	96	16,223	-3,729	-4,165	-5,708
IN/OUT		16,319			-13,602
EFF%					17%

TOTAL NITROGEN grams	RAIN	INFLOW	OUTFLOW	S.UNDER	N.UNDER
Jul-02	33.1	1,654.2	-5.9	-441.8	-401.4
Aug-02	136.0	3,262.1	-313.9	-1,348.1	-1,138.5
Sep-02	52.5	2,974.5	-604.9	-1,555.9	-2,027.7
Oct-02	42.0	654.0	0.0	-617.9	-830.7
Nov-02	15.5	834.6	0.0	-1,407.3	-1,413.5
Dec-02	63.1	2,750.6	-1,891.5	-1,312.0	-1,349.6
Jan-03	0.0	0.0	0.0	-752.9	-1,057.8
Feb-03	18.0	2,336.0	-5.3	-785.7	-814.4
Mar-03	91.3	2,974.4	-358.1	-1,321.8	-1,657.5
Apr-03	141.9	6,152.9	-522.1	-1,148.7	-1,810.3
May-03	11.1	3,131.2	0.0	-907.2	-1,346.9
Jun-03	130.7	4,589.9	-1,025.9	-1,221.3	-1,536.8
TOTAL	735	31,314	-4,728	-12,821	-15,385
IN/OUT		32,050			-32,933
EFF%					-3%

ORTHO- PHOSPHATE grams	RAIN	INFLOW	OUTFLOW	S.UNDER	N.UNDER
Jul-02	0.6	190.4	-0.1	-89.2	-53.8
Aug-02	1.4	269.1	-15.6	-252.5	-135.0
Sep-02	1.5	294.2	-18.6	-250.6	-270.3
Oct-02	0.5	32.0	0.0	-82.2	-78.5
Nov-02	0.4	100.6	0.0	-233.8	-171.5
Dec-02	1.9	307.2	-123.0	-125.9	-134.9
Jan-03	0.0	0.0	0.0	-186.3	-205.0
Feb-03	0.7	113.5	-0.2	-155.2	-129.7
Mar-03	1.4	104.8	-8.8	-272.0	-266.5
Apr-03	0.7	327.6	-9.6	-194.6	-236.3
May-03	0.3	559.4	0.0	-166.9	-185.2
Jun-03	1.8	406.8	-16.3	-213.6	-185.7
TOTAL	11	2,706	-192	-2,223	-2,052
IN/OUT		2,717			-4,467
EFF%					-64%

TOTAL - PHOSPHATE grams	RAIN	INFLOW	OUTFLOW	S.UNDER	N.UNDER
Jul-02	0.6	367.3	-0.3	-200.2	-80.8
Aug-02	1.4	769.4	-64.0	-278.5	-172.1
Sep-02	1.5	873.1	-84.2	-309.6	-366.1
Oct-02	2.0	180.8	0.0	-118.9	-145.1
Nov-02	0.4	296.0	0.0	-259.7	-212.2
Dec-02	1.9	491.9	-255.2	-148.6	-177.5
Jan-03	0.0	0.0	0.0	-201.2	-223.6
Feb-03	0.5	243.3	-0.7	-186.1	-165.5
Mar-03	4.4	534.4	-46.9	-306.2	-330.4
Apr-03	0.7	1,073.4	-51.7	-224.1	-293.8
May-03	0.3	800.3	0.0	-183.5	-220.7
Jun-03	1.8	1,201.5	-102.6	-245.8	-247.5
TOTAL	15	6,831	-606	-2,662	-2,635
IN/OUT		6,847			-5,903
EFF%					14%

TOTAL COPPER milligrams	RAIN	INFLOW	OUTFLOW	S.UNDER	N.UNDER
Jul-02	187.3	28,138.3	-6.7	-1,149.3	-709.9
Aug-02	2,177.8	82,321.6	-3,904.0	-2,788.4	-2,984.3
Sep-02	350.7	81,723.6	-5,101.2	-4,317.1	-4,226.3
Oct-02	233.2	21,688.5	0.0	-1,695.8	-1,826.6
Nov-02	89.3	29,426.3	0.0	-2,004.2	-2,848.4
Dec-02	440.1	116,173.2	-47,402.5	-6,950.9	-7,578.7
Jan-03	0.0	0.0	0.0	-809.2	-1,843.2
Feb-03	128.1	65,539.1	-41.9	-1,873.6	-4,051.6
Mar-03	238.2	91,136.0	-2,850.5	-2,762.7	-3,149.7
Apr-03	208.6	180,095.2	-5,555.8	-2,548.6	-5,521.3
May-03	93.6	89,649.6	0.0	-1,074.0	-2,750.3
Jun-03	540.4	151,982.6	-7,505.7	-2,368.2	-3,517.6
TOTAL	4,687	937,874	-72,368	-30,342	-41,008
IN/OUT		942,561			-143,718
EFF%					85%

TOTAL IRON milligrams	RAIN	INFLOW	OUTFLOW	S.UNDER	N.UNDER
Jul-02	2,200	1,524,496	-410	-10,299	-12,023
Aug-02	8,376	5,193,186	-176,982	-106,377	-162,527
Sep-02	4,575	4,517,640	-259,108	-274,725	-462,670
Oct-02	6,531	1,134,475	0	-134,731	-350,300
Nov-02	1,165	1,445,503	0	-143,782	-245,672
Dec-02	5,740	2,621,179	-1,162,273	-156,394	-259,546
Jan-03	0	0	0	-63,327	-88,154
Feb-03	1,624	901,974	-2,748	-65,733	-86,938
Mar-03	2,929	3,578,338	-160,029	-113,108	-227,410
Apr-03	5,091	7,006,198	-129,316	-101,883	-189,342
May-03	390	2,772,576	0	-51,580	-93,679
Jun-03	11,697	8,680,228	-405,579	-128,564	-280,786
TOTAL	50,319	39,375,793	-2,296,444	-1,350,504	-2,459,047
IN/OUT		39,426,112			-6,105,996
EFF%					85%

TOTAL ZINC milligrams	RAIN	INFLOW	OUTFLOW	S.UNDER	N.UNDER
Jul-02	936	103,390	-44	-10,536	-3,718
Aug-02	2,094	384,680	-26,027	-23,371	-25,220
Sep-02	2,287	456,840	-40,486	-33,796	-44,487
Oct-02	1,866	80,081	0	-12,776	-20,017
Nov-02	2,331	137,667	0	-51,195	-25,813
Dec-02	2,870	194,161	-91,159	-13,033	-15,573
Jan-03	0	0	0	-5,277	-6,010
Feb-03	658	144,705	-280	-17,767	-10,040
Mar-03	1,866	295,856	-18,119	-34,434	-13,243
Apr-03	516	529,956	-16,811	-5,512	-10,784
May-03	594	238,337	0	-11,562	-36,072
Jun-03	2,396	659,965	-29,571	-7,610	-16,528
TOTAL	18,414	3,225,638	-222,496	-226,869	-227,506
IN/OUT		3,244,052			-676,870
EFF%					79%

TSS grams	INFLOW	OUTFLOW	S.UNDER	N.UNDER
Jul-02	16,218	-47	-890	-1,042
Aug-02	146,179	-4,685	-1,805	-3,082
Sep-02	119,286	-6,883	-3,619	-5,143
Oct-02	25,959	0	-2,769	-3,218
Nov-02	48,958	0	-1,988	-2,731
Dec-02	81,839	-23,542	-3,015	-3,748
Jan-03	0	0	-739	-994
Feb-03	15,866	0	-1,462	-2,568
Mar-03	101,994	-2,908	-2,176	-4,315
Apr-03	171,562	-2,993	-2,364	-6,358
May-03	61,224	0	-1,072	-2,019
Jun-03	286,522	-9,929	-2,909	-3,518
TOTAL	1,075,607	-50,987	-24,808	-38,736
IN/OUT	1,075,607			-114,531
EFF%				89%

KEY:

RAIN-Rain on the pond
INFLOW = Inflow into pond thru storm drain
OUTFLOW= Discharge over outflow weir
S.UNDER=South under drain discharge*
N. UNDER= North under drain discharge*

*It should be remembered that the under drains discharge all the time and much of this is ground water.

EFF%= Load efficiency. Reduction or increase of pollutant loads from the inflow to the outflow. A negative % = increase load

IN/OUT= total in/total out (32,050 grams T.Nitrogen in to 32,933 grams out) For example, 3% more total nitrogen was measured leaving the pond than entered.

Table 4. Concentration of nutrients (mg/kg) and metals (ug/kg) measured in the top inch of sediments for different years. Sediments were measured at the inflow, near the equalizer pipe (mid) and near the outfall weir.

Year	Inflow	Mid-pond	Outflow	Year	Inflow	Mid-pond	Outflow
Total Kjeldahl Nitrogen				Total Lead			
1997	73	230	240	1997	0	0	15
2000	12	29	228	2000	9	15	37
2003	12000	12000	24000	2003	7	7	27
Total Phosphorus				Total Copper			
1997	190	440	470	1997	4	8	42
2000	300	440	920	2000	12	29	228
2003	20000	19000	28000	2003	11	10	11
Total Iron				Total Zinc			
1997	842	1030	2650	1997	13	25	42
2000	1040	1370	1740	2000	48	79	125
2003	2230	1120	1780	2003	45	26	36

Metals. Metal concentrations exhibit a different pattern. Concentrations were highest in 2000 before the treatment filtration pond was reshaped and clean bottom sediments exposed (Table 4). When compared to numerical sediment quality assessment guidelines for Florida Inland Water (FDEP 2003), sediments do not appear to be a problem for macroinvertebrates or fish, although samples were slightly above the threshold level for both lead and zinc in 2000. The threshold level has been established as the lowest concentration that may create a problem for sensitive species and for zinc it is 120 ug/kg and for lead it is 36 ug/kg. Copper demonstrates the same pattern as zinc and lead, but concentrations reached a level where they were probably toxic to organisms in 2003 (Table 4). The threshold effect level is 32 ug/kg for copper and the probable effect level is 150 ug/kg.

Polycyclic Arormatic Hydrocarbons (PAHs). Sediment samples were tested for more than 100 organic pollutants but there is not enough space to show this data in this report. The data showed that there were no PAHs detected in the pond except at the inflow in 2000 and it was estimated that only 17 percent of the samples tested detected PAHs. By 2003 this ratio had changed dramatically and 63 percent of the samples analyzed detected PAHs and they were detected in the surface sediments at all stations tested. Some of these concentrations were above the detection limit and indicated a possible toxicity problem. Although more PAHs were measured in the surface sediments, both strata show an increasing trend over time.

Pesticides. Pesticides measured in the sediments identified chlordane, diazinon and DDT derivatives with concentrations above the laboratory quantification limit. Of these, chlordane was measured above the probably toxic level and DDE was detected in the possibly toxic range (FDEP 2003).

Conclusions

- Each of the side bank filters is discharging water at about the same rate. One has been installed for seven years and the other for about two years.
- The filters flow continuously and are probably discharging ground water
- Dissolved nutrients are measured at significantly higher concentrations between storm events when the level of the pond is below the intake pipes.
- A water budget indicates about ten percent more water flows out of the pond than flows in during rain events.
- Bypass discharge water during storm events failed to meet state of Florida Water Quality Standards for zinc and lead in one fourth of the samples tested.
- Calculations for pollutant loads indicate the filter system is not effective for removing dissolved nutrients.
- Sediment samples show a significant increase in nitrogen and phosphorus during the final year and this was likely caused by die back of floating duckweed and associated species.
- Polycyclic aromatic hydrocarbons are increasing in the surface sediments
- Chlordane and DDE were detected in measurable quantities in the sediments.

Acknowledgements

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Conversion of an Urban Pond to a Water Quality Treatment Pond

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Abstract

An aging urban pond in Charlotte, North Carolina was monitored for a period of one year prior to a water quality enhancement project for the pond and nearby drainage network. The 11 ha (27 ac) watershed for the 0.243ha (0.6 ac) pond consisted of mixed residential and commercial uses. Inflow and outflow flow composite samples were collected from August 2003 thru July 2004. Samples from 17 storms were analyzed for a suite of pollutants and event mean concentrations (EMC's) were determined for each storm. The pond was enhanced by the addition of a forebay, a detention component and a littoral shelf around its edge. In addition a number of drainage improvements were constructed within the watershed to remove failing conveyances. The detention component of the pond was designed to store the runoff associated with a rainfall of 2.54 cm (1 in) and hold it for a period of up to 24 hours. Water Quality results for the post construction period are not available at the time of this writing.

During the pre-construction monitoring period, mean Total Kjeldahl Nitrogen (TKN) and Ammonium-Nitrogen (NH_4-N) concentrations at the outlet were 32% and 19% higher than the inlet concentrations respectively. Mean Nitrate Nitrogen (NO_3-N) and Total Nitrogen (TN) concentrations were shown to be 77% and 20% lower at the outlet. 16% and 63% reductions in Total Phosphorous (TP) and Total Suspended Sediments (TSS) concentrations respectively were observed. Copper, Lead and Zinc concentrations were reduced by 71, 39 and 49 % respectively. Statistical significance between the observed

inlet and outlet mean concentrations was observed for all pollutants excepting TP and NH_4-N.

Introduction

Small urban ponds are a common feature in many urbanized areas. Such ponds may exist for a number of reasons. Often they are rural ponds which are left during development of the nearby areas or newly constructed ponds which are installed as water features. Where stormwater regulations require control of sediment, wet ponds are often constructed for that purpose. In North Carolina properly designed wet ponds are an accepted BMP for the removal of sediment and are assumed to remove 85% of TSS (NCDENR, 1999). The primary pollutant removal mechanism for ponds is settling and adherence of pollutants to pond bottom sediments. Sediment accumulation within the pond bottom may reduce the capacity of the pond. Drainage systems may fail structurally providing for increased bank erosion and downstream sediment load. Such ponds provide much promise for stormwater BMP retrofit sites. Many improvements can be implemented on a pond which may improve the pollutant removal efficiency. The addition of forebays, littoral shelfs, and detention may provide a several mechanisms for pollutant reduction. Such mechanisms are well accepted components of recently developed BMPs such as wetlands and extended wet detention.

This project investigates the performance of an existing poorly maintained urban pond and the subsequent performance of the same pond after an extensive water quality improvement project. The pond was monitored for inclusion in the ASCE Urban Stormwater BMP database(EPA, 2002).

Study Site

This research was conducted at Shade Valley pond, an urban pond located in fully developed watershed in Charlotte, North Carolina. Constructed during the 1950's, as a water feature for a nearby multi-family housing development, Shade Valley pond sits just upstream of shade valley road. The area immediately surrounding the pond consists of an apartment complex with associated parking areas. An 11ha (27.3 ac) watershed consisting of a mix of commercial, residential and transportation areas feeds the pond via a small perennial stream. Impervious area within the watershed is nearly 86 %. Much of the watershed has connected impervious areas which quickly route runoff into conveyance structures. Prior to the summer of 2004 the condition of the pond was very poor. Due to mowing of the adjacent vegetation up to the waters edge and intense waterfowl activity, the banks of the pond were rapidly eroding. Conveyance structures at the pond edge had collapsed and were continuing to erode. Sediment deposition at the main inlet of the pond had created an exposed sand bar which nearly encircled the inlet. Fecal matter and feathers was prevalent in the adjacent areas and in the pond itself. Conditions during this time are shown in Figures 1a and 1b.


Figure 1a and 1b Condition of Pond During Pre-construction Monitoring Period.

Runoff entered the pond thru a number of poorly maintained conveyances such as culverts, and concrete channel conveyances. Approximately 78% of the contributing watershed entered the pond thru three existing culvert which discharged into a scour pool on the opposite side from the outlet. Shade valley pond was approximately .243 ha (.6 ac) in area with an average depth of 1 m (3 ft) the banks of the pond were highly eroded due to the intense waterfowl activity of the area. The outlet of the pond consisted of an undersized 76 cm (30 in) Round Concrete Pipe (RCP) culvert under shade valley road which discharged into a nearby perennial stream. A simple 1.8 m (6 ft) wooden weir maintained the level of water within the pond.

The City of Charlotte began a construction project in the summer of 2004 to modify the existing pond with the purpose of water quality improvement. The pond was drained and dredged to remove bottom sediments and increase average pond depth. The undersized outlet was replaced with a larger outlet system. The inlets were combined, where possible, and the failed conveyances replaced. In addition to general drainage system improvements, several features were incorporated into the new pond design. These features included a forebay, a littoral shelf, and a detention function in the new outlet.

The forebay was excavated at the inlet of the pond to provide for storage of heavy sediments originating in the watershed and to facilitate the removal of such sediment during maintenance operations. In addition a littoral shelf was constructed along the edge of the pond and spanning an area between the forebay and main pond body.

The littoral shelf was designed so that during periods of normal pool the water level at the shelf would be from 0-30 cm deep (0-1 ft). Emergent aquatic vegetation was planted in the shelf. The littoral shelf of the new pond composed nearly 30 % of the surface area of the pond. The banks of the pond were planted with brushy vegetation so that waterfowl use would be discouraged. The pond outlet was replaced as a result of the drainage improvements which required replacement of the 76 cm (30 in)RCP under Shade Valley road. A cast in place riser was constructed to act as the principle and overflow riser. An orifice was utilized as the low flow and drawdown control device. An overflow weir was constructed approximately 18” above the orifice so that the new pond would provide detention for the runoff associated with the first 1” of any rainfall event. The elevation of the overflow weir was set so that the storage volume within the pond between the drawdown orifice and the overflow weir corresponded with the runoff associated with a 1" rainfall event. The orifice was sized so that the water level within the pond would return to pre event level within 24 hours of the end of the runoff event. Construction activities were completed in the winter of 2005. Planting of emergent vegetation has not been completed at the time of this writing and is planned for the spring of 2005.

Material and Methods

Beginning in August 2003 event based flow composite water quality samples were collected at the inflow and outflow of Shade Valley pond. This monitoring was conducted in order to characterize the pre-existing conditions and performance of the pond. The existing 1.8 meter (6 ft) rectangular weir did not accommodate an accurate means to measure flow at the outlet. Instead, a 120° V-notch weir was attached to the existing wooden outlet weir. The invert of the V-notch was installed at the pre-existing normal pool depth so that no alteration of pond level occurred.

Figure 14a and 2b. Locations of inlet and outlet sample collection

Any detention which occurred within the pond during storm events was determined to be minor compared to the overall runoff from the event. Inlet and outlet sampling locations were outfitted with ISCO 6712 samplers for flow monitoring and sample collection. Location of Inlet and outlet sample collection is shown in Figure . An ISCO model 750 bubbler module was outfitted with the outlet sampler for low monitoring purposes. In addition, an ISCO tipping bucket rain gage was installed on the outlet sampler to allow continuous measurement of rainfall depth and intensity during sampling events.

The intake for the inlet sampler was installed just downstream of the convergence of the three major RCP culverts in an area of well mixed flow. Accurate direct measurement of inflow was not possible due to multiple inlets entering the pond and submerged conditions at those inlets. Since the pond had no appreciable detention component to its operation, if could be considered a flow thru device. As a result, it could be assumed that the inflow matched hydrologically the outflow. An acceptable system of sample collection was devised using a wireless transmitter and receiver. The wireless signal system consisted of a transmitter which was fitted to the outlet sampler. The transmitter monitored the sampler communication port for an output signal which was sent when the sampler initiated sample aliquot collection. Upon receiving the signal the transmitter sent a wireless signal to the receiver unit which was fitted to the inlet sampler. The receiving unit signaled the inlet sampler to collect an individual aliquot. Using this wireless system the inlet sampler collected a sampler at the same time as the outlet sample. The wireless system was constructed and installed by Custom Controls Inc.

Each sampler was outfitted with an array of 24 1L bottles for a total sample volume of 24L. The outlet sampler was programmed to collect an individual aliquot of 200ML for each 1000L of outflow from the pond. Each sampler could accommodate up to 120 aliquots during a runoff event. Typical sample aliquot distribution throughout an example storm hydrograph is shown in Figure 3.

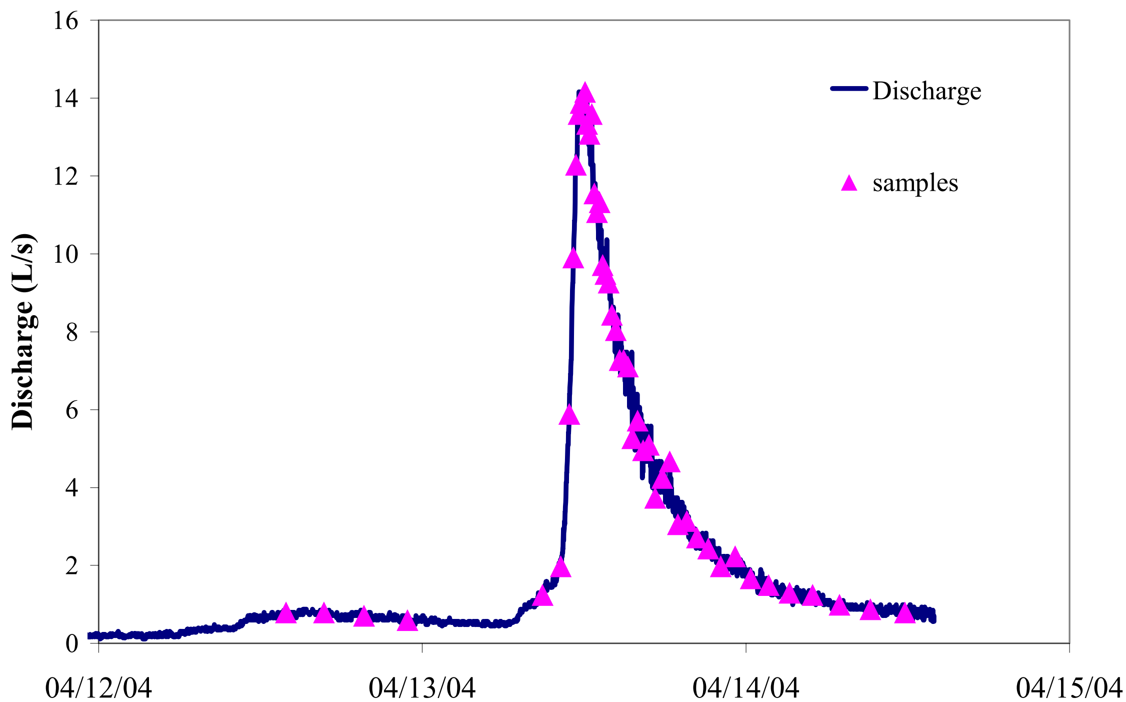

Figure 3 typical hydrograph and associated sample collection times for a 1.75 cm (.7 in) rainfall event

Water samples were collected within 48 hours of the end of a runoff event and delivered to the laboratory for chemical analysis. All Samples were analyzed by the Mecklenburg County Water Quality Program Analysis laboratory. Samples were analyzed for a series of pollutants including nutrients, heavy metals, and sediments. As a result of these analysis an Event Mean Concentration (EMC) was computed for the inflow and outflow of each event monitored for each pollutant analyzed. In addition continuous flow data was collected by the sampling equipment so that storm hydrographs of the outlet could be produced. Completion of the rehabilitation of the pond was not completed in time for post-construction monitoring results to be included in this paper.

Results

17 rainfall events producing runoff were monitored during the period beginning in August of 2003 and ending in July 2004. This period represents the pre-construction period of study for the improvements to shade valley pond. Event depth ranged from .15 cm (.09 in) to 6.9 cm (2.7 in) in depth. Effort was made to distribute the sampling events throughout the year. A minimum of one and a maximum of two runoff events were monitored in every calendar month of the year. Figure shows the distribution of sampled events throughout the year and their associated rainfall depths.

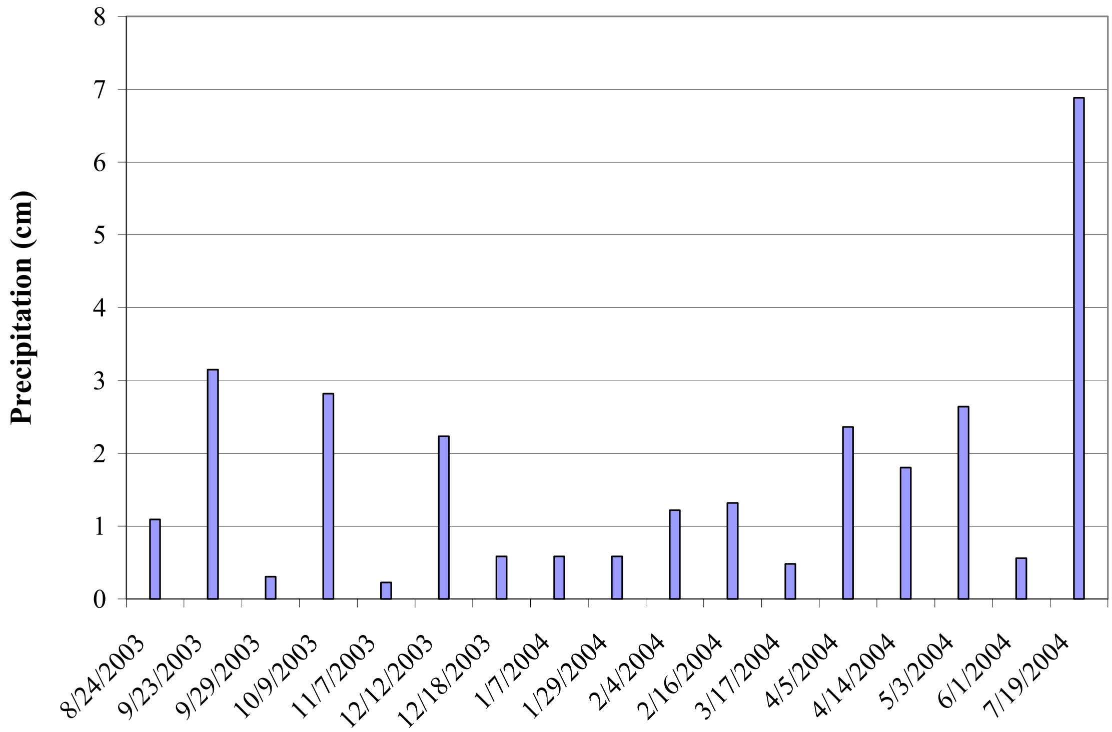

Figure 4 Distribution of monitored events

Mean EMC for the inlet and outlet sample location were computed for each pollutant. Efficiency ratios for the pond were computed from the mean EMC's (EPA 2002). Statistical significance of the inlet and outlet mean concentrations was computed using a paired t-test (SAS, 2003) Results of these methods for Nitrogen constituents are shown in Table 10. Concentrations of TN for the period of monitoring were slightly lower at the outlet than the inlet. NH_4-N

Table 10 Mean EMCs and statistical significance of Nitrogen Pollutants

Pollutant	Inflow (ppm)	Outflow (ppm)	% reduction	p-value	Significant
TKN	1.54	2.03	-31.82	.0262	Yes (increase)
NH_4-N	0.27	0.22	-18.52	.2992	no
NO_3-N	1.37	0.32	76.64	<.0001	yes
TN	2.98	2.40	19.46	.0414	yes

Mean concentrations of TKN and NH_4-N both showed a net increase between the inflow and outflow for the monitoring period. NO3-N showed a substantial decrease in mean concentration. Observed reduction in NO_3-N concentration for each storm exceeded 60%. Total Nitrogen was shown to be reduced by a mean of 15%. **Table 2** lists the results of the analysis methods for TP, TSS and Metals.

Table 2 Mean EMC and statistical significance for TP, TSS and Metals

Pollutant	Inflow (ppm)	Outflow (ppm)	% reduction	p-value	significance
TP	0.19	0.16	15.79	.1159	no
TSS	109.18	40.29	63.10	.0188	yes
Cu	13.17	3.76	71.45	.0131	yes
Pb	5.08	3.12	38.58	.0207	yes
Zn	70.35	35.59	49.41	.0070	yes

Analysis showed many samples did not meet the minimum detectable limit (MDL) for metals analysis. In particular Pb and Zn concentrations in outflow event samples were often below the detection limit. In this case the concentration was set to ½ the value of the MDL for calculation purposes. Reduction in Mean TSS concentrations was shown to be 63% which agrees well with published removal efficiencies for wet pond designed for sediment removal.

Discussion

Reduction of pollutant concentrations in the Shade Valley pond prior to water quality improvement project indicated that the existing pond does have an overall net positive impact on stormwater runoff originating in the watershed. However increased concentrations of TKN, NH_4-N and TP indicate that the status of the pond in its degraded state may have a negative impact on the downstream water quality for those pollutants in particular. Considering the advanced age of the pond it is likely that binding sites for phosphorous within the soil at the bottom of the pond have been filled and that during storm events phosphorous is re-suspended and discharges thru the outlet. Alternatively the existence of a resident population of waterfowl may explain the increased concentrations of organic-N, NH_4-N and Phosphorous pollutants which may be entering the pond thru fecal deposition rather than stormwater runoff.

Future study of the performance of the improved pond will allow better conclusions to be drawn concerning the causes of the results discussed above.

Acknowledgements

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The Dynamics of Effective Stormwater Management

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Abstract

Throughout the various stages of construction, the stormwater plan and associated best management practices (BMP's) must evolve to meet changing conditions. This paper deals with a project in southern Polk County, Florida involving the excavation and construction of an articulated concrete block lined swale and removal of a temporary cofferdam. The contractor used a full suite of turbidity control measures to prevent sediment from flowing into the adjacent lake and subsequently into the Peace River. A review of the techniques used at different stages of the project to prevent turbidity from leaving the site will be discussed. Also, suggestions for additional methods that could be used for maintaining a balance of cost effective actions while adhering to pollution control laws are presented.

This project was a constant struggle to prevent turbid water from entering the adjacent Peace River. Stormwater runoff from 688 hectares (1700 acres) reports to this structure. The Environmental Resource Permit called for a maximum/minimum discharge capacity of the 100-year/24-hour storm. The discharged water emptied into an 8.1 hectares (20 acre) lake and discharged through a rip-rap lined swale, into an intermediate reclaimed wetland system and ultimately flowed into the Peace River, north of Fort Meade, Florida.

The construction plan called for the contractor to initially excavate 15,300 cubic meters (20,000 cubic yards) of material, stockpile and protect it until construction was completed. The stockpiled material was to be used to fill the adjacent temporary swale, which had been used to accommodate the runoff until the project was complete. During the construction period, (May till October, 2004) approximately 122 centimeters (48 inches) of rain fell and the site experienced the full forces of Hurricane Charley, Hurricane Frances and Hurricane Jeanne. One rain event produced 12.5 centimeters (4.9 inches) of rain in 2.5 hours.

Introduction

This presentation deals with the changing conditions of and at a construction site and how this will dictate the management of stormwater throughout the life of the project. This was a recently completed stormwater control project in southern Polk County, Florida and will be used as an example of the need to adjust Best Management Practices (BMPs) as construction progresses and weather conditions change. The project in question, known as Pembroke 8, involved replacing a temporary earthen water control structure with a permanent articulated concrete block lined structure. The existing temporary structure would then be abandoned once the new structure was operational.

The Pembroke 8 temporary structure consisted of an earthen cofferdam with three metal pipes through the dam and a 152-meter (500 foot) long native stone rubble lined ditch. The makeup water for the dam was runoff from approximately 688 upstream hectares (1700 acres). This water reported to a 9.7 hectares (24 acre) pond, which was an abandoned phosphate mine site. The bottom of the flooded area, behind the dam, was lined with waste phosphatic clays. The cofferdam (elev. 32 meters) prevented the release of this water and the clays. The system discharged, through the ditch, to an 8.1-hectare (20 acre) lake, known as Section 23 Lake (elev. 26 meters) and in turn discharged into a second shallow lake, Lake Ann West, which is part of the Peace River flood plane.

The proposal was to excavate a new ditch and build the new water conveyance adjacent to the existing one. Once this was completed, water would be diverted through it, the existing cofferdam would be abandoned and the 15,300 cubic meters (20,000 cubic yards) of newly excavated spoil material would be used to fill the old ditch. In the design, the engineer of record, the consulting firm of BCI Engineers and Scientists, Inc. (BCI), incorporated a storm water pollution prevention plan (PPP). This included floating turbidity barriers or booms up stream of the excavation and in Section 23 Lake. In addition, staked filter barriers were called for at the base of slopes or lower elevations to control any surface runoff. All BMPs were to meet applicable standards. This plan was part of the Environmental Resource Permit (ERP) application, which was reviewed and approved by the Department of Environmental Protection (DEP).

The Work

Construction began in late May of 2004 with the work on the new swale. The plan called for 15,300 cubic meters (20,000 cubic yards) of material to be excavated from the area of the proposed improvement and stocked piled adjacent to the work. A small temporary sediment trap or basin with a berm around it was constructed at the point where the swale would discharge water into Section 23 Lake. This was not part of the original plan but was implemented as the excavation work progressed. It was intended that this effort would keep turbid water from flowing down the newly excavated swale and into Section 23 Lake. Staked turbidity screens were also placed outside the active work site to interrupt sheet flow onto and off the disturbed areas.

With the beginning of construction, daily turbidity monitoring was initiated. The equipment and procedures used were according to DEPs Standard Operating Procedures for Field Sampling. Measurements were taken at the County Road 640 Peace River Bridge, some distance upstream from the point where Lake Ann West discharged into

Peace River. This established a background with which to compare water leaving the work site. This second monitoring station was located where water passed from Section 23 Lake into Lake Ann West, downstream of the floating turbidity boom and before entering Peace River. It was referred to as the Pembroke 8 or on-site station.

The first measurable rainfall event during construction occurred on June 3, 2004. This 2.54-centimeter (one-inch) event resulted in an on site reading of 5.3 NTUs. This first event, although not significant, gave a good indication of onsite surface water flow and erosion patterns. As a result, additional silt fencing and hay bales were installed in the newly excavated swale. They were placed perpendicular to flow and extended approximately two feet up the side of the swale. There were also strong indications that the 20,000 cubic yards of stockpiled material was a large contributor to the measured turbidity. Silt fencing was placed in double rows around the stockpile.

As the project advanced, it became clear that BMP's that worked today might not work tomorrow. The advancement of the excavation produced more exposed and erodeable surface area as well as increasing the sloped area. Additional silt fences were used.

On June 10, 2004, 6.1 centimeters (2.4 inches) of rain fell and produced turbidity measured at 23.1 NTUs above background. Up until this point, the contractor had received only verbal instructions about what type of BMP's to use and where they were to be placed. Initially, this seemed to work well but the dynamics of the job called for more details. The day after this most recent rain event, the consultant, BCI, produced an AutoCAD drawing of the site, showing the type as well as the location of BMP products to be used. This was called the "BMP PLAN". The drawing was given to all personnel on site. This resulted in a heighten awareness and understanding the importance of BMP's by everyone working on the job. As needed, this plan was updated during construction. In addition, the prime contractor and all sub-contractors were given printed material illustrating the proper installation of floating booms, hay bales and silt fences. In essence, the consulting engineer implemented a training program that was unique to the site and the changing needs of the project.

During the month of June 2004, 21.1 centimeters (8.3 inches) of rain fell on the work site. With continuing rains and erosion, the basin at the bottom of the new swale was becoming filled with sediment and overwhelmed. The contractor began to clean it on a regular basis. He placed a pump there to increase the capacity. Also the berm around the sediment trap was heighten and armored with a small portion of the concrete mat and filter cloth. The repair and maintenance of the silt fence became a regular task and a small Ditch Witch was permanently moved to the site. There was too much fencing to anchor by hand. Fiber logs (Floc Logs) were also used. These logs contain a flocculating substance and were used where turbid water discharged into Section 23 Lake from the swale. Some powdered flocculants was also used mornings after overnight rain events to minimize turbidity before discharging into Lake Ann West. The contractor began placing the articulated mats on the floor of the new swale in July 2004. One intense rain event undermined the matting and caused a breach in the berm around the holding pond. At this point, maintenance of the BMP's was very time consuming.

In addition to the pump used at the sediment trap, the contractor placed a second, high capacity pump at the cofferdam to aid in the dewatering of the 9.7-hectares (24 acre) pond. This minimized the likelihood of water entering the partially completed swale. On a daily basis, the forecast for rain at the project site was always high. As a result, the contractor kept a man at the site, sometimes around the clock, as needed to maintain the pumps.

The Storms

Hurricane Charlie struck the work site on August 13, 2004. In preparation for the storm, the consultant directed the contractor to inspect and maintain erosion control and sediment containment features on the site. The contractor emphasized grading to minimize rills and other small channels. He also cleaned the sediment trap and dug it deeper. Floc Logs were checked and additional powered flocculent was stored on the site. All silt fences, hay bale barriers and turbidity booms were inspected. And, as an effort to minimize material exposure, the contractor covered the stockpile of excavated earth with plastic sheeting.

Monitoring immediately after the storm showed that no turbidity generated onsite and exceeding 29 Nephelometric Turbidity Units above background was discharged off the site. Actually, it was determined that for nine of the following ten days after the storm the Section 23 Lake turbidity measurements were less than those taken for Peace River. The only drawback to the preparatory work effort was finding and cleaning up the plastic sheeting.

By the first week of September, 90% of the articulating mats were in place and day-to-day erosion problems were very well under control. In spite of 35.5 centimeters (14 inches) of rain in August, work was going well. Unfortunately, strong winds and rains from Hurricane Frances began in the evening hours of September 4, 2004. This slow moving and very wet storm passed within 10.7 kilometers (ten miles) of the site and dropped 12.5 Centimeters (4.9 inches) of rain in 48 hours, as measured by the rain gauge on the site. Needless to say, after August's rains, the ground was saturated. The morning of September 6, 2004, revealed bad erosion at the north and south ends of the site. Erosion repair began immediately and the same steps that were taken after Hurricane Charlie were repeated.

Turbidity measurements were taken that day after the storm and indicated that water leaving the site and into Peace River exceeded the maximum of 29 NTUs above background. The consultant, BCI, had prepared a contingency plan that was immediately initiated. This included placing additional Floc Logs at the cofferdam pipes and at the base of the old and new swales where the water entered Section 23 Lake. Powdered flocculent was added to the flow at crucial points including where water discharged from Section 23 Lake into Lake Ann West. This improved water quality substantially. This work effort continued hourly until the early morning of September 7, 2004.

A review of the data collected on September 6, and 7, 2004, indicate that a maximum turbidity value at the point of discharge was 56.9 NTU or 51.4 NTU over background. At 1:27 am on September 7, 2004, the turbidity was reduced to 25.5 NTU above background. Measurements 12 hours later showed that the turbidity levels were down to 5.81 NTU above background and by the end of the day, that level was 0.5 NTU.

On September 26, 2004, Hurricane Jeanne moved through Polk County and passed approximately 16 kilometers (15 miles) south of the Pembroke 8 site. Prior to the storm, the contractor reinforced all silt fences; check anchors on the floating booms and cleaned sediment traps. In addition, all pumps were serviced and the site was graded to minimize indentions, rills and other conditions that would cause erosion. Floc Logs were in place and powdered flocculants were on stand-by at the site. This storm contained less rain and was faster moving than the previous storms. As a result, it did not impact the work site like the two previous hurricanes. Previously learned lessons were put to use and no turbid water, exceeding standards, was released from the site.

Conclusion

The project was completed in mid October 2004. Summer rains and three hurricanes produced approximately122 centimeters (48 inches) of rain for the months of June, July, August and September at the Pembroke 8 site. The average annual rainfall in Polk County, Florida is125 centimeters (49 inches). The opportunity for the violation of Florida water quality standards were abundant. Only one occurred. This occurrence was beyond the control of the owner, consultant or the contractor. This success was because of diligent planning and implementation of water quality best management practices and a willingness of all involved to adjust the PPP to fit changing conditions. The group learned to anticipate the unexpected and make quick changes. In spite of the one undesirable event the principals involved continued to learn from the experience and did not hesitate to implement that knowledge.

Arc Hydro Enhanced Database for Water Resources Management

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Abstract

The Arc Hydro Enhanced Database (AHED) provides a platform for Water Resources Management in South Florida. Starting with the nationally recognized GIS data model for hydrology—the Arc Hydro Geodatabase and the Arc Hydro Tools for data input and maintenance—the project addresses the needs of watershed management, hydroperiod analysis, operations decision support, and hydrologic and hydraulic modeling. The design and implementation of the data model as an Environmental Systems Research Institute (ESRI®) enterprise geodatabase based on the Arc Hydro framework was a successful collaboration between the public sector, academia, and the consulting world. The results included the invention or refinement of a series of innovative new approaches, database structures, and tools to meet the needs of the South Florida Water Management District. The time-series concepts developed are being carried forward by ESRI to integrate into the national Hydro Data Model (Arc Hydro). The implementation of the enhanced data model and new tools provides a common data structure and geographic platform for multiple different project types to share data about how water is moving through the environment and share results in a regional GIS. AHED forms a unique relational database environment in which the natural connection between lakes, canals, control structures, monitoring points, and drainage basins are represented along with the features themselves. Most importantly, the spatial data are linked to the large quantity of time-series measurements such as rainfall and water level, and to project and model results. Procedures and "hooks" are provided to use the GIS with time-series data to model changing hydrologic conditions for adaptive management.

Introduction

Our natural water resources are a double-edged sword. We need to maintain and develop sources of water to sustain life while protecting a growing population from the ravages that seasonal floodwaters may bring. New tools are making it possible to address the complexities associated with these water resources challenges head on. The Arc Hydro data model is becoming widely accepted as a standard geographic framework for water resources management. This data model forms a foundation linking spatial objects like water bodies, structures, and basins to measurements of levels, flows, fluxes and quality. It acts as an "enabling" technology for efficient and collaborative water resource data analysis.

The South Florida Water Management District (the District) is responsible for managing the water resources of south Florida and the Everglades including water quality, flood control, natural systems, and water supply. Recognizing the need for a better means of data integration, and to correct data fragmentation issues, District staff chose to fast-track development of a prototype integrated water resources geographic information system (GIS). Using the Arc Hydro framework, real-world data from key flood control, natural system restoration, operations decision support, and regional modeling projects were used to populate a prototype enterprise database–the Arc Hydro Enhanced Database (AHED).

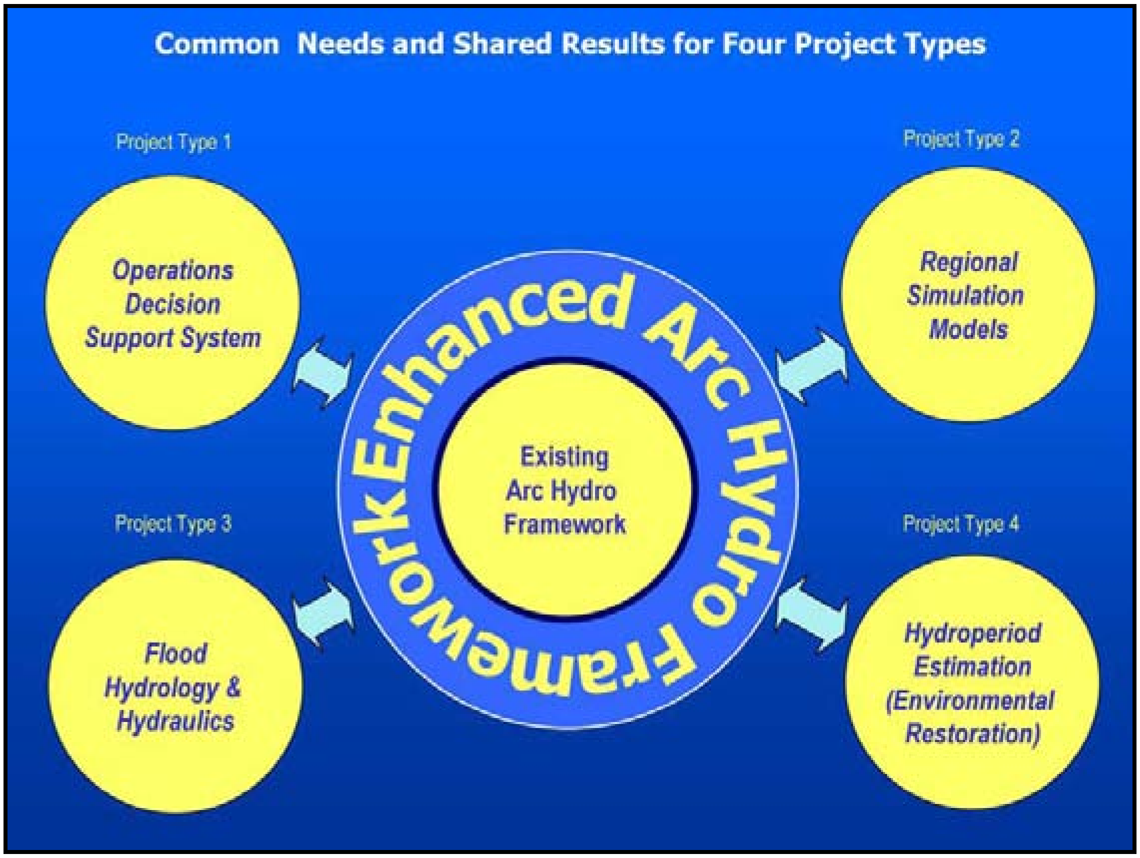

Figure 1. Expansion of the existing ESRI Arc Hydro™ data model embraced common needs of four key district project types.

Working as a team, PBS&J, District staff, and Dr. David Maidment and students of the Center for Research in Water Resources (CRWR), the University of Texas at Austin, developed AHED to address common data needs, common functional needs, data sharing opportunities, and decision support. PBS&J's role was to coordinate the participants for development of the conceptual system framework, translate the conceptual framework into an ESRI geodatabase design, populate it with sample data, implement the result at the District, and translate prototype tools into fully functional products. The geodatabase and a toolbox for loading, managing, and using the data were executed in the GIS under ArcGIS Version 9.0, taking advantage of Model Builder, and using ArcSDE Version 9.0

with Oracle 9i. On a parallel track, applications were developed to use the AHED platform for analysis of water levels in a prototype data set from the Kissimmee River Restoration. The tools permit interpolation of hydroperiod statistics for individual habitats. The GIS tools were developed in .NET to help build, maintain, and utilize the AHED as a shared resource at the District. Accompanying project deliverables included a logical design manual, data dictionary, user manual, and installation manual.

Due to the highly collaborative nature of this project, District staff and the CRWR director and staff are all partners in the results and deserving of credit. Staffs from various departments within the District including operations, regional modeling, environmental restoration, flood modeling, and information technology were active participants in the process. An initial two-day seminar seeking consensus on the common needs was followed by continuous review and input as part of the iterative development of the final products. Dr. David Maidment and students contributed heavily to the conceptual framework and development of "proof-of-concept" examples. Dr. Maidment was able to use this opportunity to consolidate his ideas on integrating time-series data into GIS.

Study Data

The initial AHED design focused on the common and unique needs of four specific projects at the District using the enterprise GIS as a unifying framework. The goal of developing an enhanced Arc Hydro framework for the District was to extend the existing Arc Hydro geodatabase model to account for the unique environments of south Florida, including its extensive network of canals, control structures, and the multiple system-variable outfalls for each watershed, and to link the activities of four business units or "project types" represented as:

- Hydroperiod Analysis for Restoration
- Operations Decision Support
- Detailed Hydrologic and Hydraulic Models for Flood Mitigation
- Regional Simulation Model

The prototype database used sample data from all four projects to design and test the data model. Figure 2 shows the location of the sample data sets. The core datasets that provided the core base for the Arc Hydro framework were the Primary Canal System from Operations (Lake Kissimmee south to Lake Okeechobee) and an embedded detailed area representing the Kissimee River Restoration project and 21 water surface elevation monitoring points.

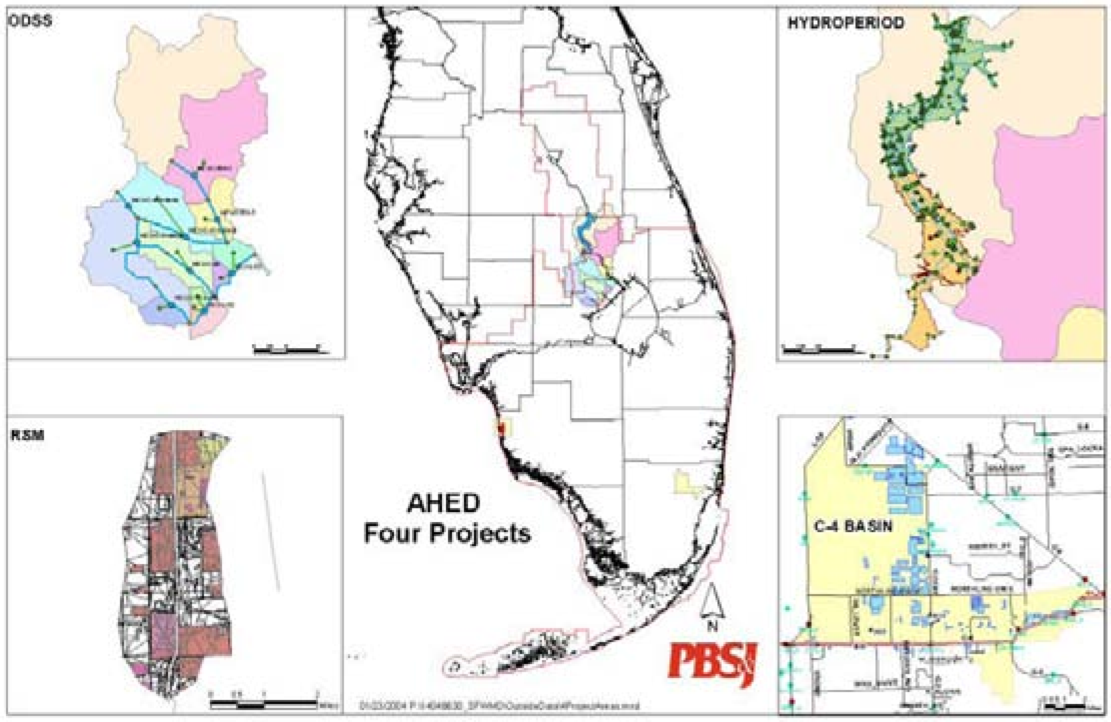

Figure 2. Location of prototype datasets within the South Florida Water Management District.

Methods

The underlying design approach was based on all four projects sharing a common need for timely and accurate data about the hydrologic and hydraulic systems of south Florida; having overlapping needs that can be met through a common data structure; and feeding results into and utilizing results from a common data structure for displaying these results. The design provided a centralized index into detailed project data; a relational data structure for displaying the natural connections between canals, structures, water bodies, drainage areas, and monitoring points; "hooks" into time-series data (model results and gauge readings) for quality control; decision support and adaptive management; and a platform from which to calculate water balance across the district.

The Arc Hydro design process involved an initial conceptual design phase, followed by an iterative design and training process in which feedback was received from both project participants and data stewards for the District's fundamental data layers. A key component in the review process involved bringing all the participants up to speed on the underlying fundamental concepts and decisions behind the design of the core Arc Hydro data model. To explain the enhanced Arc Hydro framework, it was useful to examine how the current Arc Hydro framework model was developed by CRWR. A great many ideas were suggested as to what feature classes (GIS layers) and related components should be used to form part of the model. Early on, it was clear that these should be divided into areas or components, the most fundamental of which was a geographic framework and a time-series component. The time-series component serves to store the hydrologic information measured or modeled on the geographic framework.

It became apparent during the design process that it would be desirable to have a basic core dataset that all applications could use. This was derived from the Arc Hydro framework data model, which consisted of just six basic feature classes stored in a single feature dataset: **HydroEdge**–a network "edge" feature class representing flowlines through streams, canals and water bodies, which is the linear feature class of a geometric network called the HydroNetwork; **HydroJunction**–a simple junction feature class representing important points linking other features to the HydroNetwork; **MonitoringPoint**–a point feature class representing gages and sampling points for hydrologic information; **Structures**–point locations of water control structures such as gates, locks, weirs and culverts; **Watershed**–a polygon feature class representing drainage areas; **Waterbody**–a polygon feature class representing water bodies. It became obvious in working with implementations of Arc Hydro that this simplification of the complete model to a core framework formed a standard point of departure for further customization to fit particular circumstances within an organization. The relationships between these core elements, and special subtypes to help describe the South Florida environment were sketched in a "Conceptual Unified Modeling Language(UML)" diagram, simplified from full UML in order to be easy for project participants to understand and review.

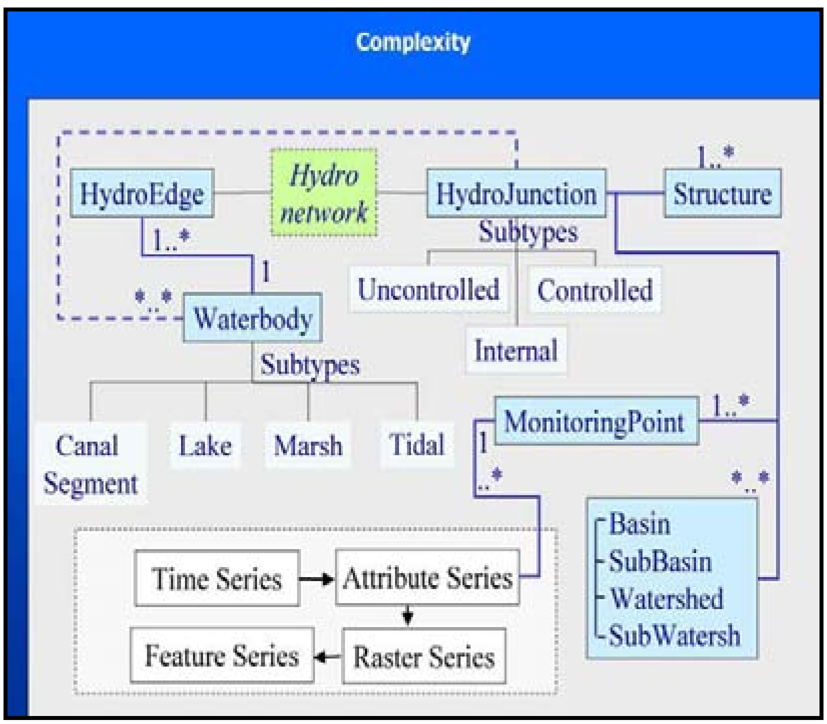

Figure 3. The Arc Hydro framework was enhanced to include many-to-many relationships that account for looped networks, and integrated with an advanced time-series data model.

After the initial conceptual design document was submitted and approved, the conceptual UML was converted to full Unified Model Language in Visio with each feature class fully enumerated in regard to attributes, attribute "domains" (acceptable values) and relationships. ESRI permits properly formed VisioUML diagrams to be exported to a geodatabase ready to be populated with data. The UML was exported to a geodatabase, and populated with sample data from the project areas. These data were collected from the existing GIS coverages of structures and monitoring points held by the district, along with flowlines and waterbodies derived from the National Hydrography Dataset 1:100K NHDinGeo format.

An intial review by selected District staff for features and attributes was followed by a second review for relationships between features. In order for the District staff to review and understand the database, initial training was provided in using ArcGIS 9.0 and geodatabases.

During the initial review periods, work began on developing tools to automate translation of NHD data into the geodatabase and building the natural relationships defined between the elements (this waterbody falls in this watershed and is bounded by these structures).

Following the second review NHDinGeo detailed 1:24K data became available in the prototype area, and the tools for importing data and building relationships were used to recreate the centerlines and waterbodies using the detailed dataset.

As District staff became more familiar with the database design and how it is used, the tools for performing hydroperiod analysis were introduced. A prototype of the Hydroperiod tools were provided by Jenifer Sorenson as a part of her Masters thesis research under Dr. Maidment at UT Austin (See references). PBS&J converted the prototype tools into a production application with a control panel and help files in cooperation with District scientists working on the Kissimee River Restoration, integrating a Time Series management application provided by Danish Hydraulic Institute (DHI). The tools use Arc GIS 9.0 Model Builder to give scientists a flexible system of interpolating water surface elevation readings from multiple monitoring points into a series of daily ponded depth rasters. The tools then distribute the raster ponded depth cells across habitat polygons and create average, minimum, maximum and median ponded depth curves within the habitat in order to analyze hydroperiod restoration withing specific habitats.

Results and Conclusions

An important contribution of this project was the definition of new subtypes and relationships between these features to support the unique environment of south Florida. The new relationships allowed for the looped network of canals and structures in south Florida, accomodating multiple inlets and outlets with water levels monitored on both the headwater and tailwater sides of each structure.

The AHED, as developed, is expected to be a living design, intended to grow and evolve with use. To accommodate this growth, a flexible system for additional features was built around project types. Links to detailed project data were introduced to provide access to project detail without overloading the regional framework. Modifications and enhancements will continue as new insights are gained during implementation and use. The design is flexible and expandable so that new project types can be added as well as enhancements made to existing features.

A unique relational database environment was also formed in which the natural connection between lakes, canals, control structures, monitoring points, and drainage basins were represented along with the features themselves. Most importantly and most technically complex, the spatial data are now related to the large quantity of time-series data stored, and procedures and "hooks" are provided to use the GIS with time-series data for hydrology, heads and flows, system state, system projections, water balancing, and adaptive management.

New concepts integrating time series with GIS grew out of the project. Dr. David Maidment expanded the basic Time Series table in Arc Hydro to include: Attribute Series- time series linked to the unique Hydro ID of a fixed individual features in the GIS, such as a monitoring point; Raster Series – stacks of time-indexed spatial grids representing changing values through time on a surface; and Feature Series- representing GIS features that move or change through time, such as ponded depth contours. These features are linked by the ability to interpolate Attribute Series to create surfaces in a Raster Series, and by the ability to classify Raster Series to generate the changing contours in an Attribute Series. These concepts are illustrated in Figure 4.

The Time Series concepts illustrated in Figure 4 were incorporated into the Hydroperiod tools to interpolate Attribute series representing water surface monitoring points to create ponded depth rasters and ponded depth contours (feature series). These data generate hydroperiod statistics for individual habitats in the study area. The initial results of this analysis for a small dataset are published online (Sorenson and Maidment, 2004). The tools are being currently being utilized by the Kissimmee River Restoration team at SFWMD to extend the analysis to 7 years worth of data. Figure 5 shows the link between habitat polygons and the statistical analyses.

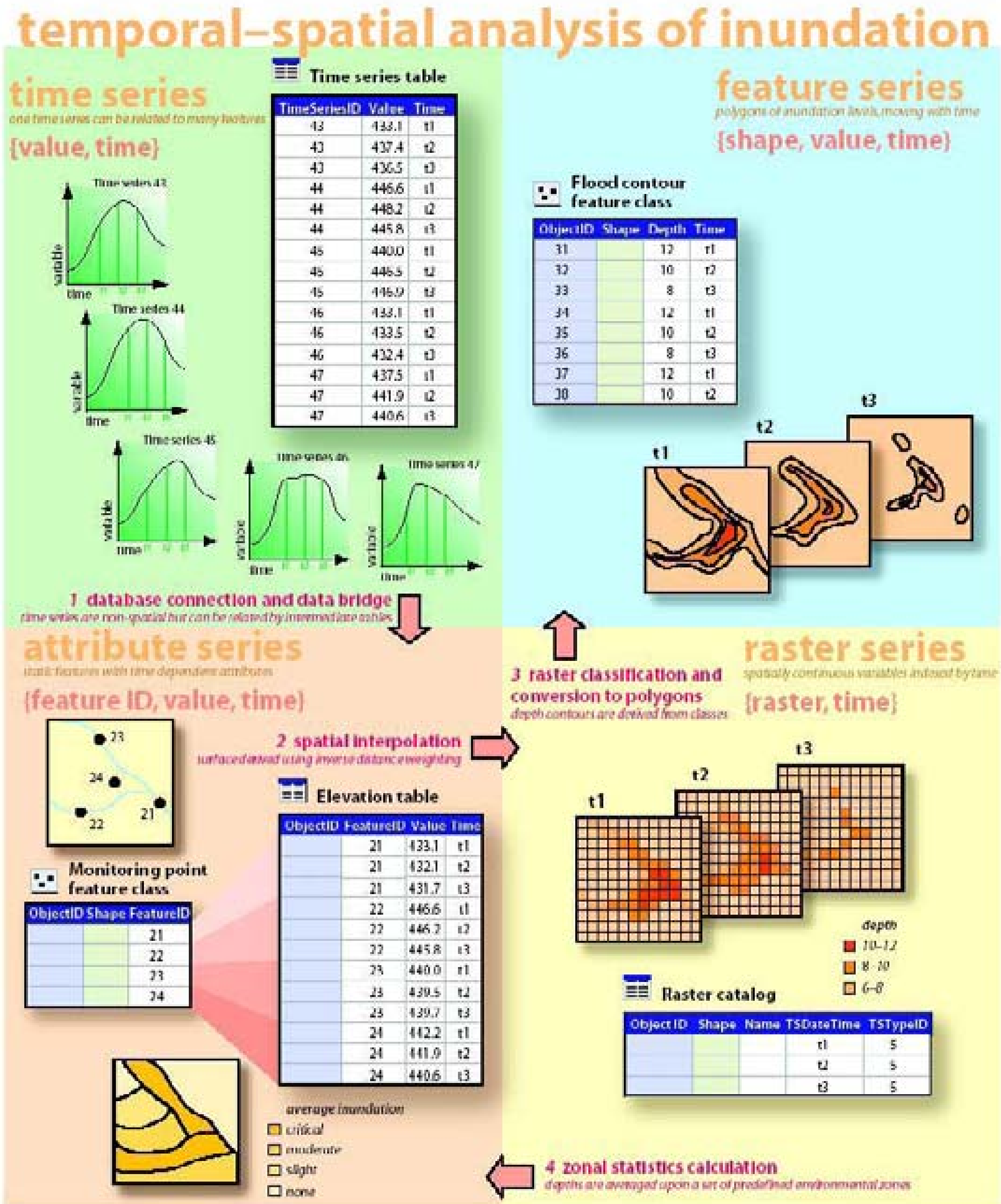

Figure 4. Illustration of relationships between the new Time Series features developed by CRWR for the Arc Hydro Enhanced Database.

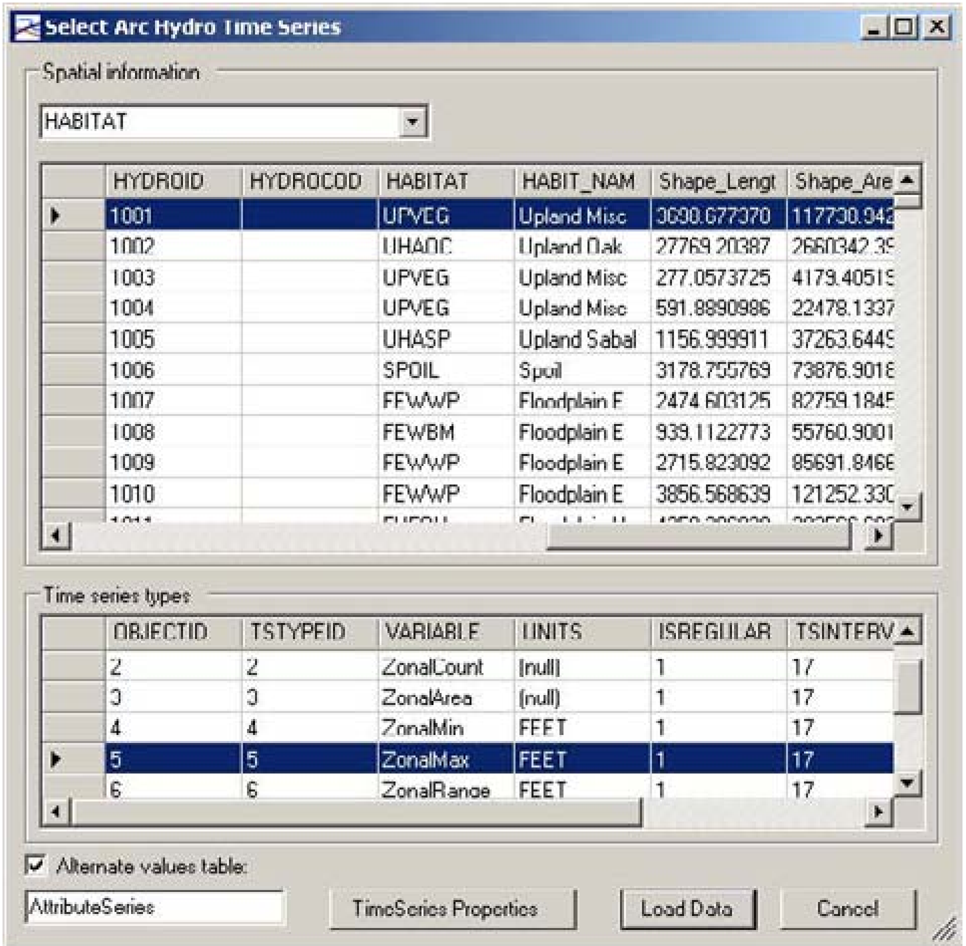

Figure 5. By interpolating water surface elevation measurements, and subtracting ground surface elevations, the Hydroperiod Tools generate Ponded-Depth Time Series showing Daily Values for each habitat polygon of Ponded Depth Area, Minimum, Maximum, Range, Mean and Median.

The real-world application and advancement of the Arc Hydro data model developed during the course of this project has resulted in an enhanced template that can be used for applications ranging from hydrologic data storage to regional data sharing between water control agencies and even for real-time model creation from GIS data. PBS&J is continuing to apply the data model to projects:

- Using the Arc Hydro data model as the basis for an enterprise database to manage flood risk mapping for FEMA in South Florida Water Management District. The ability to view model results from individual watersheds in a regional context provides added benefits for comparison, calibration, and planning.
- Extending the time series concepts developed for Hydroperiod analysis into water balancing equations and tools for day-to-day water management operations and powerful tools for predicting system response to predicted rainfall. The creation of time series objects in the Arc Hydro Environment promises to provide a fundamental tool for integrating past, present, and predicted flows and fluxes into a GIS framework for rapid analysis, comparison, and prediction.

References

Details regarding the Arc Hydro Enhanced Database for South Florida-conceptual design, database design, and tools:
Arc Hydro Enhanced Database, 2004; Logical Design Document (including database description and Data Dictionary); Physical Design Document (including Installation Manual and proposed production configuration); and Tools User Manual. http://www.sfwmd.gov/gis

Technical paper on Hydroperiod Analysis for Kissimmee River Restoration:
Jennifer Kay Sorenson, M.S.E. and David R. Maidment, PhD, 2004; Temporal Geoprocessing for Hydroperiod Analysis of the Kissimmee River;
http://www.crwr.utexas.edu/reports/2004/rpt04-5.shtml

Assessment of Gage-Adjusted Radar Rainfall Data for Use in a Watershed Management Program at the South Florida Water Management District

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Abstract

Rainfall is the primary consideration for many watershed management programs and corresponding operating decisions. While many scientists, engineers, and operators are accustomed to working with rainfall data collected at a few, widely-spaced rainfall gages, gage-adjusted radar rainfall data, made increasingly available via expansive radar and communication networks, may be superior to gage data for some applications. The trade-offs between gage data and gage-adjusted radar rainfall data are well known: gage data give precise information about very small, discrete regions (e.g., incremental or cumulative rainfall over an area measuring a few square centimeters), while gage-adjusted radar rainfall data give more generalized information (e.g., average rainfall over pixels measuring four square kilometers in area), but cover continuous areas as large as—and larger than—any one of Florida's water management districts. The purpose of the present paper is to explore some challenges in attempting to reconcile data from both rain gage and gage-adjusted radar rainfall data sets that cover the same time period and geographic area. The paper represents the status of an effort (in progress) by the South Florida Water Management District to assess the worth of gage-adjusted radar rainfall data that it purchases from a private vendor whose precise algorithm for computing the data is not disclosed. Unlike other studies, in which multiple rain gages are found within a pixel (the region over which gage-adjusted radar rainfall is computed), our problem focuses on situations where there are only one or zero rain gages per pixel.

Introduction

This paper presents the results of several analyses that we conducted in order to characterize bias in gage-adjusted radar rainfall data. The purpose of the present paper is to explore some challenges in attempting to reconcile data from both rain gage and gage-adjusted radar rainfall data sets that cover the same time period and geographic area. In the original study, we examined rainfall data integrated over several time scales. Herein, we present mainly those results associated with annual rainfall, for brevity.

Rainfall is the primary consideration for many watershed management programs and corresponding operating decisions. While many scientists, engineers, and operators are accustomed to working with rainfall data collected at a few, widely-spaced rainfall gages, gage-adjusted radar rainfall data, made increasingly available via expansive radar and communication networks, may be superior to gage data for some applications. The trade-offs between gage data and gage-adjusted radar rainfall data are well known: gage data give precise information about very small, discrete regions (e.g., incremental or cumulative rainfall over an area measuring a few square centimeters), while gage-adjusted radar rainfall data give more generalized information (e.g., average rainfall over pixels measuring four square kilometers in area), but cover continuous areas as large as—and larger than—any one of Florida's water management districts.

OneRain, Inc. provides the South Florida Water Management District (SFWMD) with gage-adjusted radar rainfall data (Hoblit, et al. 2003) that covers the jurisdiction of the SFWMD. The data are provided at 15-minute intervals and have a resolution of approximately 2 km x 2 km. At the end of each month, OneRain examines the monthly data and makes any corrections or adjustments necessary in order to bring the gage-adjusted radar rainfall values into agreement with the areal rainfall amounts that are calculated directly from quality-checked gage data. The precise algorithm that OneRain uses to generate the gage-adjusted radar rainfall data is not disclosed.

Unlike other studies (e.g., Brandes 1975, Ciach and Krajewski 1999) in which multiple rain gages are found within a pixel (the region over which gage-adjusted radar rainfall is computed), our problem focuses on situations where there are only one or zero rain gages per pixel.

For this study, we used data from three sources. Rain gage data came from the SFWMD's DBHYDRO and DCVP databases. In this study, we limited gage data to that associated with the SFWMD's telemetry network, which enables transmission of rainfall data to SFWMD offices in near real time. The third source is the gage-adjusted radar rainfall data that comes from the "end-of-the-month" (EOM) electronic files provided by the OneRain corporation, which are available from the NRDD database at the SFWMD.

Study Site

The study site is the jurisdiction of the SFWMD. We are particularly interested in rain gage stations and surrounding areas. In Figure 1, we present the rain gage stations with telemetry. The unshaded region is the jurisdiction of the SFWMD.

Materials, Methods, and Results

In this section, we present four methods of observing the properties of rain gage data and gage-adjusted radar rainfall data. All calculations were made on a personal computer using spreadsheet software and a BASIC interpreter.

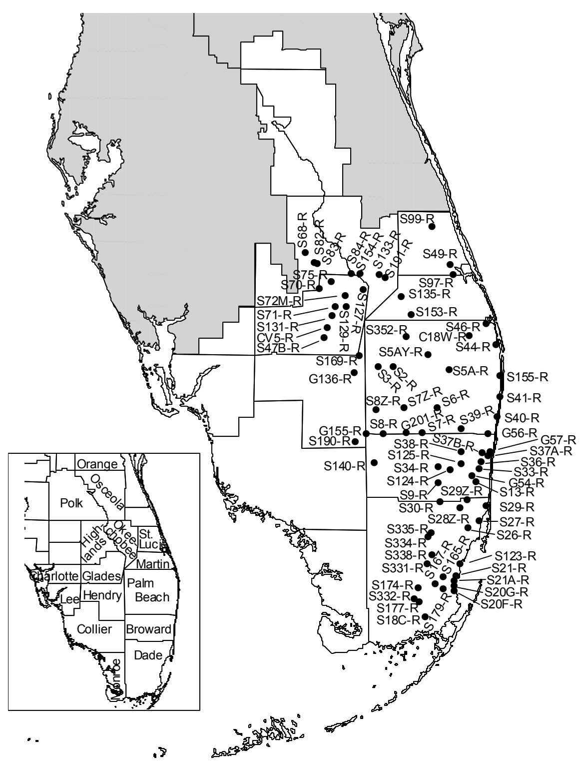

Figure 1: Rain gages in the SFWMD's telemetry network

Relative Behavior of Integrated Data

We examined rain gage and gage-adjusted radar rainfall data corresponding to 12 pixel-gage pairs (i.e., a rainfall gage together with the pixel that covers it), for the month of June 2002. This preliminary examination indicated that bias may or may not accumulate over a given interval. Cumulative rainfall curves for pixel and gage data may converge, diverge, or may be intertwined over this one-month period. If bias accumulates over the period, it may become either more and more positive, or more and more negative.

Next, we examined annual time series for water year 2003 (WY03; May 1, 2002 through April 30, 2003), for the same pixel-gage pairs that we examined for June 2002. Annual time series data for three pairs of the twelve, one of which is shown in Figure 2, were consistent with our preliminary observations of one month of data.

We present another time series in Figure 3, representative of pixel-gage pairs whose corresponding data are divergent. Overall, it seems that bias accumulates gradually, and almost always in the same direction, via the addition of many small differences, rather than large differences associated with a small number of events.

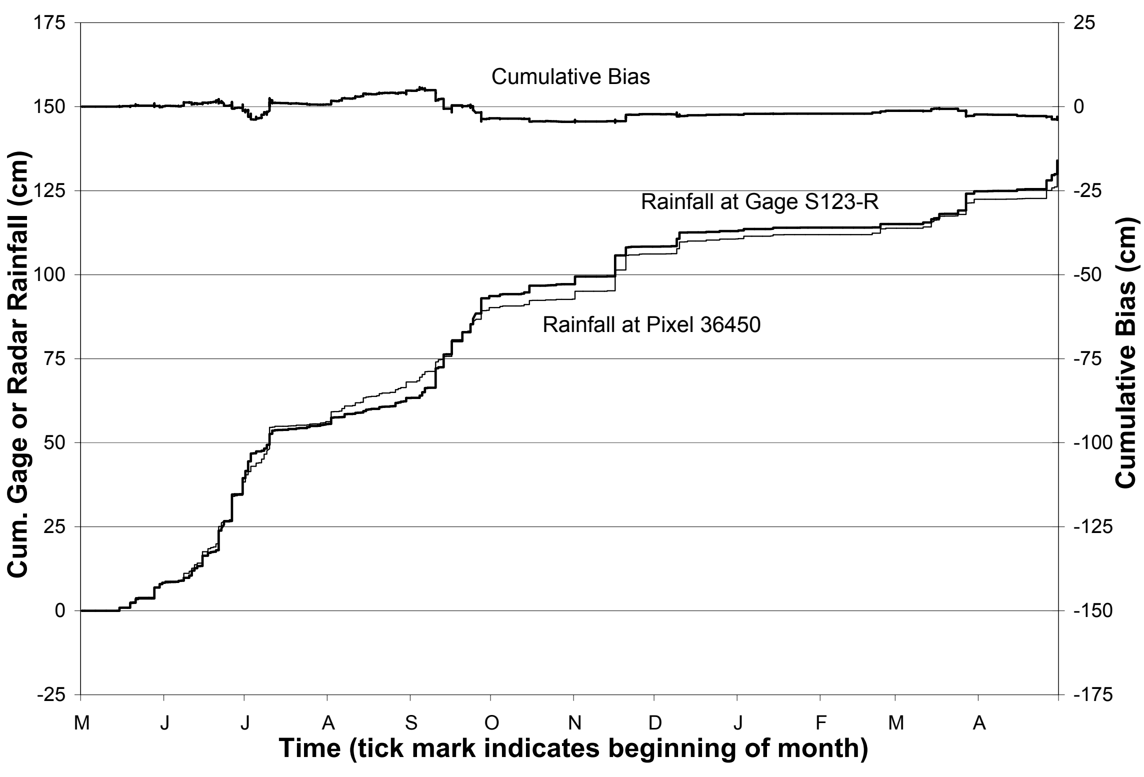

Figure 2: Time series for gage S123-R and Pixel 36450, for WY03

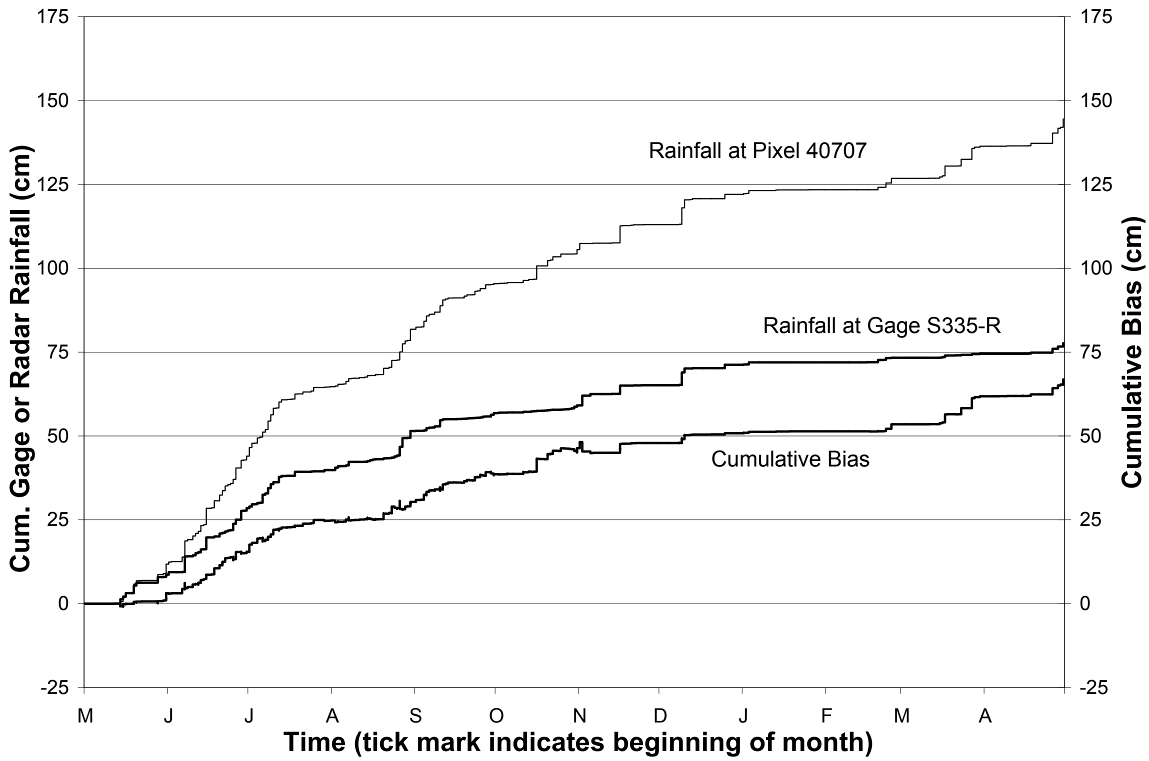

Figure 3: Time series for gage S335-R and Pixel 40707, for WY03

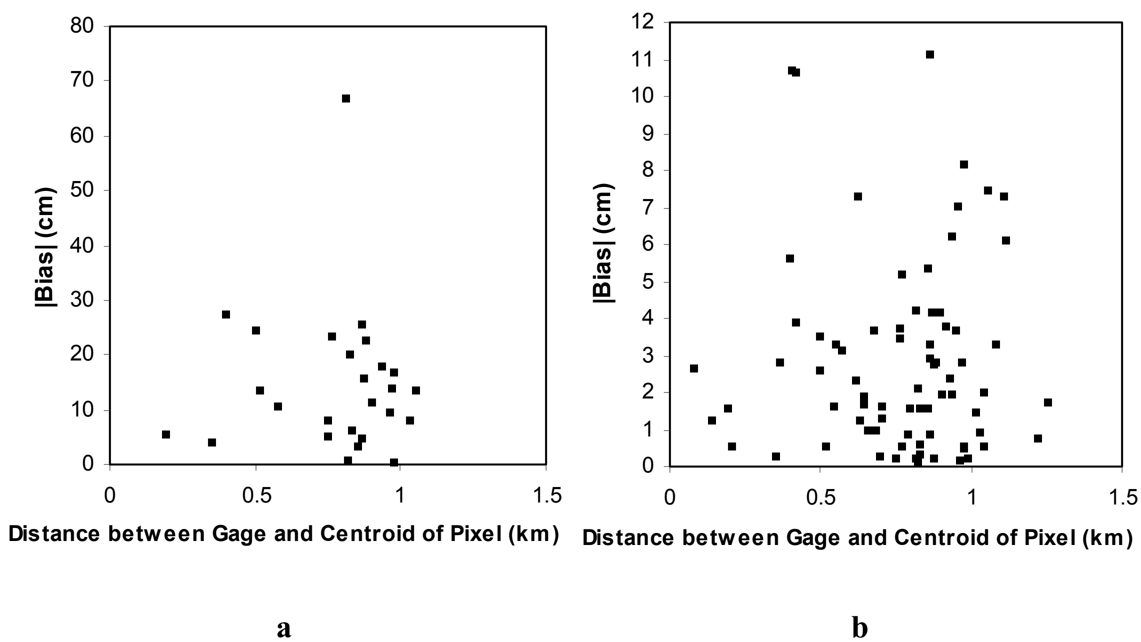

Figure 4: Absolute value of bias versus distance between gage and centroid of pixel a) bias for annual rainfall, b) bias for maximum daily rainfall.

Dependence of Bias upon Gage Position

Two sets of data were examined in order to determine whether the position of a gage within a pixel had any bearing on the bias for that pixel-gage pair. The first data set corresponds to annual rainfall for WY03. Only complete data records were considered; thus, although there are 79 telemetry stations, there were only 26 pixel-gage pairs in the first data set. The second data set consists of the greatest total daily rainfall for each rain gage; 75 data points are available for this second set. Plots of bias magnitude versus distance between the gage and pixel centroid are presented in Figure 4. Neither plot suggests a definite relationship between the two variables.

Supposing that there may be some dependence of bias upon gage position not discernable from the plots, we divided the second data set into two subsets: one subset contains the values of bias for gages within 0.822 km (2700 feet) of the centroid of the pixel, while the other subset contains the remaining values of bias. The mean of the bias values of the "near" subset is 2.845 cm (1.120 in; STD = 2.670 cm = 1.051 in, n = 30), while the mean of the "far" bias values is 2.835 cm (1.116 in; STD = 2.576 cm = 1.014 in, n = 45). A one-tailed, large-sample test of the hypothesis (H_0) that the difference between the means is zero (z = .014) does not result in the rejection of H_0 at the 10% level.

Probability Distribution of Annual Rainfall Bias

We compared total annual rainfalls for WY03 at 26 telemetry stations, for which complete records were available in the SFWMD's DCVP database. Table 1 compares rain gage data to gage-adjusted radar rainfall data, on a station-by-station basis, for WY03. The bias data appear to be uniformly distributed (Figure 5).

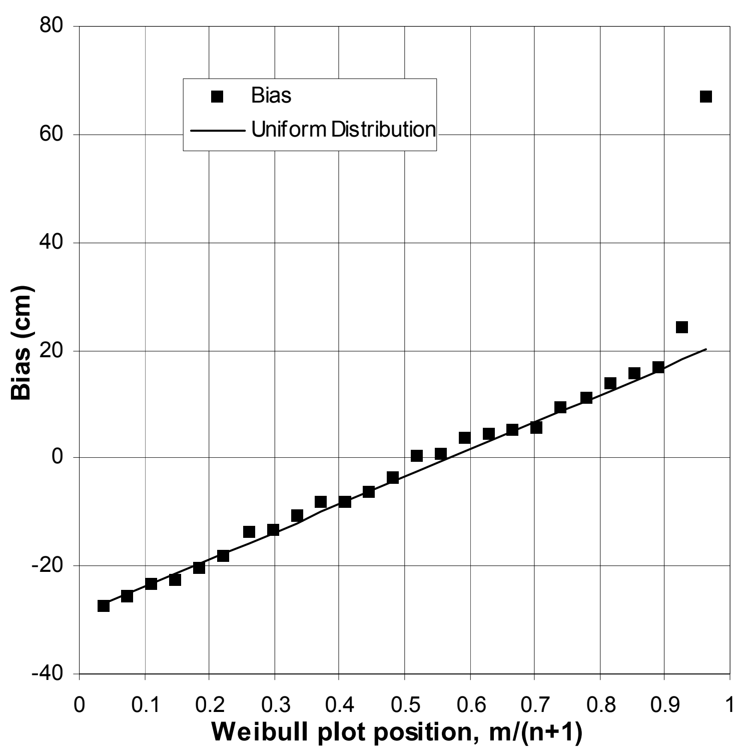

Figure 5: Probability distribution of annual rainfall bias

The largest bias was for station S335-R and pixel 40707 (66.9 cm ≈ 26.3 in). The records for the closest neighbor of S335-R (gage data: 77.7 cm ≈ 30.60 in; gage-adjusted radar rainfall data: 144.7 cm ≈ 56.9 in), S334-R, give an annual rainfall of 158.1 cm (62.26 in; where the 67% of data missing for May and the 30% of data missing for June is taken as zero). The rainfall at this neighboring

Table 1: Comparison of gage and gage-adjusted radar rainfall data for 26 telemetry stations (annual rainfall for WY03; cm)

Station	Pixel	Gage Data	NEXRAD Data	Bias
S123-R	36450	134.0	130.5	-3.4
S125-R	51145	140.4	117.2	-23.2
S131-R	72433	137.3	119.3	-18.0
S135-R	75300	86.8	92.4	5.6
S13-R	48306	124.3	140.1	15.8
S140-R	51117	99.0	102.8	3.8
S153-R	72459	92.1	106.0	14.0
S154-R	78604	78.8	90.2	11.4
S155-R	63482	125.4	111.9	-13.5
S179-R	32653	132.5	119.1	-13.4
S190-R	53955	143.2	120.6	-22.5
S20F-R	32656	127.1	119.1	-8.0
S26-R	41667	144.7	169.0	24.3
S335-R	40707	77.7	144.6	66.9
S36-R	51151	153.0	125.6	-27.4
S37A-R	52102	132.9	126.8	-6.1
S37B-R	52574	152.2	126.7	-25.5
S41-R	60637	116.9	122.2	5.3
S49-R	80056	88.7	89.5	0.8
S5A-R	64887	92.5	97.1	4.6
S5AY-R	66776	105.4	97.2	-8.2
S68-R	81905	135.5	135.9	0.4
S75-R	77647	112.7	122.3	9.6
S7-R	55398	125.7	105.5	-20.2
S7Z-R	59185	105.1	94.6	-10.5
S82-R	80012	110.4	127.3	16.9

station is much closer to the gage-adjusted radar rainfall data corresponding to S335-R. Observations such as this suggest that the cause of the bias may be attributed directly to errors in the gage data. However, this presumption deserves more thorough consideration.

Characterization of Convergence

In this section, we qualitatively examine the convergence properties of the gage-adjusted radar rainfall data. An expectation of zero bias between gage and gage-adjusted radar rainfall data presupposes that the areal average value of rainfall over the pixel in which the gage resides is spatially converged to the point value, for some time interval. This assumption may be expressed as:

$$\lim_{A \to A'} [R_A(t_0, t, A) - R_P(t_0, t, A')] = 0 \quad (1)$$

where R_A and R_P are areal average rainfall amounts (in units of rainfall depth, e.g., cm) accumulated for a period of time t, beginning at time t_0, where A is the area associated with R_A, while A' is the area associated with R_P.

We test the spatial conversion of areal average rainfall to a single pixel. We take R_P to equal the cumulative rainfall over a single pixel, the target pixel, and we consider the behavior of $R_A(t_0,t,A)$ as A approaches the area of one pixel, where A is the effective area enclosed by a circle whose center is at the centroid of the target pixel. The effective area is the total area of the pixels whose centroids are within the circle. We select t equal to one year. The value of t_0 (May 1, 2002) is selected so that the one-year period is WY03. We reason that if the data are not converged over such a length of time, it is unlikely that convergence occurs over shorter time intervals.

Figure 6 shows R_A versus A (number of pixels) for six randomly selected target pixels that cover telemetry stations. It is evident from the figure that the areal average rainfall does not converge as A approaches the size of a single target pixel (i.e., R_A does not approach R_P). Of the 79 pixels about which convergence has been tested, the rainfall data is most nearly converged about the pixel that covers the S7-R rain gage, and most poorly converged about the pixel that covers the S46-R rain gage. Even the annual rainfall amount for the pixel which contains S7-R differs by over five cm (two in) from the amounts corresponding to some of its neighbors. The annual rainfall amount for the pixel which contains S46-R differs by over 28 cm (11 in) from that of an adjacent pixel.

The results of the convergence test suggest that the assumption that the gage-adjusted radar rainfall data converge to the gage data is invalid, because the radar data themselves do not converge.

Discussion and Conclusion

There is apparently no relationship between bias and the location of a rainfall gage within a pixel. This is probably due to variability in the rainfall distribution at the sub-pixel scale.

Judging by visual inspection of Figure 5, it would seem that there is a good argument for the lack of any bias in the data set it represents, since half of the bias data fall below zero, half fall above, and the distribution is uniform. Still, it seems unreasonable that bias at individual gages / pixels can be less than –25 cm or greater than 25 cm. Although it is possible for rain to fall over a pixel and not over a rain gage, these differences still seem too great to be the result of a natural process, especially when one considers the span of the period of integration. Additionally, while there may be no overall bias in the gage-adjusted radar rainfall data, Figure 3 indicates that bias is significant in certain locations.

Some Challenges in Assessing Gage-Adjusted Radar Rainfall Bias

For some stations, rainfall amounts, as represented by gage-adjusted radar rainfall and gage rainfall, diverge over periods as long as one year, and may diverge over longer periods. This prompts further investigation into whether these divergences between radar rainfall and gage rainfall propagate beyond a single pixel. If so, it could make for significant bias in areal average rainfall over larger areas such as watersheds.

Areal average gage-adjusted radar rainfall data do not converge as the area over which the average is computed becomes smaller. This suggests that it is not reasonable to assume that the gage-adjusted radar rainfall amount for a single pixel is convergent upon the gage rainfall amount. This nonconvergence is probably due to the highly nonlinear nature of rainfall events. This nonconvergence of gage-adjusted radar rainfall data, although preventing the meaningful interpretation of a direct comparison between an individual gage value and an individual radar rainfall value, does not prevent the use of multiple pairs of data points in the assessment of tendencies in the differences between gage and gage-adjusted radar rainfall.

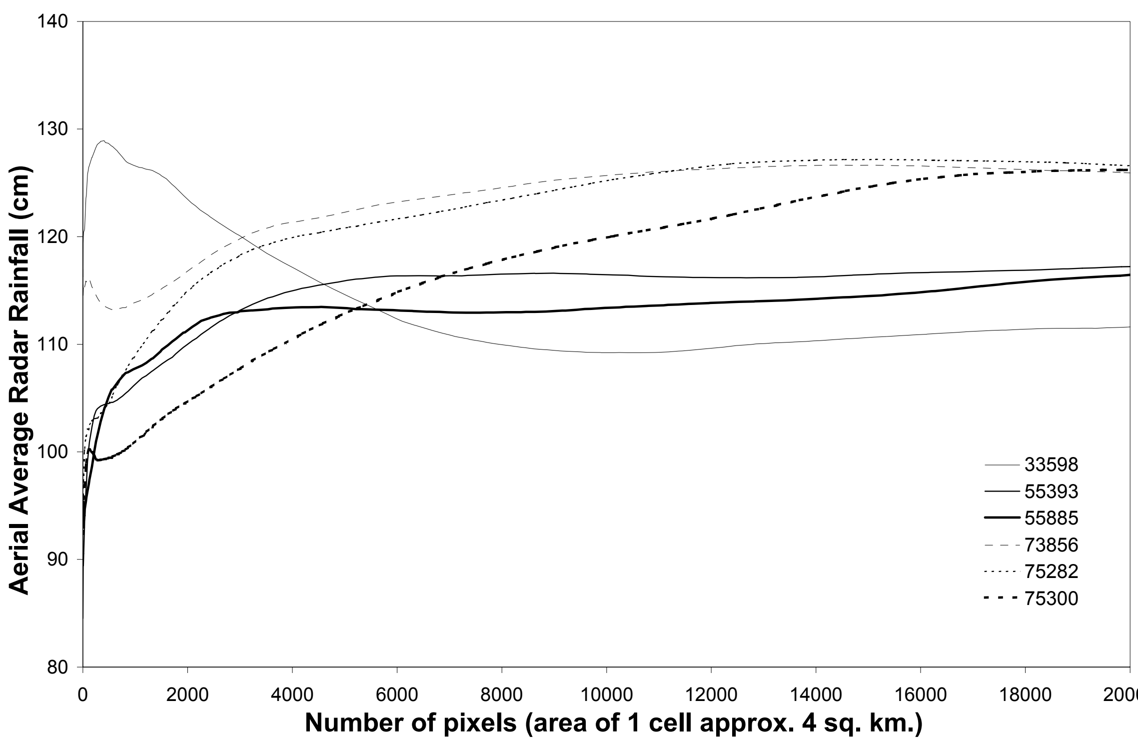

Figure 6: Convergence for six randomly selected pixels (indicated by line style). Note that pixels are paired with gages as follows: (33598, S167-R), (55393, G201-R), (55885, S39-R), (73856, S71-R), (75282, S72M-R), (75300, S135-R)

Future Work

Further studies may provide for more robust characterization of bias as a function of time interval, location, and rainstorm properties. The analyses presented herein support an investigation into the sources of the differences between the radar and gage data, and support revision of the methods and / or parameters used to compute the gage-adjusted radar rainfall data. It may be preferable for apparent discrepancies between gage-adjusted radar rainfall data and gage rainfall data to be explored more deeply before applying gage-adjusted radar rainfall data to watershed management at the SFWMD.

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Stormwater Pond Treatment Efficiency Evaluation

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Abstract

A series of wet detention ponds constructed in the late 1980s and early 1990s were studied to determine their treatment efficiency 15 years after pond construction. The treatment capacity of these wet detention ponds has been compromised due to infilling of the pond basins over time. The study indicates that sedimentation has reduced permanent pool volumes by up to 24% within 15 years of pond construction. The ponds captured up to 1,128 metric tons of sediment, 3.31 metric tons of nitrogen and 0.66 metric tons of phosphorus. Up to 64% of the studied ponds did not maintain their required littoral zones, with 55% having little to no littoral vegetation at all.

Introduction

The Southwest Florida Water Management District (District) became involved in the regulation of stormwater runoff in 1982 when it first received delegation of Chapter 17-25 F.A.C. from the Florida Department of Environmental Protection (FDEP). District Rules and Basis of Review specify performance standards to be met in the design and function of stormwater ponds. The ponds selected for this study were constructed using design criteria as set forth in Chapter 17-25 F.A.C. Wet detention ponds constructed using these design criteria were required to treat the first one-inch of runoff from the contributing area. The ponds were designed with a permanent pool and a littoral zone covering a minimum of 35 % of the pond's surface area at the control elevation.

There is limited information available about the physical condition and treatment performance of stormwater ponds that have been in service for many years. This study was conducted to answer three major questions regarding the performance of stormwater treatment ponds: (1) Has the treatment capacity of wet detention ponds been compromised due to the infilling of the pond basin over time? (2) How much material have the ponds captured since construction? (3) How does the current littoral zone development compare to design criteria?

Study Sites

The District chose 12 stormwater ponds for this study. The ponds were located in Pasco, Hillsborough, and Pinellas counties, as shown in Figure 1. These ponds were constructed in the late 1980s and early 1990s and range in size from 0.09 to 4.25 acres.

Materials and Methods

The District hired a consultant in 2004 to survey each pond in the study. The purpose of the survey was to identify the amount of sediment that had accumulated within each pond's basin. This was accomplished through linear transects using a standard 25-foot graduated survey rod with a 6-inch disc attached to the end of the rod. Probes were logged at the first noticeable point of resistance. The survey also included points along the top of bank, toe of slope, and pond bottom at each transect.

District staff researched permit files to determine control elevations, normal pool elevations, and percent coverage of littoral areas for each pond. Each site was visited on several occasions to photograph and document littoral shelf coverage. GIS software was used to determine the volume of sedimentation that had developed since pond construction, as well as verify vegetative cover (where applicable).

District staff conducted an extensive literature search to determine previous studies that involve stormwater pond sediment accumulation. Staff identified a study that was conducted in 1994 by Schuler and Yousef that quantified stormwater pond sediment density as well as the amount of nitrogen and phosphorus contained in a given amount of sediment.

Results

The survey data provided by the consultant was analyzed to determine the volume of sedimentation that occurred in each pond. Table 1 shows the pond size, sediment volume, permanent pool volume, and percent permanent pool reduction for each pond in the study. Over half of the ponds studied showed more than 10% permanent pool reduction with a maximum of 24% permanent pool reduction found in Pond 2. Pond 1 was not included in this part of the study because a permanent pool elevation could not be determined.

Table 2 shows how much sediment was captured in each pond as well as how much nitrogen and phosphorus had been retained. Pond 16 captured over 1100 metric tons of sediment since it was constructed. Pond 1, the smallest pond studied, captured over 40 metric tons of sediment. The amount of nitrogen removed from stormwater runoff via wet detention ranged from 0.02 to 3.31 metric tons. Phosphorus removal ranged from 0.00 to 0.66 metric tons.

Table 3 shows the designed and current littoral coverage of each pond studied. Only one pond studied (Pond 6a) was not designed to have a vegetated littoral zone. The remaining 11 ponds were required to have at least a 35% vegetated littoral zone with three ponds (Ponds 3, 6b, and 13) being designed to have greater than 35% vegetated littoral zones. Seven of the 11 ponds designed to have at least a 35% littoral coverage had 10% or less vegetative coverage. Six of the 11 ponds designed to have at least a 35% littoral coverage had 5% or less. Only four of the 11 ponds had 30% or greater littoral coverage.

Conclusions

In wet detention ponds, pollutant removal occurs primarily within the permanent pool. Sedimentation is the primary mechanism for the removal of particulate forms of pollutants. The following important pollutant removal processes occur within the permanent pool: uptake of nutrients by algae and rooted aquatic plants, adsorption of nutrients and heavy metals onto bottom sediments, biological oxidation of organic materials, and sedimentation of suspended solids and attached pollutants. The permanent pool also extends the residence time of water passing through the treatment pond. All of the ponds studied have lost permanent pool volumes to some degree, therefore their ability to treat stormwater has also been reduced. The magnitude of lost treatment capacity varies from pond to pond and there are varying opinions as to how much permanent pool loss is significant.

A literature review identified a previous stormwater pond study that quantified sediment density as well as the amount of nitrogen and phosphorus contained in a given amount of sediment. A 1994 study conducted by Schueler and Yousef examined the internal dynamics within the muck layer of over 50 stormwater ponds and wetlands aging from 3 to 25 years. The study contained information on the physical, chemical, and biological nature of the muck layer reported from 14 different researchers. Although the research studies covered a broad geographic range, nearly half of the sites were located in Florida or the Mid-Atlantic states. According to Schueler and Yousef (1994), the muck layer of a pond is high in organic matter and has a low density, averaging approximately 1.3g/cm^3. The muck layer is also highly enriched with nutrients. Schueler and Yousef reported phosphorous concentrations for 23 studies ranging from 110 to 1,936 mg/kg, with an average concentration of 583mg/kg. Nearly all the nitrogen found in pond muck is organic in nature. Total Kjeldahl nitrogen concentrations were reported from 20 studies ranged from 219 to 11,200mg/kg, with an average concentration of 2,931mg/kg. These average values were used to calculate metric tons of sediment, total Kjeldahl nitrogen, and total phosphorous as shown in Table 2.

Up to 64% of the 12 ponds studied did not maintain the required 35% littoral zone, with 55% having little to no littoral vegetation at all. Littoral vegetation creates the physical and biological conditions required for the successful removal of fine sediments, nutrients, and metals. Physical processes are more important in trapping pollutants during storm events because most pollutants are being transported at this time. Sufficient vegetation located perpendicular to the direction of flow increases uniform flow distribution and flow retardation, leading to increased pollutant contact with plant surfaces. Biological processes become important under low flow conditions when trapped pollutants are transformed and recycled. Vegetation also provides surface area for epiphytic algae to grow and aid in the removal of fine particles and uptake of nutrients. The root-zone binds and stabilizes deposited sediments preventing them from re-suspension. The root-zone can also help maintain an oxidized sediment surface layer preventing chemical transformation of settled pollutants (Wong et al. 1999).

It is evident wet detention ponds are in need of periodic maintenance that should include sediment removal and the preservation of vegetated littoral areas. How often this maintenance should occur varies for each pond. Further analysis of pond sedimentation is needed to determine how much sedimentation is too much and when it should be removed. It is also evident that wet detention ponds have the ability to capture significant amounts of sediment and related pollutants that would otherwise discharge into receiving water bodies.

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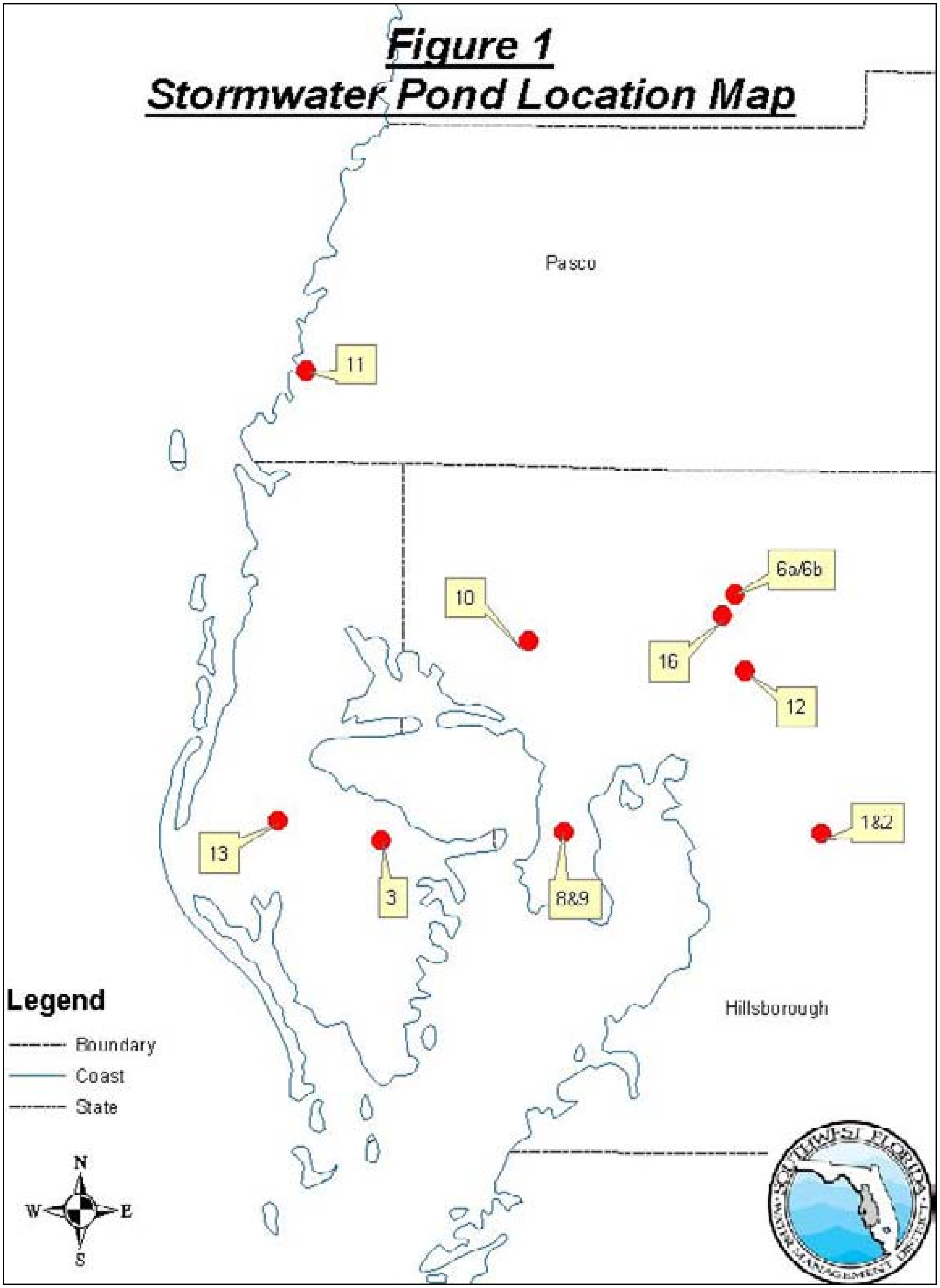
Figure 1
Stormwater Pond Location Map
Pasco
11
10
6a/6b
16
12
13
3
8&9
1&2
Hillsborough
Legend
Boundary
Coast
State
N
W
E
S
SOUTHWEST FLORIDA
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Table 1

Effects of Sedimentation in Stormwater Ponds

Pond Name	Pond #	Pond Size (ac)	Sediment Volume (m^3)	Permanent Pool Volume (m^3)	% Permanent Pool Reduction
Bloomingdale Plaza	2	0.99	240.73	994.04	24.22
Carillon	3	3.00	622.55	12813.87	4.86
Hidden River (sump)	6a	0.22	68.77	330.91	20.78
Hidden River	6b	4.25	621.16	5744.59	10.81
Lighthouse Bay	8	0.94	218.32	2333.97	9.35
Lighthouse Bay	9	0.28	22.06	112.52	19.61
Linebaugh Warehouse	10	0.11	4.41	358.30	1.23
Mariners Way	11	1.08	82.84	1444.07	5.74
SWFWMD TSO	12	0.78	292.15	1904.85	15.34
Trico Electrical Supplies	13	0.13	15.43	84.29	18.31
GTE Tech Center	16	3.73	867.86	10978.99	7.90

Table 2

Quantification of Accumulated Sediments in Stormwater Ponds

Pond Name	Pond #	Pond Size (ac)	Sediment Volume (m^3)	Sediment (metric tons)	Nitrogen (metric tons)	Phosphorus (metric tons)
Bloomingdale Plaza	1	0.09	31.59	41.07	0.12	0.02
Bloomingdale Plaza	2	0.99	240.73	312.95	0.92	0.18
Carillon	3	3.00	622.55	809.32	2.37	0.47
Hidden River (sump)	6a	0.22	68.77	89.40	0.26	0.05
Hidden River	6b	4.25	621.16	807.51	2.37	0.47
Lighthouse Bay	8	0.94	218.32	283.82	0.83	0.17
Lighthouse Bay	9	0.28	22.06	28.68	0.08	0.02
Linebaugh Warehouse	10	0.11	4.41	5.73	0.02	0.00
Mariners Way	11	1.08	82.84	107.69	0.32	0.06
SWFWMD TSO	12	0.78	292.15	379.80	1.11	0.22
Trico Electrical Supplies	13	0.13	15.43	20.06	0.06	0.01
GTE Tech Center	16	3.73	867.86	1128.22	3.31	0.66

Table 3

Littoral Zone Coverage

Pond Name	Pond #	Designed Littoral Shelf	Current Littoral Shelf
Bloomingdale Plaza	1	35%	0%
Bloomingdale Plaza	2	35%	35%
Carillon	3	>35%	5%
Hidden River (sump)	6a	0%	0%
Hidden River	6b	>35%	>35%
Lighthouse Bay	8	35%	5%
Lighthouse Bay	9	35%	10%
Linebaugh Warehouse	10	35%	1%
Mariners Way	11	40%	2%
SWFWMD TSO	12	35%	33%
Trico Electrical Supplies	13	100%	100%
GTE Tech Center	16	35%	2%

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