Effects of Urbanization on Groundwater

An Engineering Case-Based Approach for Sustainable Development

Edited by Ni-Bin Chang

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EFFECTS OF URBANIZATION ON GROUNDWATER

AN ENGINEERING CASE-BASED APPROACH FOR SUSTAINABLE DEVELOPMENT

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Foreword

For the past 50 years the population of the world has increased from 3 billion to 6.5 billion, and it is likely to rise by 2 billion by 2025 and by 3 billion by 2050. Following the current trends it is safe to say that the increasing number of people will dwell in cities. This will imply rapid urbanization, accelerating land use change, depleting groundwater resources, pollution of surface streams and rivers at an alarming rate, and decaying infrastructure at the same time. Water demand in urban regions would rise correspondingly or even more.

To make matters worse, there is the specter of climate change hanging over our heads. During the last one hundred years the temperature has arisen by nearly 0.6 degree C, and it is expected to rise by 2 degree C during the next 100 years. This would translate into the intensification of hydrologic cycle, rising sea levels, more variable patterns of rainfall (more intense rainfall, more extremes), more variable patterns of runoff (more frequently occurring floods and droughts), shorter snowfall season, spring snowmelt season starting earlier, increasing evaporation, deterioration in water quality, changing of ecosystems, migration of species, changes in the way plants grow, trees reacting to downpours, drying up of biomass during droughts, and quicker growing and then wilting of crops.

The impact on water management would entail serious ramifications. Larger floods would overwhelm existing control structures, reservoirs would not have enough water to store for people and plants during droughts, global warming would melt glaciers and cause snow to fall as rain, regimes of snow and ice, which are natural regulators-storing water in winter and releasing in summer, would undergo change, and there would be more swings between floods and droughts. It is likely that dams, after three decades of lull, might witness a come back.

Current patterns of use and abuse of water resources result in the amount being withdrawn dangerously close to the limit and even beyond; an alarming number of rivers no longer reach the sea: The Indus, the Rio Grande, the Colorado, the Murray-Darling, the Yellow River-the arteries of main grain growing areas in many parts of the world; freshwater fish populations are in precipitous decline: Fish stocks have fallen by 30% (WWF for Nature), larger than fall in populations of animals in any ecosystem; 50% of world's wetlands have been drained, damaged or destroyed in the 20th century; in addition to fall in volume of freshwater in rivers, invasion of saltwater in delta, and changing in balance between freshwater and salt water.

As compared to the global water resources situation, local water shortages are even multiplying; Australia has suffered a decade long drought; Brazil and South America which depend on hydroelectric power have suffered repeated brownouts-not enough water to drive turbines; excess pumping of water from rivers feeding led to an almost collapse of Aral Sea in Central Asia in 1980; global water crisis impinges on the supplies of food and other goods.

Water resources situation in the U.S. is facing the same trend with decaying infrastructure built 50 to 100 years ago, i.e., where 17% of treated water is lost due to leaky pipes. In Texas, there is ongoing drought; ranchers have already lost nearly 1 billion; worst hit are Central Texas and the Hill Country; December 2008-February 2009 has been the driest on record; 60% of the state's beef cows are in counties with severe to exceptional drought; in 2006, drought related crop and livestock losses were the worst for a single year, totaling 4.1 billions; effects are long-term.

The book "Effects of Urbanization on Groundwater: An Engineering Case-Based Approach to Sustainable Development," edited by Ni-Bin Chang is timely and addresses a number of key questions gravitating around the interactions amongst energy, environment, ecology, and socio-economic paradigms. The subject matter of the book will help promote sustainable management, with due consideration to linkages between regional economic development, population growth, and terrestrial hydrologic systems. It states challenges of and opportunities for science, technology and policy related to sustainable management of water.

Introducing sustainable development in urban regions in Chapter 1, the subject matter of the book is organized into four parts encompassing the remaining 13 chapters, each part corresponding to a specific theme. The theme of Part I is water supply and pollution prevention. Storm water management with regional infiltration technologies is the theme of Part II. Wastewater treatment and disposal with nutrient removal is the theme of Part III, and low impact development with landscape architecture technologies is covered in Part IV. These thematic areas cover the aspects from the fundamental theory to physical, chemical, and biological processes to the coupled human and natural environment, and to the representation of simulated evolutionary pathways. The linkage between these themes is thus becoming ever more important. Models of differing complexity have been used to study a wealth of well formulated engineering and management issues with risk assessment implications. Various real world applications in each chapter explore different impacts with varying degrees of sophistication.

The book will help improve our understanding of the sensitivity of key water quantity and quality management targets to urban development. The book is therefore timely and makes a strong case for sustainable development and management. The book is well written and well organized. Dr. Chang deserves a lot of applause to assemble an excellent array of chapters written by established professionals known for their technical contributions.

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Preface

During the last few decades, fast urbanization has altered such hydrologic cycle and related watershed processes that affect water resources and a range of potential consequences of urban development. This urbanization combined with economic growth and improving living standards in cities led to an addition to the quantity and complexity of generated wastewater effluents and stormwater runoff, which interrupt the hydrologic cycle and endanger the structure, function, and services provided by aquatic ecosystems. The negative feedbacks thus actuate an acute need to enhance fundamental understanding of the complex interactions within and among natural and human systems due to fast urbanization and its relevant countermeasures. These countermeasures that may lead to significant impacts on regional-scale hydrologic processes are basically linked with several disciplinary areas from water supply, to stormwater management, to wastewater treatment, and to groundwater conservation.

It is recognized that sustainable management is necessary at all phases of impact from the interactions among energy, environment, ecology and socioeconomic paradigms in human society. To promote the concept of sustainable management, this unique publication may be capable of presenting and applying sustainable systems engineering technologies to improve the overall understanding of the sensitivity of key water quantity and quality management targets to the types of human perturbations due to urban development. Hence, this book aims to address the following research topics in the context of the urbanization effects on groundwater:

- What are the potential impacts of water supply on groundwater aquifer and groundwater recharge rates, and how will these changes affect groundwater quality and/or quantity in both inland and coastal areas?
- What are the regional differences in stormwater and wastewater management technologies to urbanization?
- How can wetland extent and function be incorporated as an integral part of urban infrastructure systems, including effects on groundwater level?
- How will green infrastructure design philosophy influence the availability of suitable stormwater reuse and recharge for groundwater recovery?
- How can process-level models be improved to better represent the sensitivity of key water quality or quantity management targets to urbanization?
- How will changes in the low impact development strategies impact the hydrologic cycle in terms of both water quantity and quality in the nexus of stormwater management and groundwater conservation?

While focusing on the regional and urban watershed issues necessary for dealing with groundwater usage and quality endpoints, this book tries to answer all of the above questions as much as possible that capture important linkages between regional economic development, population growth, and terrestrial hydrologic systems.

Ni-Bin Chang, Editor
Orlando, Florida, March 25, 2009

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Table of Contents

Part IV: Low Impact Development with Landscape Architecture Technologies

CHAPTER 1

Sustainable Development in Urban Regions

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1.1 INTRODUCTION

Sustainable development is a means of improving civilization by such a way that we are able to retain our current quality of life and prosperity while not compromising the same benefit of future generations. To achieve such an overarching goal, it is recognized that sustainable management of the effects of urbanization is necessary at all phases of impact from the interactions among energy, environment, ecology and socioeconomic paradigms in human society. Sustainability engineering that is an emerging paradigm aims to preserve biodiversity, environmental resources and natural ecosystems as well as plan and act for the ability to maintain environmental quality, food security, public health, and energy supply in the very long run with the aid of all possible engineering alternatives optimized for applications. Among those concerns, the Integrated Water Resources Management (IWRM) is one of the most critical problems in relation to many aspects of urbanization.

In the last few decades, natural growth of population, economic development, landscape changes, reclassifications of habitation and migration trends are common in urban regions. When urbanization has become a global phenomenon, its ramifications are more pronounced in highly populated regions or countries. This urbanization, economic growth and improving living standards in cities, led to an addition to the quantity and complexity of generated wastewater and stormwater streams and compound the safety of drinking water supply. It is known that lack of clean water is responsible for more deaths in the world than war (NAE, 2009). About 1 out of every 6 people living today do not have adequate access to water, and more than double that number lack basic sanitation, for which water is needed (NAE, 2009). In some countries, half the population does not have access to safe drinking water, and hence is afflicted with poor health (NAE, 2009). As a consequence, how to assess the water cycle in urban environments leading to improvements in the efficiency and effectiveness of water usage becomes one of the keys to sustainable development.

In this book, an effort has been made to characterize the challenges and opportunities of the science, technology, and policy related to sustainable management of water. In particular, it responds to three out of fourteen grand challenges identified by the National Academy of Engineering (NAE), and pinpoints the relevance to the impacts of urbanization effects on groundwater. These three challenges are to: 1) manage the nitrogen cycle, 2) provide the access to clean water, and 3) restore and improve urban infrastructure (NAE, 2009). Concerns about wastewater and stormwater are not only because of their rising quantities but also because of the nutrient impacts. Besides, the effectiveness of management system has also received wide attention. The spatial characterization of wastewater and stormwater in urban areas is crucial for designing management strategies including its segregation, collection, transportation, and disposal through centralized versus decentralized networks. These arrangements not only result in unhygienic conditions in surface waters but also affect the groundwater quality and quantity to a great extent. The problems formulated in this regard are highly interdisciplinary while the engineering solutions for solving these issues are highly multidisciplinary.

1.2 CURRENT SCOPE AND FEATURE AREAS

Current research in the context of sustainablility engineering related to the urbanization effects on groundwater include but are not limited to: 1) deep well injection technologies for wastewater disposal, 2) passive on-site wastewater treatment technologies, 3) tertiary wastewater treatment with advanced nutrient removal technologies, 4) secondary wastewater treatment technologies with the addition of wetland systems for nutrient removal, 5) aquifer storage and recovery technologies for water supply and ecosystem conservation, 6) green infrastructure for regional stormwater management, 7) green building design with implications for the sustainable energy and environment, 8) low impact development (LID) technologies for integrating green street, green building, and green highway, 9) coastal aquifer remediation, 10) conjunctive use of surface and groundwater resources for water supply and ecosystem restoration, 11) river bank filtration technologies for water supply, stormwater treatment, erosion control, and river rehabilitation, 12) landfill leachate control using bioreactor technologies, 13) carbon dioxide (CO_2) separation in the flue gas of power plants and sequestration in geological formations, and 14) buffer zone design for coastal cities in dealing with storm surge, erosion control, and coastal management. Due to limitations of space, the main focus is on addressing four main topical areas in this book as below:

- Topical Area I: Water Supply and Pollution Prevention (Chapters 2-5)
- Topical Area II: Stormwater Management with Regional Infiltration Technologies (Chapters 6-8)
- Topical Area III: Wastewater Treatment and Disposal with Nutrient Removal (Chapters 9-11)
- Topical Area IV: Low Impact Development with Landscape Architecture Technologies (Chapters 12-14)

Both quantity and quality of water in the hydrologic cycle are driven by a variety of physical, chemical, and biological processes that operate at different space and time scales. Global climate change and cyclical droughts and floods threaten water supplies in many cities across the world. At the same time, rapid population growth has occurred in many urban regions, bringing additional demand for adequate quantity and quality of drinking water. Topical area I aims at studying the adaptive water resources management strategies by installing some Aquifer Storage and Recovery (ASR) wells for a semi-arid city in Texas where the agricultural irrigation might be a concern too (Chapter 2). To assess the impact, modeling the seawater intrusion in a highly complex coastal region may be required for remediation assessment and decision making (Chapter 4). Due to the embedded risk and involved uncertainties, the relevant policy analysis of the operational rules in such coastal aquifer may be analyzed by using the fuzzy rule base that leads to a multiobjective decision analysis via an integrated simulation and optimization model (Chapter 4). As for the watersheds located at some distance away from the coastal region, numerical analysis is still able to simulate the impacts on groundwater due to urbanization effects on a long-term basis (Chapter 5). To expand the water supply in a fast growing region, sustainable use of stone quarries turns out to be a possible option in the near future, which may be applied based on a holistic assessment approach (Chapter 3).

Urbanization increases pollutant loads in stormwater runoff because of the increased pollutant production by land use activities. Yet, green infrastructure is the interconnected network of open spaces and natural areas, such as greenways, wetlands, parks, forest preserves, and native plant vegetation, that naturally reduces flooding risk, manages stormwater, polish wastewater effluents, increase groundwater recharge, and improves water quality the use of proper infiltration technologies. Green infrastructure usually costs less to install and maintain than traditional wastewater and stormwater infrastructure. Topical area II focuses on studying the regional infiltration technologies for stormwater retention and detention ponds (Chapter 6 and Chapter 8) as well as for riverbank filtration to mitigate urban stormwater and polluted river water impacts on water quality (Chapter 7). It led to the development of multifunctional sorption materials and soil mixtures that may enhance the removal of various pollutants, including nutrient, bacterial, organic pollutants, and heavy metals, in the vadose zone. Several case studies throughout these chapters support the design philosophy.

In traditional wastewater treatment systems, both decentralized, on-site wastewater treatment systems (OWTSs) and centralized, public owned wastewater treatment systems (POWTSs) handle the needs for wastewater disposal. Novel approaches to proper planning, design and operation of the OWTS and POWTSs over different characteristic timescales will be necessary for integrating the components of the water cycle on land surface and in the subsurface systems. Topical area III concentrates on exploring the use of passive OWTSs with the sustainable implications of energy saving and material conservation on one hand (Chapter 9) and using wetland systems and rapid infiltration basins to polish the wastewater effluents and enhance the groundwater recharge on the other hand (Chapters 10 and 11). This trend combines the ecological engineering approach with wastewater treatment technologies in the biogeochemical cycle to smooth out the interfaces between the coupled natural and human systems. These green infrastructure systems make nutrient control and groundwater conservation part of sustainable cities at the regional scale in the 21st century.

At the local scale, LID is a new, comprehensive land planning and sustainable engineering design approach with a goal of maintaining and enhancing the pre-development hydrologic regime of urban and developing watersheds. Topical area IV therefore emphasizes the use of LID technologies to improve the landscape architecture design in the context of green street, green highway, and green building to improve both groundwater recharge and urban aesthetic value. Designing to infiltrate in various urban environments is the key component in LID practices (Chapter 12). Practical implementation may be assessed by the installations of greenways, rain-gardens, wetland restoration, green roofs, swales, permeable pavement, and native landscape. Within a wealth of LID technologies, use of permeable pavement systems, denoted as porous or open-graded pavement, can result in a reduction of construction costs for developers on one hand, and improve the groundwater recharge in the natural process with water purification potential on the other hand. Given a suit of natural conditions of concern, the pro and con effects of pervious pavement in urban areas need to be evaluated thoroughly with a practical approach. Its environmental effects should be examined to a great extent (Chapter 13). To have a sound systematic design, the modeling analyses of stormwater infiltration process through the pervious pavement may confirm the rises of water head in the saturated zone, thereby reducing its salinity levels in a costal city (Chapter 14). Such demonstration is just an example as to how to perform a holistic assessment of sustainability in urban regions.

1.3 FUTURE DIRECTIONS

Sustainability that by now inspires all cities from all points along the scientific spectrum toward the socioeconomic gateway has become one of the most passionately regarded notions in twenty-first century. In the future, novel approaches will be needed to accurately describe relevant hydrologic processes at resolutions suitable for modeling differing environmental and hydrological systems and handling various degrees of pollution prevention programs as a whole in urban regions. These environmental and hydrological systems are basically related to feedbacks between soil moisture and seasonal precipitation, the dependence of evapotranspiration on the dynamic evolution of the surface and groundwater water tables, and groundwater-stream exchanges combined with multimedia pollutant fate and transport processes. Surrounding two-way coupling of climate/meteorology with important surface and sub-surface hydrologic and biogeochemical cycles would be of interest in the IWRM regimes (Koster, 2004; Fan et al., 2007). Major challenges in engineering practices mainly include proper integration among sustainable water supply, LID implementation, stormwater management and green infrastructure, and sustainable wastewater treatment and disposal.

Taken together, these principles and practices embedded in the above four topical areas in the book provide a strategic approach and analytical framework for resources conservation and environmental management that can advance the sustainable use of land while providing urban regions with an interconnected system in relation to natural habitat. For instance, future research on how two-way coupling across interfaces between regional-scale infiltration technologies and local-scale landscape architecture technologies to minimize the urban runoff can be important determinants of urban sustainability. As a consequence, LID landscape architecture technologies may be combined with regional infiltration technologies via stormwater retention and detention ponds to improve the combined sewer overflow (CSO) management strategies, if any. Such an integrated system may be extended to household-based separate collection system between the black and gray water streams leading to the improvement of stormwater and wastewater reuse at different scales. It is anticipated that the coupling of climate change impacts, population growth and migration and economic development with existing or new hydrologic and hydrodynamics models and adaptive water resources management strategies over this breadth of space and time scales will pose a number of significant challenges. Successful initiatives can be based on common principles and share similar sustainable systems engineering strategies, such as greenway planning and design, ecosystem management, watershed protection, conservation development, habitat restoration, stream improvement, and greenprints, to achieve the overarching goal of urban sustainability.

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CHAPTER 2

Design, Installation, and Operation Challenges of Large-Scale Aquifer Storage and Recovery Wells in San Antonio, South Texas

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ABSTRACT: Six locations and five separate aquifer systems for Aquifer Storage Recovery (ASR) were evaluated before selecting the Carrizo Aquifer option in San Antonio, Texas in 2004. This site was selected since it provided the lowest costs for transmission, site development and operation. The Twin Oaks ASR facility, located at forty-eight kilometers (30 miles) south of San Antonio, Texas, was then developed by the San Antonio Water System to capture surplus water during wet months and store it underground for drought management and emergency relief. Water is injected into a semi-confined sand aquifer, forming a large water bubble. During times when groundwater levels are high, water from the Edwards Aquifer is treated to meet drinking water standards and is then pumped to the Carrizo Aquifer for storage. A total of 29 high capacity ASR wells and three native Carrizo Aquifer pumping wells were installed using flooded reverse circulation and mud rotary techniques. Well tests were conducted between 2,725 to 19,075 cubic meters per day, m^3/d (500 to 3,500 gpm) with design pumping yields around 9,810 to 13,625 m^3/d (1,800 to 2,500 gpm). Recharge capacities range from 6,540 to 10,900 m^3/d (1,200 to 2,000 gpm). The receiving Carrizo Aquifer has a natural pH of 5.5 and is high in iron and manganese, so a storage, treatment, and pumping system was built to treat the recovered water and deliver up to 2.3 million cubic meters per day, MCMD (60 million gallons per day) back to San Antonio. To date, the facility has been mostly used for banking the recharged water, achieving a cumulative storage volume of up to 75 million cubic meters, MCM (61,000 acre feet). Recovery pumping is currently underway, augmenting local water supplies during the current drought. The recovered water from the ASR wells has not required retreatment, other than disinfection. The wells are backwash pumped monthly to maintain recharge efficiency.

2.1 INTRODUCTION

The need to store water underground is increasingly recognized as a key component of any plan to achieve water supply reliability and sustainability, whether to meet urban needs or to meet industrial, agricultural, environmental and other water requirements. Small ASR projects to meet local needs are increasingly being expanded to meet regional water management objectives. One big advantage of storing water in an underground sand aquifer instead of a reservoir is that no water evaporates. The primary driver for rapid development and implementation of ASR technology during the past 40 years has been favorable economics. ASR typically has a unit capital cost of about $0.33 per liter ($1.25 per gallon) per day of recovery capacity, within a typical range of $0.13 to $0.53 per liter ($0.50 to $2.00 per gallon), depending primarily upon individual well yield. Higher capacity wells tend to correlate with low unit capital costs. When compared with other options for water storage, ASR wells are typically less than half the capital cost. When compared with other options for augmenting peak water supplies, ASR technology is usually economically favorable. Other drivers for ASR implementation include the demonstrated viability of this technology in a broad range of applications and geographic settings; the insignificant adverse impacts upon the environment and groundwater quality; and the ability to add wells in small increments of capacity, as needed to keep pace with increasing water demands.

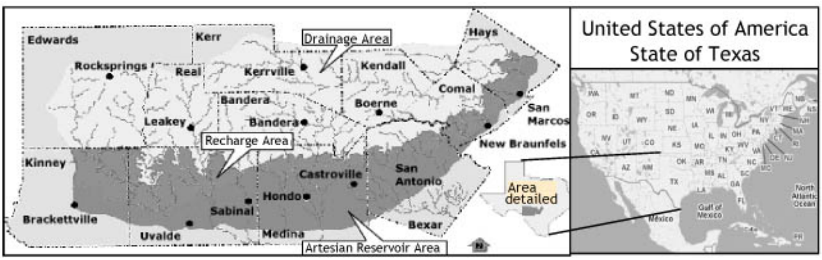

Figure 1 Edwards Aquifer, San Antonio, Texas (Plummer, 2000)

For this reason, the San Antonio Water System (SAWS) has developed a unique aquifer storage recovery (ASR) program, banking groundwater from a karst limestone aquifer with high transmissivity into a distant sand aquifer with relatively low transmissivity. In the rural farmland of south central Texas, 48.3 kilometers (30 miles) south of San Antonio, the Twin Oaks ASR facility was built to bank potable water for seasonal recovery during peak demand periods. The system is designed to recharge excess Edwards Aquifer groundwater during times of plenty into the Carrizo Aquifer through several ASR wells (see Figure 1). Before water can be injected it has to meet drinking water quality standards so that there is no chance that water already in the ground could be contaminated. To ensure that no adverse water quality impacts occur from recovery of the recharged water and blending with any native Carrizo Aquifer water, all recovered water is treated back to the normal system quality before it is used by the city of San Antonio.

The Balcones Fault Zone Edwards Aquifer (Edwards Aquifer) in south central Texas is one of the most permeable and productive aquifers in the United States. The aquifer is the primary source of water for approximately 1.7 million people in the region and provides most of the water for agriculture and industry. The San Antonio segment of the aquifer extends a distance of approximately 290 kilometers (180 miles). The flow from two notable aquifer discharge points, Comal Springs and San Marcos Springs, was greatly reduced, or ceased, during drought periods in 1956, 1966, 1971, 1984, 1989, 1990, and 1996 (Schindel, 2005). In response, the Texas legislature passed Senate Bill 1477 in 1993 mandating a reduction in pumping from the Edwards Aquifer. The Edwards Aquifer Authority was formed to protect the aquifer and set staged limits for withdrawals. In order to maintain adequate spring flow that protects aquatic habitat for a number of threatened and endangered species, the elevation of the aquifer at selected control point wells dictates the pumping limitations. Spring flow also provides a significant portion of water for downstream interests in the Guadalupe River basin (Todd Engineers, 2005). If the water level is above the 70-year mean, then normal allotment pumping is allowed. As the aquifer level drops into concerned and critical levels, allowable pumping volumes are reduced.

The Twin Oaks ASR facility is designed to bank water from the Edwards Aquifer during high water level conditions for delivery back to the community during restricted aquifer pumping periods. The $250 million facility began operation in 2004 and consisted of 48.3 kilometers (30 miles) of transmission pipeline, ASR wells capable of storing up to 65 million cubic meters (53,000 acre feet, AF) per year; a water treatment plant to remove iron and manganese that occurs in the Carrizo Aquifer; an on-site storage tank; and a bank of 1000+ horsepower pumps to reverse the flow in the sole supply line and return the treated water back into the San Antonio community water system. The system was further upgraded in 2006 to include additional ASR and native Carrizo pumping wells, storage capacity, and delivery pumping capacity.

The ASR technology allows underground storage of seasonally available water supplies in the San Antonio area. It provides an additional source of water in the SAWS portfolio that can be called upon during drought periods or emergencies. The sustainability of the storage facility is excellent for short term and moderate term storage. Its viability for long term storage will depend upon the rate of movement of the stored water towards adjacent pumping centers outside the property boundary. Since the aquifer historically has been in equilibrium between natural recharge and current local pumping demand, a "storage pocket" does not exist at this site to retain the recharged water. Instead the stored water bubble displaces ambient groundwater around the individual wells. Eventually the stored water bubbles will tend to coalesce, forming a single large bubble around the wellfield when the stored water volume reaches the large storage capacity of this 1,295 hectares (3,200 acre) site. The recharged water will likely tend to move slowly towards any pumping center at a velocity dependent upon the gradient of water levels in the aquifer near the property line. Careful management will be required to ensure that, during long term storage, migration of the recharge water bubble away from the ASR wells does not occur.

At present, SAWS operates the third largest ASR wellfield in the United States, following behind Las Vegas Valley Water District in Nevada with 594,245 m^3/d (157 MGD) recovery capacity, and Calleguas Municipal Water District in California with

257,380 m^3/d (68 MGD) recovery capacity. It is currently estimated that more than 95 ASR wellfields are operational in at least 20 states, with a total of about 500 ASR wells in operation. Many more ASR wells nationwide are in design, permitting, construction or testing, but have not yet received operational permits. States with the greatest amount of ASR activity include New Jersey, South Carolina, Florida, Arizona, California and Oregon. Storage aquifers include confined and unconfined aquifers; fresh, brackish and saline aquifers with ambient groundwater total dissolved solids concentrations up to 37,000 mg/l. A broad range of geologic settings includes karst limestone and dolomite, sand, sandstone, alluvial deposits, basalt, conglomerates, glacial and fractured hardrock aquifers. ASR well depths are as deep as 823 meters (2,700 feet) and individual well yields are as great as 30,208 m^3/d (8 MGD) (Pyne, 2005).

2.2 GEOLOGIC FORMATION

A unique set of strata, the Carrizo-Wilcox Formation, was selected for its marginal water quality and low demand, depth and confinement, and the lack of hydraulic connection to the Edwards Aquifer or regional rivers. The Carrizo Aquifer that is used locally for farmland irrigation is a sandstone aquifer with a pH of 5.5, containing elevated iron and manganese. Up to 75.3 MCM (61,000 AF) of water have been stored during the first five years of operation. The ASR wellfield is located in the Texas Coastal Plain on the downthrown side of the Balcones Escarpment. Formation dip is approximately 28.4 meters per kilometer (150 feet per mile) in the south south-east direction. Several mapped and un-mapped extensional faults trend in the north-northeast direction. The target formation, the Carrizo Sand, is a medium to very coarse grained, noncalcareous sandstone. It is friable to indurate with thick beds and local iron-oxide banding. The Carrizo Sand ranges from 213 to 244 meters (700 to 800 feet) thick in the area and yields moderate to large supplies of fresh water. The wells are typically screened between 122 and 213 meters (400 and 700 feet) below surface in the formation. The Reklaw Formation comprises the confining layer over the Carrizo Sand. The 61 meter (200 foot) thick unit is a fine to medium grained sandstone and silty clay with abundant hematite, glauconite and muscovite. The Wilcox Group underlies the Carrizo Sand and is composed of mudstone and varying amounts of sandstone and lignite.

The local farmers and Bexar Metropolitan Water District had concerns that the ASR facility would adversely impact the local aquifer. This resulted in an agreement with SAWS to take steps to implement mitigation measures to minimize impacts to area residents. Under rules of the local groundwater district and the agreement, property owners are permitted 2,467 cubic meters (2 acre feet) per year groundwater production for each 0.405 hectare (1-acre) of land. The facility includes 1,295 hectares (3,200 acres) of farmland netting 7.8 MCM (6,400 AF) potential water production rights from the native Carrizo Aquifer. During the recovery of the banked water some mixing is anticipated toward the end of the recovery periods, resulting in recovery of Carrizo aquifer water blended with stored drinking water. Ultimately the banked water, as well as the permitted Carrizo water, adds to the total water sources for SAWS.

2.3 SITE CHARACTERIZATION

The Twin Oaks Recharge facility design incorporates recent technology advancements for a large scale ASR program. SAWS interviewed many other agencies performing aquifer recharge through wells. The agency then conducted a site characterization and feasibility program; designed an isolated recharge and recovery facility and developed the program into a full scale operation in 2004.

In 1996, SAWS and Bexar Metropolitan Water District began investigating the feasibility of banking water underground. Site selection and source water compatibility were thoroughly evaluated, considering potential pre and post treatment costs, in ranking the potential ASR application scenarios. Site selection included the evaluation of five different aquifer types at six unique locations. This included the Middle Trinity Aquifer; the Lower Trinity Aquifer; the brackish Edwards Aquifer; the Wilcox Group; and the Carrizo Aquifer. Source water considerations included raw surface water to treated groundwater. Potential sources included raw water from Lake Medina; raw water from the Medina River; raw water from Canyon Lake; treated water from Canyon Regional Water Authority; or treated Edwards Aquifer water. Qualitative geochemical analysis suggested that although adverse reactions are possible, each of the six storage sites was technically suitable for ASR development. Further considerations for each area were made as to the historical and projected water use; general distribution of groundwater users; and delivery requirements.

Finally, the capital, operation and maintenance cost estimates were developed for a typical ASR installation within each zone assuming large-scale ASR implementation. Analysis indicated operation and maintenance costs would range from a low of $.03 per cubic meter ($0.11 per 1,000 gallons) for the Carrizo Aquifer option to a high of $0.09 per cubic meter ($0.34 per 1,000 gallons) for the Lower Trinity Aquifer option. The marginal cost of water produced from an ASR facility including capital costs was estimated to range from $0.66 per cubic meter ($82 per acre-foot) for the Carrizo Aquifer option to $3.21 per cubic meter ($398 per acre-foot) for the Lower Trinity Aquifer option.

By the spring of 2000, the Carrizo Aquifer option was further evaluated in the second phase of the feasibility evaluation. This evaluation included the installation of test and monitoring wells. The five test borings were installed at depths of 213 meters to 579 meters (700 feet to 1,900 feet) to evaluate the aquifer strata. The ASR wells were subsequently completed with screens extending down 183 to 213 meters (600 to 700 feet). Geophysical logging, lithologic descriptions, and core samples were obtained along with water chemistry data from the completed monitoring wells. Water samples were collected and analyzed to determine the chemical constituents in the native groundwater sections encountered. Chemical and geochemical analysis on the water and the core samples concluded that there were no significant concerns regarding implementation of an ASR project in the Carrizo Aquifer using treated Edwards Aquifer water as the source water.

The aquifer capacity evaluation suggested that up to 227,100 m^3/d (60 MGD) of ASR capacity could be developed. Assuming an average seasonal recovery cycle of four months, the ASR wellfield would supply 27.6 MCM (22,400 acre-feet) of supply to help meet peak demands. The analysis indicated that continuous two year recovery at 227,100 m^3/d (60 MGD) was possible with no adverse impacts to the aquifer. Under more demanding conditions, the analysis also indicated that 113,550 m^3/d (30 MGD) of native Carrizo Aquifer water could be recovered for a 50-year yield with no adverse impacts. The conceptual design for the ASR wellfield recommended approximately 30 wells and 21 kilometers (13 miles) of piping connecting to surface storage and pumping facilities. By October 2000, a preliminary design was completed for the wells, treatment plant, on-site piping, and supply line. A total of 11 options for the supply line connection to the San Antonio source water were considered. The 48.3 kilometer (30 mile) pipeline route was selected based upon project schedule, property acquisition, ease of constructability, costs and integration of the returned water at points of greatest need to support growth.

The wellfield geochemical compatibility was tested through three cycle tests. The first test consisted of three days continuous recharge at two test wells for a total volume of approximately 22,710 cubic meters (6-million gallons) each. This test utilized a mixture of water that was in storage in the 48.3 kilometer (30-mile) pipeline and allowed the pipeline to be filled entirely with the projected Edwards Aquifer source water. No stand time was allowed and the wells were immediately pumped, recovering nearly 30,280 cubic meters (8-million gallons) each. The second test increased the radial impact with a recharge volume of 68,130 to 75,700 cubic meters (18 to 20 million gallons - MG) each over 10 days. The water was allowed a 7-day stand time, and then approximately 80 to 90 percent of the recharge volume was recovered. The third cycle included over 22 days recharge with volumes of approximately 166,540 cubic meters (44 million gallons) per well. A 19-day stand time was provided before recovering 75 to 80 percent of the recharge volume.

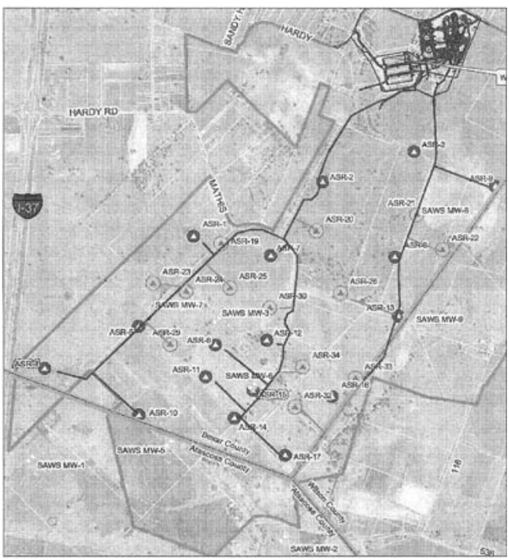

Figure 2: ASR Wellfield Layout (SAWS, 2005)

Testing indicated that mixing of the native Carrizo aquifer water became apparent around 80 percent recovery, so the "mixed" volume of water from the last two cycle tests was left in the ground to create a buffer zone around the wells. The recovered water was injected into other portions of the wellfield to prevent a resource loss. The cycle test volumes were too small to activate the 48.3 kilometer (30-mile) pipeline system and pumps for recovery of the water to the city of San Antonio.

The cycle tests indicated that the recharge water bubble did not mobilize constituents such as arsenic, and the recovered water met drinking water standards. Water levels did not exceed land surface during any of the testing periods. There were minor changes in well efficiency during injection with most losses being fully restored following recovery pumping. During 2006 the ASR wellfield was expanded to include 12 new ASR wells, raising the total to 29 ASR wells and recharge/recovery capacity to 227,100 m^3/d (60 MGD). Figure 2 shows the wellfield layout. Three Carrizo Aquifer wells (production only) were added in order to develop the 7.9 million cubic meters (6,400 AF) water right on the wellfield property. A 75,700 m^3/d (20 MGD) high service pump was added, plus three 52,990 m^3/d (14 MGD) transfer pumps, five booster pumps, a pressure reducing station and a 28,387 cubic meter (7.5 MG) clear well.

2.4 AQUIFER RECHARGE

Many ASR programs are developed in previously stressed aquifers that have experienced significant lowering of water levels, affording a "pocket" for storing water. Many others store water in aquifers that are in an approximate equilibrium between natural recharge and discharge. For these ASR wellfields the stored water laterally displaces the ambient groundwater, forming a bubble around individual ASR wells or around the entire wellfield. The SAWS program utilizes an aquifer that is confined and near equilibrium with small scale use for agriculture, domestic and minor municipal supply. Therefore, no pocket exists into which to place the recharged water. This may create a challenge to retain the stored water bubble that is being banked from season to season. If the water is not recovered in a timely manner, the water may tend to disperse rather than remain in the area of proposed recovery. The potential water banking duration will depend upon the rate of lateral groundwater movement of the stored water bubble underground. This, in turn, will depend upon the gradient of the water levels in the aquifer at the wellfield property line, imposed by pumping from nearby wells.

During cycle testing, the managed recharge into the aquifer was generally at a rate of 7,630 m^3/d (1,400 gpm). This produced a net water level rise of about 7.6 meters (25 feet) at 60-minutes for an average injection specific capacity of 1001 cubic meters per day per meter (56 gpm/ft). This is similar to the production specific capacity observed during the recovery pumping cycles. The source water availability for recharge can be highly variable, depending on seasonal weather conditions. Cyclic droughts will result in seasons with little to no recharge water available. During seasons with rainfall above the mean, water will be placed into storage.

Banked water not taken by the summer demand will then be available as a resource during drought years.

To boost the water available from the Edwards Aquifer, the regional aquifer authority at the time installed infiltration basins on the exposed sections, or natural recharge sections, of the Edwards Formation in the mid 1970's and early 1980's. The pilot basins recharged over 185 million cubic meters (150,000 AF) in the 30 years of operation. This was only a fraction of the nearly 34,542 million cubic meters (28-million AF) of surplus storm water that flowed through the creeks during the same period. Currently, the ability is being evaluated to install more infiltration basins in the Edwards Aquifer recharge zone. Computer modeling is being used to evaluate the hydraulic impact on the aquifer as a result of more recharge. In the karst setting, the recharged water could move quickly through the system. Spring discharges will increase as a result of the added system head. These losses could add up if the water is allowed to sit there too long. This is where the Twin Oaks Recharge Facility allows this managed recharge to the Edwards Aquifer to be wheeled some distance through the aquifer, removed through pumping, and then piped south to be placed back underground in the Carrizo Aquifer for extended storage. Development of this program will increase the sustainability of the Twin Oaks operation.

2.5 AQUIFER DISCHARGE

The Carrizo Aquifer is a confined unit with recharge/discharge occurring through communication with surrounding stratigraphic layers. Physical exits within the aquifer where stored water could be lost were not identified in the feasibility study. The confined nature of the aquifer will prevent the loss of the stored water into adjacent stratigraphic layers, helping protect from losses to undeveloped aquifer sections. The ASR well construction also helps prevent upward fluid migration through placement of annular seals from surface to the top of the target aquifer sections.

Through installation and testing of the 41 cm and 51 cm (16-inch and 20-inch) ASR wells, it was determined that the aquifer could pump 13,625 to 19,075 m^3/d (2,500 to 3,500 gpm) maximum with recovery designs averaging 9,810 to 13,625 m^3/d (1,800 to 2,500 gpm). Recovery specific capacities ranged from 411 to 1,556 m^3/d per meter (23 to 87 gpm/ft) at design rates, with most wells around 805 to 1,073 m^3/d per meter (45 to 60 gpm/ft). Differences in specific capacity are attributed primarily to well construction deficiencies. Different well drilling procedures caused significantly different formation clogging, and poor construction resulted in cement in the screen for several wells.

Water produced from the Twin Oaks ASR facility must match the typical distribution system water quality in San Antonio. The properties of the water delivered to the city must not create any adverse conditions such as corrosion, color, odor, or taste. So the water recovered from the ASR wells and any portion of native Carrizo Aquifer water that is blended with the ASR recovered water, is processed at the on-site treatment facility as needed to remove the iron and manganese, stabilize the pH and restore a disinfection residual. If no blending occurs with native Carrizo

Aquifer water, and if the only water being treated is the water recovered from the ASR wells, only disinfection treatment is needed.

Figure 3: Water Treatment Plant (Morris, 2006)

2.6 WATER QUALITY

Typically there is concern whether the source water will plug the receiving aquifer and what level of pretreatment is necessary to prevent this. In this case, native groundwater pumped from one aquifer is piped and stored in another aquifer of lesser quality. The storage aquifer has much slower groundwater movement, allowing retention of the water for later recovery. The source water for recharge is taken from the potable distribution system and has already been treated with chlorine for disinfection. The water has very low turbidity, total suspended solids, salts and metals. Figure 3 shows the outlook of the water treatment plant.

The Twin Oaks Recharge facility must be maintained in some sort of recharge or production operation most of the time. During non-operational periods, there is a chance that slow or non-moving water in the large diameter transmission main between the facility and San Antonio will stagnate. The volume of water stored in the pipeline is significant, on the order of 68,130 to 75,700 cubic meters (18 to 20 million gallons). Therefore, purging this water to waste on any routine basis is a large loss. During the first two years of operation (2004 to 2006) regional precipitation was above normal, allowing the facility to be operated in the recharge mode to develop a water bank in the Carrizo Aquifer. Since then, some water has been recovered to help meet peak demands during drought months.

The water quality issues for the recharge program are tied to the marginal quality of the receiving aquifer and the impact this has on the recovered water. The native Carrizo Aquifer water has a pH of approximately 5.5 due to the extremely high levels of carbon dioxide, Iron at 50 times the drinking water standards, Odor at 67 times the recommended limit, Hydrogen Sulfide at 40 times the limit, and Manganese at 6 times the recommended maximum concentration level for drinking water. In order to recover all of the recharged water, some portion of the native Carrizo Aquifer water would need to be removed from the transition zone of the two water types, degrading the resulting water quality.

Table 1: Receiving Aquifer Water Quality Requirements (SAWS, 2005)

CONTAMINANT	UNITS	RAW WATER DESIGN	MCL	FINISHED WATER GOAL
Primary Drinking Water Contaminant Levels				
Radium- 226&228, Total	pCi/l	4.50	5.00	<3.0
Arsenic	mg/L	<0.01	0.01	<0.01
Secondary Drinking Water Contaminant Levels				
Color, True	Color units	20	15	<3.00
Hydrogen Sulfide	mg/l	2	0.05	<0.03
Iron	mg/l	15	0.3	<0.2
Manganese	mg/l	0.30	0.05	<0.03
pH		5.50	>7.0	>7.8
Odor	T.O.N.	200	3	<2.5
Other Parameters				
Alkalinity, Total	mg/l as $CaCO_3$	30	NA	>100
Calcium	mg/l	23	NA	>40
Dissolved Oxygen	mg/l	0.10	NA	>2.0
Hardness, Total	mg/l as $CaCO_3$	35	NA	>100
Radon	pCi/l	160	300[1]	<100
Turbidity	NTU	0.20	<0.5	<0.1
Corrosion Indices				
Langelier Index		-2.0	NA	>0.20
Ryznar Index		10.1	NA	<7.1
Disinfection By-Products				
Bromate	mg/l	NA	0.010	<0.005
Total Trihalomethanes (TTHM)	mg/l	0.007	0.080	<0.040
Haloacetic Acids (HAA5)	mg/l	0.004	0.060	<0.030

[1] Radon is not currently regulated, MCL shown is proposed.

The 7.8 million cubic meters (6,400 AF) of water rights that SAWS has on the Carrizo Aquifer supports the volume of native water removed for mixing, though the primary intent for this right is to serve as an additional water supply for the City of San Antonio. For this reason, a full scale water treatment plant was constructed at the Twin Oaks Facility to treat the native Carrizo Aquifer water and recharge water blends to normal distribution system standards. Table 1 lists the receiving aquifer water quality requirements.

Experience at other ASR wellfields nationwide has demonstrated the wisdom of maintaining a buffer zone that separates the stored drinking water from the surrounding ambient groundwater. Where such a buffer zone is formed and maintained, recovered water quality usually remains relatively stable. For SAWS, maintaining a buffer zone has resulted in the recovered water not requiring retreatment other than disinfection. The buffer zone volume is usually a fixed volume, estimated initially and then adjusted through operating experience. It is not water that is stored and lost each year.

The water treatment plant processes include alkalinity adjustment, aeration, chemical addition, solids content clarification, filtration, and then disinfection. Back-flush fluids are routed to lagoons for evaporation and solids separation. The plant output meets or exceeds all Texas Commission of Environmental Quality (TCEQ) regulatory requirements including all national primary and secondary drinking water quality standards. This ensures that the water provided by the Twin Oaks Facility does not impact or corrode the distribution system and is aesthetically pleasing.

2.7 WELL DESIGN

A total of 29 ASR wells have been installed during construction phases in 2000 and 2006. The wells were installed using direct and reverse drilling programs with variable fluid control programs. The first 17 wells were designed with 41 cm (16-inch) casing and the 12 newer wells with 51 cm (20-inch) casing. All wells are constructed with 304-stainless steel rod-based wire-wrapped well screen. Slot openings ranged from 0.076 to 0.114 cm (0.030 to 0.045 inches). The blank well casing was mild steel with an internal 9-mill epoxy coating machined with threaded end sections. A dielectric coupling with Teflon was installed between the stainless steel well screen and the mild steel casing to limit potential galvanic corrosion. Each well is equipped with a water level sounding tube and gravel feed tube. The screen was enveloped in a minimum 10.2 cm (4-inch) gravel pack composed of well rounded silica sand. The annular space of each well was sealed with cement grout from the top of the target formation to land surface.

The well design included ample room for drawdown under pumping conditions without exposing the well screen. Increasing the well casing size from 41 cm to 51 cm (16-inch to 20-inch) enabled the installation of larger pumps and the increase in individual well yield by nearly 5,450 m^3/d (1,000 gpm). Much of this efficiency improvement is attributed to installation of the wells using a high quality, low formation invasion, fluid control program during drilling. In one well design, ASR-17, a pre-gravel packed screen was used in place of an installed envelope. No adverse performance issues are noted for this well and unique design.

2.8 WELL DRILLING AND CASING INSTALLATION

In the first well construction phase, 17 wells were installed by two different contractors using different drilling methods. Nearly half of the ASR wells were drilled mud rotary with bentonite and synthetic polymers. The other half were drilled with flooded reverse circulation utilizing a light-weight water, polymer, and bentonite fluid. Each well site included a small diameter pilot boring drilled mud rotary to depths of 183 to 236 meters (600 to 775 feet) for lithologic control and geophysical logging. Each ASR well drilling method resulted in substantial formation invasion by the drilling fluids. The direct mud rotary wells produced higher efficiencies after chemical cleaning and development than the reverse circulation wells did. This was attributed to the high invasion of solids into the formation during reverse circulation drilling using a high solids, water-based fluid program. The formation invasion and well losses were confirmed by the geophysical logs and subsequent well yields. In the second well construction phase in 2006, 12 wells were drilled flooded reverse circulation utilizing a high quality bentonite polymer water-based drilling fluid program resulting in much less formation invasion (CH2M HILL, 2006).

Numerous problems were encountered with borehole stability. In several cases, the 41 cm (16-inch) casing was not able to be installed in the specified 61 cm (24-inch) boreholes, resulting in casing removal and re-reaming to 66 cm (26-inches) or larger due to washouts. In some cases, exceedingly low well yields were partly attributed to having too large a gravel envelope and not being able to effectively clean the borehole wall. The 51 cm (20-inch) cased wells were reamed to 86 cm (34-inches) at a minimum with very few problems during installation (CH2M HILL, 2003). In two wells, gravel feed tube failures resulted in cement grout flowing into the tubing breaks and down into the screen zone. Numerous attempts to brush, jet, sonic blast, acidize, and ream out the cement were unsuccessful until another contractor came in and filled the well casing with sand to the affected area and applied a concentrated dose of acid. This opened the screens to the aquifer in one well. The sand was subsequently removed from the well casing. The other well was damaged beyond repair, leaving 16 wells available for service during the first phase of well construction.

2.9 GEOPHYSICAL TESTS

Borehole geophysics study was completed in nearly every boring including lithologic test borings, monitoring wells, and all of the ASR wells. Geophysical data from the test wells to 579 meters (1,900 feet) below the facility helped with design of the target formation depth. Monitoring well geophysical logs were used to define formation diversity and enabled arrangement of the wells within the wellfield. The individual ASR well geophysical logs enabled selection of screen locations and aided in slot sizing.

The geophysical log suite included Caliper, Spontaneous Potential, Natural Gamma, Natural Gamma Ray Spectroscopy, Density, Induction, Magnetic Resonance, and Elemental Capture Spectroscopy. The gamma, density and induction logs aided in identifying coarse grained sections of the aquifer, aquifer porosity, and determined the degree and depth of fine grained material in each stratigraphic layer. This very sophisticated log suite was also used to evaluate the degree of drilling fluid invasion and estimate the formation hydraulic yield (transmissivity) through the use of the Magnetic Resonance log and the Elemental Capture Spectroscopy log.

During the first phase of the drilling program, the wells drilled with flooded reverse circulation with a 41 to 46 cm (16 to 18-inch) minimum bit size, required an eight inch diameter mud rotary exploratory well drilled within 6 meters (20-feet) of the final ASR well. These wells were used for geophysical logging and lithologic control at these sites to ensure accuracy of the logs and correlation to other sites. Many of these geophysical logging holes were completed as monitoring wells while others were abandoned with grout.

2.10 ACIDIZATION AND CHEMICAL TREATMENTS

In the first phase of well installation bentonite clays were used extensively in the drilling fluid control programs, resulting in significant formation invasion. In the mud rotary applications, numerous rounds of mud dispersant chemicals were added to the wells and developed out early in the development program to dislodge and remove the drilling mud. In all cases, the process worked well and produced wells of acceptable quality and efficiency. For the flooded reverse circulation drilled wells, numerous applications of mud dispersants and acids were attempted in the wells to remove invaded material and restore performance. Chemical treatments were repeated during development on low performing wells. The approach resulted in numerous low efficiency wells and a few unacceptable wells. If the formation invasion is not stopped at the borehole wall, then re-circulated solids will be allowed to penetrate deep into the formation to distances that are not reachable with conventional mechanical or chemical development techniques.

During the installation of the second phase wells, a high quality drilling fluid program was maintained, minimizing formation invasion. During gravel pack installation, chlorine and clay dispersants were added. Additional clay dispersant chemicals were added in the first few days of development. This approach produced very clean results. The wells easily met the contact efficiency requirements.

2.11 WELL DEVELOPMENT

The wells were drilled on all accounts with bentonite and water, requiring some combination of chemical and mechanical development techniques to bring the wells into service. By contract, the driller was to deliver a well of 75% or better efficiency. The drilling method and development approach was left to the drilling contractor. In all cases, the wells were developed by swab and air lifting to remove the heavy materials. Mud dispersant chemicals were used to aid in drill fluid removal. This was followed by high capacity surge pumping. The wells were developed until pre-test specific capacity checks met the specification. In several cases, repeated development and chemical treatments were applied to improve performance.

The wells were pumped nearly 5,450 m^3/d (1,000 gpm) over the reasonable design pumping rate during development. During testing, the wells produced only trace amounts of sand, even at the high rates. This indicates good formation stability and the correct selection of the well screen slot and gravel pack size for the hydrogeologic setting. In a few cases, the wells sat for a few months before well development was even started. This stand time with drill fluids resulted in lower than normal efficiency wells.

It should be noted that the wells were developed for production service, leaving the potential for grain reversal during the recharge operations. The development programs did not use recharge simulations to develop the wells for recharge action. This is apparent during the three cycle recharge events. On each recharge cycle, the injection hydraulics improved (less rise required for the rate). The recovery pumping is clearing the recharge clogging material, resulting in increased performance on the next startup. The forces of these tests are providing development for recharge operations. With time, the recharge and backwash operation will complete the development for injection.

2.12 PUMP TESTS

During the feasibility study period, many assumptions were made concerning the yield of the Carrizo Aquifer, though no full scale test wells were installed to validate the proposed ASR well design. This resulted in the installation of seventeen undersized ASR wells during the first phase. During phase two construction in 2006, a dozen 51 cm (20-inch) wells were installed between the existing 41 cm (16-inch) wells. The 51 cm (20-inch) wells netted an average 5,450 m^3/d (1,000 gpm) greater yield than the smaller wells.

Tests on the wells included high speed gyroscopic plumbness and alignment tests, step drawdown pumping tests, continuous rate pumping tests, and three unique recharge and recovery tests. The tests provided the specific capacity, average transmissivity, storage coefficient, and well efficiency for each pumping well. The cycle tests provided evaluation of the recharge hydraulic response (specific capacity) and the average mounding slope before clogging sets in. The recovery pumping cleared almost all notable recharge clogging. The subsequent recharge test showed that the clogging was removed and the recharge specific capacity improved.

Pump test results for the first 17 wells had mixed results due to poor drilling and construction practices. Pumping ranged from 2,725 to 13,625 m^3/d (500 to 2,500 gpm). Pumping specific capacities ranged from 411 to 1,573 m^3/d per meter (23 to 88 gpm/ft). The average range is about 858 to 1,109 m^3/d per meter (48 to 62 gpm/ft) with an average of 1,001 m^3/d per meter (56 gpm/ft) (CH2M HILL, 2003). Test results vary due to high formation invasion, incomplete development, enlarged gravel envelopes, and cement in screen. Pumping tests for the final 51 cm (20-inch) wells ranged from 2,725 to 19,075 m^3/d (500 to 3,500 gpm). These step drawdown tests represent the baseline performance for each well under pumping conditions.

Three cycle tests were conducted in two of the 41 cm (16-inch) wells primarily to evaluate whether the process of recharge and recovery of the Edwards Aquifer source water might impact potable water quality. Each cycle contained increasingly larger volumes ranging from approximately 22,710 cubic meters (6 million gallons) to over 83,270 cubic meters (22 million gallons). The mixing stand times increased from immediate turn-around, to 7 days, and then to 19 days (CH2M HILL, 2005). Figure 4 illustrates the injection cycle trend for ASR-15 over cycle test 3. Note the nice straight mounding slope between minute 300 and 2,000. Following the 2,000 minute mark, minor clogging starts to accumulate, causing detectible changes in the water level trend, forcing an upward climb.

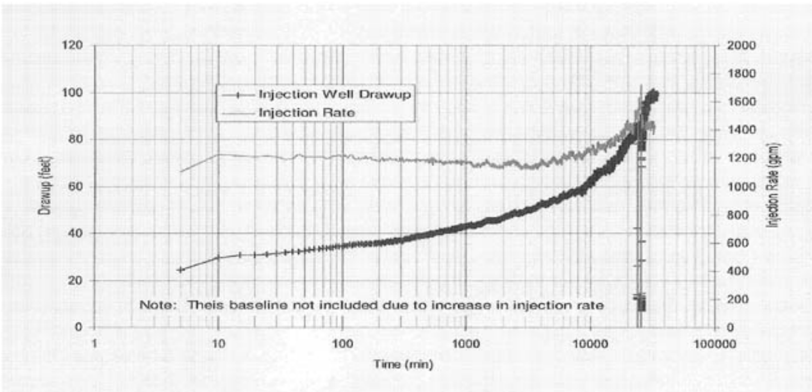

Figure 4: Cycle 3 Injection Test ASR-15 (CH2M HILL, 2005)

ASR wells are typically backflushed regularly to remove particulates that cause clogging. Typical backflushing frequencies are in the range of every few days to every few weeks. Duration of backflushing is typically from ten minutes to about two hours, depending at least in part upon materials of well construction. The Twin Oak ASR wells typically backflush on a monthly basis to maintain water level rise and efficiency.

2.13 PUMP AND EQUIPMENT DESIGN

The wells were equipped with vertical line shaft turbine pumps fitted with reverse rotation ratchets to allow recharge through the pump bowls. Five wells are equipped with downhole flow control valves to allow recharge at greater rates than that achieved through the pump bowls. The wellheads are equipped with pressure gauges, vents, and water level transducers to support a fully automated operation. The above ground piping layout is one of the best observed. The distribution system water line enters the site to an above ground piping manifold with four ports controlled by pressure operated globe valves and butterfly valves. The ports are for recovery pumping from well, recharge supply to well, pipeline pre-startup purging to waste, and pressure blow-off to waste. The well collector line also contains a waste discharge line and control valve. The wellhead and associated conduits are fully sealed to allow full well bore pressure recharge. In support of this operation, an additional by-pass pipe and isolation valve has been placed on the flow line enabling the recharge water to be re-directed to the well casing annular space rather than down the pump column. This would avoid the head losses of injection through the pump string, allowing higher rates. Recharge may occur down the pump column and also down the casing annulus, thereby maximizing recharge rates.

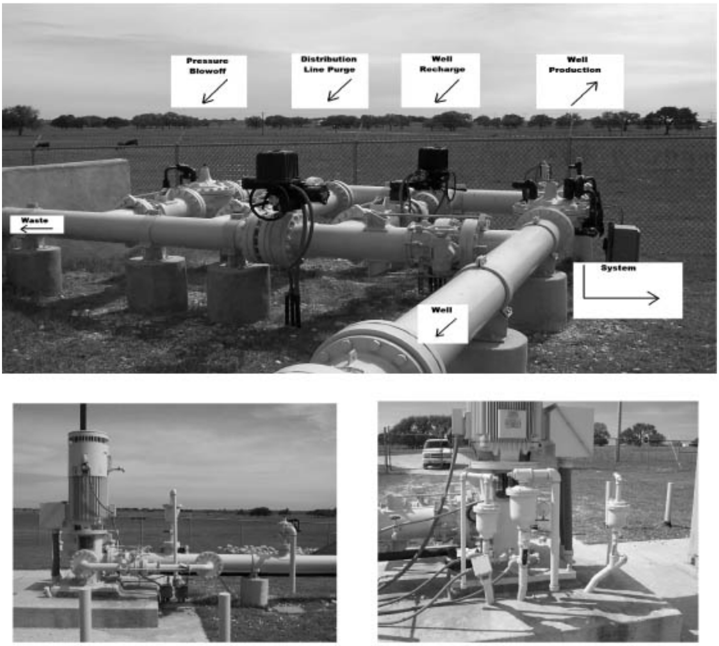

Figure 5: Well Equipping (Morris, 2006)

Unlike a typical potable water system where the tanks and reservoirs provide pressure stabilization, the Twin Oaks wellfield is a closed system. Pressure spikes from the startup and stopping of recharge are relieved to waste at the well site relief valve. If the distribution line contains undesirable water quality, the system operator at the command center has the ability to purge the distribution system water to waste until it is acceptable for recharge. The well is then automatically placed into the injection mode. Recharge startup forces the column air into the formation, contributing to well clogging. A positive back-pressure is soon achieved and maintained on the column throughout the recharge operation. If a downhole control valve is used to increase the base recharge rate during startup of recharge and to prevent air entry into the well, the valve is manually opened in the field to the desired position and locked into place. The start and stop control is by the surface globe valve at the manifold's recharge port. Recharge is stopped once a month to back-flush the wells and to check the operation of the pumping equipment. With the routine pumping activity, the wells are maintained in a high efficiency state. Very little to no declines in recharge performance have been observed (SAWS, 2005).

Conversion between recharge and production is an automated routine. The system control calls for a stop of recharge by closing the supply globe valve. After a stabilization period, the waste valve is opened and the pump is called to start. After discharging to waste for a predetermined time, the logic control switches the flow to the distribution system line, or stops the pump and returns the site back to recharge. To date, most but not all of the recovery pumping has been limited to monthly preventive maintenance events and original facility testing. In the first two years of operation, recharge water was available to sustain the plant in the recharge mode, banking over 24.7 million cubic meters (20,000 AF). This was subsequently increased to as much as 75.3 million cubic meters (61,000 AF). More recent recovery during a drought has reduced this to 62.9 million cubic meters (51,000 AF) as of June 2009. Other than disinfection, the recovered water has not required retreatment.

The facility is in the early stages of developing long-term operational practices, purge durations, equipment calibration procedures, and field monitoring programs aimed at wellbore performance (see Figure 5). Unlike typical utility-based systems that add a few wells at a time, allowing fine tuning of the operations, this facility is a stand-alone 227,100 m^3/d (60 MGD) recharge, recovery, and treatment facility that is essentially completely new.

2.14 INSTALLATION CHALLENGES AND CRITICAL DECISIONS

2.14.1 Groundwater Recharge Feasibility

The location selected for aquifer storage does not need to be in the local community, or even within the local municipal supply aquifer. The location could be in some other basin, valley, or hydrologic system connected via pump stations and pipes. In this case, the selected aquifer was located 48.3 kilometers (30-miles) south of the urban center in a completely separate hydrologic system. As for the water quality concern, the existing water quality of the storage aquifer does not need to be pristine for potable water storage. The selected aquifer has a pH of 5.5 with high iron and manganese content. Thus, an aquifer with non-compatible water quality is quite acceptable for ASR. However, treatment may be required if an adequate buffer zone is not formed and maintained at each ASR well.

Stakeholders' involvement is important. An agreement was reached with local water agencies representing domestic and agricultural interests to implement mitigation measures to minimize adverse impacts to residents during the operation of the ASR facility. Such an agreement should also account for water ownership and benefits of the ASR program to the aquifer. Texas law and groundwater regulations do not provide any protection of the stored water. They do not prevent the local groundwater producers on adjacent lands from drawing from the fresh water bubble. So facility design must account, through modeling and operational monitoring, for the maximum concentrated pumping effort that may occur on the facility border by others before the fresh water bubble may be drawn into their production. Groundwater barriers using several ASR wells on their critical border can be formed with the intent of injecting native water. This will feed the local over-pumping with native water, and force the fresh water bubble back to the recovery wellfield.

In this project, preliminary planning and designs included property and pipelines sized for the full scale operation. SAWS purchased enough land upfront to provide them 7.8 million cubic meters (6,400 AF) per year water rights from the native aquifer, and space for 29 ASR wells and three Carrizo extraction wells. The delivery, treatment, and pumping equipment were sized for the maximum 227,100 m^3/d (60 MGD) operation. A treatment plant was built to ensure water developed from the wells meets the normal water quality of the distribution system.

2.14.2 Well Installation

Most wells were drilled with flooded reverse circulation rotary methods. Eight out of twenty of these wells resulted in low efficiency due to poor fluid control programs during drilling. The remainder of the reverse drilled wells were drilled with a bentonite-polymer drilling fluid resulting in very high efficiency wells. Eight of the wells were drilled direct mud rotary. Following the use of mud dispersant chemicals, the mud rotary wells met the required efficiency requirements.

All 29 wells were constructed with blank casing that is composed of mild steel with epoxy coating. No issues have been noted to date, though the wells are only a few years old. The well screens are 304 stainless steel wire-wrap surrounded by a silica gravel envelope. Pump columns are mild steel with no special protection for the well casing epoxy coating during installation.

2.14.3 Well Equipping

The wells are equipped with several manual and electronic devices. Gauges and transducers monitor the wellhead and system pressure. Bi-directional electro-magnetic flow meters are used. The boreholes are equipped with water level transducers but with no automated air release valves. Most of the valves have position indicators. All of this data, along with motor performance data, is being transmitted back to the command center and alarmed for operation.

Borehole flow control valves are equipped in five of the wells. They are used to provide recharge flow at rates higher than that going through the pump bowls and to prevent air entry into the well screens. The valves are operated manually to a set orifice size when used. The wellhead and associated conduits are fully sealed to allow full well bore pressure recharge. In support of this operation, an additional by-pass pipe and isolation valve has been placed on the flow line, enabling the recharge water to be directed to the well casing annular space rather than, or in addition to, down the pump column. This would minimize the effect of head losses of injection through the pump string, allowing higher recharge rates.

The well site piping configuration is among the best for larger ASR wellfields. The system line enters the site to an above ground manifold with four ports controlled by pressure operated globe valves and butterfly valves. The ports are for recovery pumping, recharge supply, pipeline pre-startup purging, and pressure blow-off. The well collector line also contains a waste discharge line and control valve. Large "Y" strainers have been placed on the well recharge lines to prevent the introduction of anything larger than 1/8-inch into the pump bowls.

2.14.4 ASR Operations

The wells are designed to recharge at a rate lower than the production rate to allow pumping purge forces to be greater than injection forces. This facilitates back-flush development of the well with the existing equipment as opposed to rehabilitating the well with a higher capacity development pump. The system is fully monitored and controlled by SCADA. The system operates by a "scheduler program" placing the most efficient unit or well in a specific location, brought online first and least needed unit last. All wells are equipped with recovery pumps. The pumps are operated once a month for preventive maintenance, vibration, and performance checks to ensure operation when needed. This purge cycle helps back-flush the wells and maintain efficiency. A heavy reliance on instrumentation and automated operation must be supported by real-time data collection of the parameters. Common operation parameters such as discharge-to-waste durations change with operation of ASR wells. Waste discharge quality (sand and turbidity) and duration must be frequently checked to ensure proper flush times to remove the harmful debris. Premature shut-down during wasting could cause well or pump damage.

2.15 CONCLUSIONS

The San Antonio urban area needed to make better use of the water available within the local Edwards Aquifer system, diverting water during spring runoff and storing it for use during peak summer days and periodic droughts. Over 10 years of feasibility studies, tests, and construction were required to develop the current Twin Oaks ASR facility. This started with evaluation of six potential sites in five unique geologic settings. Considering pre- and post- water treatment costs, delivery options, and infrastructure development costs, the option to utilize the Carrizo Aquifer 48.3 kilometers (30 miles) south of San Antonio was selected for further study. Test wells and soil borings indicated the site was suitable for a large scale recharge and recovery program with minimal risk of geochemical reactions and water loss. The recovered water quality may at times include mixtures of Carrizo Aquifer water with the recovered drinking water from the Edwards Aquifer, requiring that the blend be treated to remove iron and manganese. Yet the recovered water from the ASR wells does not require retreatment, other than disinfection to restore the chlorine residual.

Without a test well program, SAWS moved immediately into installing seventeen wells cased at 41 cm (16-inches). These wells yielded production flows of 8,175 to 13,625 m^3/d (1,500 to 2,500 gpm) with recharge abilities averaging 6,540 to 7,630 m^3/d (1,200 to 1,400 gpm). The well yields are limited by the casing size. In the second phase of construction in 2006, the new wells were installed with 20-inch casing. The well yields increased nearly 5,450 m^3/d (1,000 gpm) per well. A total of 29 wells were installed at the facility up to depths of 213 meters (700 feet). The wells were constructed with stainless steel wire wrapped well screen with a 0.076 to 0.114 cm (0.030 to 0.045-inch) slot opening. The blank casing sections utilized mild steel with a 9-mil epoxy coating on the inside. Threaded casing was used to prevent epoxy damage from casing welds. To date, the epoxy casing has not posed any issues with well operation or maintenance.

Poor fluid control during drilling contributed to several of the wells being low in efficiency. High quality, bentonite and polymer fluid programs proved best to minimize formation invasion. Extensive development of the wells with chemical mud dispersants, acids, and surge pumping prepared the wells for operational service. The wells were tested with step drawdown pumping tests, continuous rate pumping tests, and three multi-day injection cycle tests. The testing indicated a wide variance in well efficiencies attributed to well installation methods. The cycle testing indicated that 80% of the water could be recovered before mixing of the native Carrizo Aquifer water became a concern. With development and maintenance of a buffer zone around each well, higher recovery efficiencies approaching 100% should be attainable in subsequent cycles, depending upon the rate of groundwater movement to offsite wells. No adverse chemical reactions or exceedences in water quality standards were detected throughout the testing.

The operation is currently focused on water banking with little recovery to date. This has focused concerns on equipment maintenance programs to ensure the readiness of the facility and keeping track of where the banked water is going during storage. In the lifespan of the facility, the program is in the early stages of exploring the limits of operation and storage. Until several complete cycles of recharge and recovery have been completed, the operation will be in a learning mode with potential operation and design adjustments to enhance performance and ensure sustainability.

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CHAPTER 3

Environmental Assessment of Using Stone Quarries as Part of an Integrative Water Supply System in Fast Growing Urban Regions

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ABSTRACT: Global climate change and cyclical droughts threaten water supplies in many cities in the United States and across the world. At the same time, rapid population growth has occurred in many urban regions, bringing additional demand for adequate quantity and quality of drinking water. Development of more water-supply reservoirs is a long-term solution to the threats of droughts, climate change, and population growth. One group of potential sites, namely crushed stone quarries, holds promise for expanding the reservoir capacity in water supply systems. Inspired by the past and future trend of global change and the need to generate adaptive water resources management strategies, this study demonstrates a conceptual evaluative framework with a preliminary assessment of the feasibility of expanding regional water supply systems using abandoned and active stone quarries. Whether or not a particular quarry can be used as a water supply reservoir depends on many factors, such as the size of the quarry and the volume of water that can be stored, the location of the quarry and its distance to the city or community that needs water supply, the source of water that can be diverted into the quarry, the surface runoff from the watershed to the quarry, the evaporative loss of water in the quarry, the groundwater input to the quarry, and the possible contamination from surface and groundwater flows that can affect public health. Our case study in the southeastern United States demonstrates that water storage and aquifer recharge by stone quarry reservoirs must be achieved with caution since water quality may be changed by biogeochemical reactions within aquifer materials. Recharge water may be warmer or cooler than native groundwater and it may dissolve certain minerals while allowing others to precipitate. A quarry reservoir can become thermally stratified during summer if it is deep enough, and this stratification can lead to oxygen depletion in the bottom waters, which may then require hypolimnetic oxygenation (aeration) to improve the water quality.

3.1 INTRODUCTION

The scientific community has reached a clear consensus that one of the most certain outcomes of global climate change is temperature increase due to rising atmospheric concentrations of carbon dioxide. This will lead to an increase of water loss by evaporation. Such loss, combined with cyclical droughts such as the current drought in the southeastern United States, could pose dire threats to water supplies in fast-growing urban regions. One model (Burke et al., 2006) suggests that "if global warming–related precipitation changes continue apace, the percentage of the Earth's surface in severe drought could rise from the current 3% to 30% by 2100". Other researchers "project that there is going to be competition for water in the future. We want to take a look over the horizon and see what kind of innovative approaches we need to survive the drought crisis (Manuel, 2008). The first common practice for solving the shortage of water is to develop and implement various policies and efforts to conserve water. These conservation practices can be very effective for short periods. Yet the development of sources and/or storage of a water supply that can carry a community through extended periods of low streamflow are needed for a long-term solution to the threats of droughts, climate change, and population growth.

In recent years, the southeast United States has been the most rain-starved region of the country. Although there currently is no immediate public health threat posed by the southeastern drought, it does pose significant challenge and struggle to policy makers and utility companies or authorities to maintain an adequate supply of potable water. At a time when there is a need for more water-supply reservoirs, the supply of natural sites appears to be quite limited. Most economically attractive and environmentally acceptable natural sites for storage reservoir development in the USA have already been developed. "One group of unnatural sites, namely crushed stone quarries, holds some promise for expanding reservoir capacity" (Moreau, 2007).

Inspired by the past and future trend of global change and the need to generate adaptive water resources management strategies, the objective of this study is to conduct a preliminary impact assessment for evaluating the feasibility of expanding the water supply system using abandoned and active stone quarries at the regional basis. In this writing, we develop a conceptual and numerical framework for understanding, quantifying, and modeling the key attributes and criteria that might affect the decision making of using stone quarries as water-supply reservoirs under normal operation and/or disaster conditions. Impact assessment associated with surface water and groundwater is also emphasized to minimize the risk and increase the reliability of the civil infrastructures.

3.2 LITERATURE REVIEW

3.2.1 Water Supply and Drought

Provision of an adequate quantity of water has been a matter of concern since the beginning of civilization. Even in ancient cities, local supplies were often inadequate and aqueducts were built to convey water from distant sources (McGhee and Steel, 1991). The National Academy of Engineering in the USA has developed 14 grand challenges for engineering, and one of them is to provide access of clean water: "The world's water supplies are facing new threats; affordable, advanced technologies could make a difference for millions of people around the world." The average daily per capita water consumption in American cities varies from 130 to 2000 liters (35 to 530 gallons) (McGhee and Steel 1991). Variation in water consumption depends upon many factors, such as the size of the community, presence of industries, quality of the water, cost, available hydraulic pressure to deliver water, climate, characteristics of the population, and the efficiency with which the system is maintained. The sources of water supply include groundwater and surface water supplies. Surface water supplies require the construction of impoundments (storage reservoirs) to meet demand during periods of low river flow.

Water consumption estimates are typically based on population projections, and rapid growth of population has occurred in many urban regions in the USA and over the world. For example, Baldwin County in Alabama had a population of 78,556 in 1980, which increased to 140,415 in 2000, and to 171,769 in 2007 (about a 30% increase per decade). The Mecklenburg-Union-Cabarrus County area in North Carolina has grown by almost 160,000 over the five years and the state of North Carolina as a whole has increased by nearly 640,000 since the census of 2000 (Moreau, 2007). Moreau (2007) states that most population increases have come in areas where surface water is the primary supply option. To develop surface water supplies to meet these increased demands, reservoirs must be used to hold sufficient volumes that will carry a community through extended periods of low streamflow (Moreau, 2007).

In this century persistent droughts have plagued the western United States. In the past few years, drought impacts have strained big metropolitan water supplies in Maryland and the Chesapeake Bay in 2001 through 2002, Lake Mead in Las Vegas in 2000 through 2004, the Peace River and Lake Okeechobee in south Florida in 2006, and Lake Lanier in Atlanta in 2007. The last affected the water resources distribution in three states - Alabama, Florida, and Georgia. In February 2008, eight major US water agencies united to form the Water Utility Climate Alliance, acknowledging that plans for future investment in water infrastructure must be made to accommodate climate change projections (SFPUC, 2008). The U.S. Drought Monitor on May 13, 2008 (Figure 3.1) showed that about 18% of the Southeast remains either in severe or extreme drought even after most of the region had received 10 to 12 inches of rain in the previous three months. Forecasters say it will take monumental storms to ease drought areas.

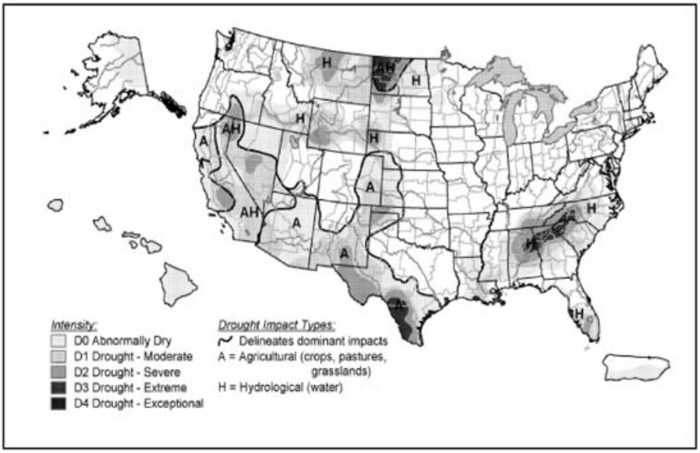

Figure 3.1: U.S. Drought Monitor for May 13, 2008 (courtesy of National Drought Mitigation Center, University of Nebraska – Lincoln)

3.2.2 Stone Quarries

Aggregates are natural materials mined either from gravel pits or from quarries. Generally, "pits" produce sand and gravel and "quarries" produce crushed hard rock. Crushed-stone mining includes four possible important components: (1) stripping - removal of the topsoil that covers the mineral deposit; (2) excavation - breaking of the stone away from the quarry wall and also breaking of the stone into smaller pieces by the use of explosives; (3) processing - operations at the processing plant such as crushing, screening, size classification, material handling, and storage; and (4) trucking - transport of crushed stone to various construction sites. Major rock types processed by the crushed stone industry include limestone, granite, dolomite, sandstone, and quartzite. For example, the limestone mined in Florida is typically extracted by strip mining to be crushed for use in road constructions. Explosives are frequently used to fragment the rock for extraction. Most of these quarry mines were dug below the local groundwater table, sometimes exceeding 304 meters (1,000 feet) beneath the ground surface. A quarry lake forms and develops over time when groundwater, surface water and other post-mining drainage accumulates inside an inactive (abandoned) open quarry after dewatering stops. Two examples of quarry lakes are shown in Figure 3.2. There are many potential end uses for an exhausted quarry. To slope the banks of an exhausted quarry and revegetate the area is a common reclamation practice. It is, after all, the least expensive way to satisfy government regulations. Because real estate prices are rising, more and more crushed-rock producers have reclaimed quarries for commercial and residential building and recreational development (from http://www.Pitandquarry.com/).

Figure 3.2: Abandoned quarries in Newberry, near Gainesville, Florida (left) and in Wake County, North Carolina (right)

Data published by the United States Geological Survey (USGS) indicate that a total of 1.72 billion metric tons of crushed stone valued at $13.8 billion was reported to be produced in the United States in 2006 by 1,367 companies with 3,212 operations and 3,358 active quarries (USGS, 2006). The 10 leading producing states were, in descending order of tonnage, Texas, Florida, Pennsylvania, Georgia, Missouri, North Carolina, Illinois, Virginia, Ohio and Tennessee, together accounting for 53% of the total output of crushed stone in the US. The four leading producers of crushed stone are Vulcan Materials Company, Martin Marietta Aggregates, Oldcastle, Inc./Materials Group, and Hanson Building Materials American, Inc. The Vulcan Materials Company had 183 active quarries and 87 sale yards in 2006, and 17 of its active quarries were located in Alabama. Alabama produces about 50 million metric tons of crushed stone every year, with an estimated commercial value of 400 million dollars. There are about 56 active quarries in Alabama. Table 3.1, developed by David H. Moreau in 2007, shows the number of stone quarries by permit status in North Carolina: 126 active permits, 22 inactive permits, and 43 that have been released from permits. Active stone quarry sites range from just a few acres up to 2,002 acres. Some of these quarries could provide excellent opportunities for water supply reservoirs.

Table 3.1: Number of stone quarries in North Carolina by permit status (adopted from Moreau, 2007)

Size in acres	Active	Inactive	Released
Under 100	35	12	31
100-249	41	5	8
250-499	50	5	2
500-999	13	0	2
Over 1000	7	0	0

3.2.3 Current Practices – Stone Quarries Reservoirs

One interesting case of using stone quarries as water-supply reservoirs is the Orange Water and Sewer Authority (OWASA) in North Carolina. Figure 3.3 shows the water supply reservoir system for OWASA. During the 1980s, when OWASA was going through lengthy proceedings to build the Cane Creek Reservoir, storage was added by purchasing an abandoned stone quarry (#1 in Figure 3.3) along the route of a pipeline from Cane Creek Reservoir to University Lake. The quarry holds about 200 million gallons, which was about a 30-day supply at the time when it was put in service (Moreau, 2007). Because there is a very small drainage area that contributes runoff to the quarry, facilities were put in place to refill it from the pipeline as needed during periods of high streamflow into Cane Creek Reservoir. Since the purchase of the first quarry, OWASA has purchased an additional nearby actively mined quarry (#2 in Figure 3.3). It was purchased with a leaseback arrangement that allows the former owner to continue mining operations for a specified time period. When mining operations are complete in 2030, it is anticipated that the quarry will hold 2,400-3,000 million gallons, which is about the same size as the Cane Creek Reservoir. The second quarry will be connected to the first quarry, and plans are to fill the system with pumping from either Cane Creek Reservoir or University Lake (Moreau, 2007).

A second case is the City of Durham in North Carolina, which has a quarry under active development for off-stream storage. Teer Quarry, where active mining ceased more than 10 years ago, is located in Mill Grove of Durham County, about 182.8 meters (600 ft) south of Eno River. There are three options under consideration for filling Teer Quarry by diversions from the Eno River when flow in the river is above levels that would cause environmental damage to the stream (Moreau, 2007). One option is to fill the quarry to a level controlled by gravity flow from the Eno River. The other two options require pumping and the construction of dikes (Moreau, 2007). Volumes for the options range from 4.98 to 18.71 billion liters (1.32 to 1.95 billion gallons) of water. The quarry would add 28.3 to 44.6 million liters per day (7.5 to 11.8 million gallons per day) depending on the storage option and pumping rate selected. After a long period of drought in North Carolina in January of 2008, when Durham's two primary reservoirs (i.e., Lake Michie and the Little River Reservoir) had less than 40 days of quality drinking water left, a permanent pump was installed and Teer Quarry supplied about 1512 million liters (400 million gallons) of water to Durham. The pump was turned off on February 22, 2008, when the city had 180 days of available drinking water from the two main reservoirs. These projects demonstrated that using these quarries as reservoirs makes an asset out of what might otherwise be an environmental liability for crushed stone producers.

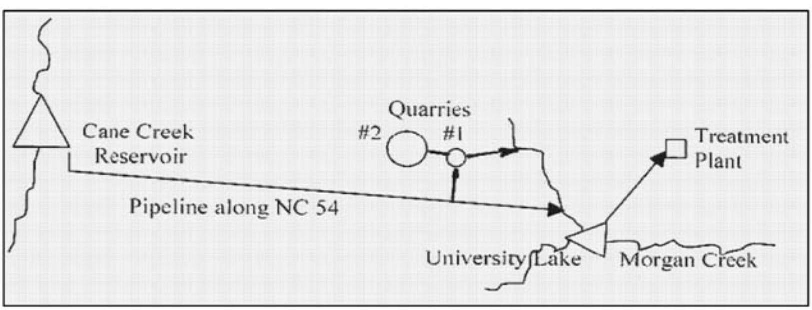

Figure 3.3: Water supply reservoir system for OWASA (Moreau, 2007)

In Wake County of North Carolina, abandoned quarries (see Fig. 3.2) have been identified that contain billions of gallons of water that could be tapped to augment the supply of available drinking water in the future. Johnston County in North Carolina plans to spend $1.2 million to purchase a 40.46 ha (100-acre) property just north of Princeton, NC, that has a 6.47 ha (16-acre) quarry with a 1890 million liters (500 million gallon) capacity, but it could be five to seven years before the county starts drawing water from the quarry. Johnston County is also negotiating with the owners of a larger active quarry nearby that would hold over 26.6 billion liters (7 billion gallons) of water but that will probably continue to be mined for another 25 years (from http://www.newsobserver.com/weather/drought/story/ 840422.html, accessed on August 26, 2008).

3.3 FEASIBILITY OF USING STONE QUARRIES AS WATER SUPPLY RESERVOIRS

Most drinking water in the Southeast is stored in man-made reservoirs, such as Falls Lake and Lake Jordan for the city of Durham in North Carolina. "Abandoned quarries have been largely overlooked as storage sites until recently", as stated by Dr. David H. Moreau, Director of the Water Resources Research Institute of The University of North Carolina. It is worth mentioning that some abandoned quarries work better than others for storing water. Any site, according to Dr. Moreau, would need to be examined for contaminants that could threaten public health. Another factor to consider is the cost of moving water from the quarry to treatment plants and distribution pipes.

3.3.1 General Background

Potential climate change and cyclical droughts pose threats to water supplies for many cities in the United States. Water supply systems are "sociotechnical" systems with strong links to society (Grigg, 1996). For instance, the Southeast drought has already had not only serious economic consequences (losses of more than $1.3 billion in 2007), but also political consequences, pitting downstream and upstream users (i.e., three states) against one another. The first common practice for solving water shortage is to develop and implement various polices and efforts to conserve water. These conservation practices can be very effective for short periods. Yet the development of sources and/or storage of water supply that can carry a community through extended periods of low streamflow could be an important long-term alternative. The USGS Mineral Industry Surveys show that there are 3,358 active quarries for crushed stone production and there are many abandoned or inactive quarries in the United States. "What is the feasibility of using active and abandoned quarries for water supply development and solving current and future water supply crisis?" is a question that has yet to be studied holistically.

Whether or not a particular abandoned or active quarry can be used as a water supply reservoir depends on many different factors. Some important factors are the size of the quarry and the volume of water that can be stored, the location of the quarry and its distance to the city or community that needs water supply, the source of water that can be diverted into the quarry, surface runoff input from the watershed to the quarry, groundwater input to the quarry, evaporative loss of water in the quarry, etc. To face the future challenges, water supply planning requires us to not only identify abandoned quarries that can be possibly developed as storage reservoirs in the near future but also to consider and identify active quarries for long-term future development, especially in karst regions where the hydrogeological environment is challenging in terms of groundwater and surface management because of rapid infiltration rate (see Figure 3.4, where karst terrain covers nearly 25 percent of the United States). In the Southeast the karst geology might strongly affect the decision making process of using stone quarries as alternative water storage. In considering the planning, design, and expansion of a water supply system, many competing issues must be addressed and the competing demands must be balanced within the legal and administrative framework (Maidment, 1993). Modifications to the water supply system also depend on the extent to which the alternative stone quarries are integrated into the community's water resources.

3.3.2 Formation of a Quarry Lake

What is a quarry lake? It begins as an open pit or quarry that must be dewatered during mining if it was dug below the local groundwater table. Once mining stops and dewatering pumps are removed, groundwater begins to seep into the quarry, and surface runoff from rainfall also flows from the surrounding watershed into the quarry. Some pit or quarry lakes maintain a depression in the water table and soluble metals can accumulate inside and increase in concentration over time when local evaporation and transpiration is greater than local precipitation. Some pit lakes have ground and surface water outflows when local evaporation and transpiration is less than local precipitation. After the formation of a quarry lake, residential communities and urbanization may develop alongshore the beautiful quarry lake, as has occurred, for example, in the quarry lake communities at Greenspring of Baltimore County, MD; Fremont, CA; Amherst, OH; Monroe, MI, etc. Recently, some quarry lakes were converted to water storage sites and filled by diverting water from nearby rivers, streams, and reservoirs when contributions from groundwater and surface runoff were limited. Whether or not surface runoff or groundwater flow from the surrounding watershed can provide adequate source water during wet seasons for the quarry reservoir is one of the factors limiting the inclusion of a quarry into an integrative water supply. Another consideration is that diverting water from a nearby stream to a quarry must satisfy aquatic habitat and in-stream flow requirements to maintain a healthy ecosystem downstream of the diverting point.

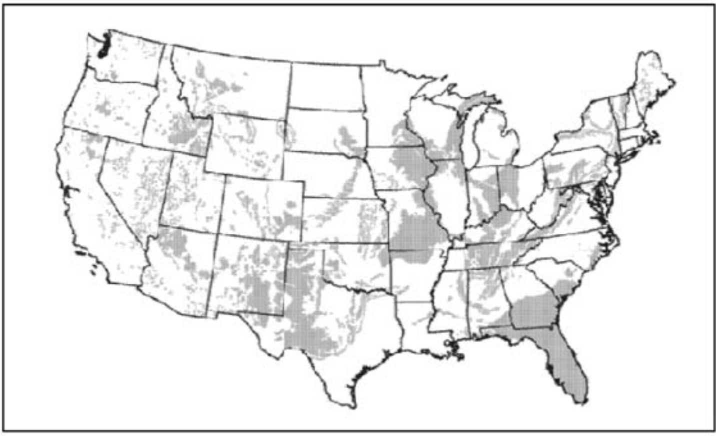

Figure 3.4: The karst regions (green areas) of the contiguous United States (Courtesy of U.S. Geological Survey, Davies et al. 1984)

3.3.3 Surface Water Input to Quarry Lakes

Regional precipitation variation and extreme drought events are important to water supply development for fast growing urban regions. Surface runoff input to a quarry lake depends on the size of the watershed surrounding the lake, geological conditions, and regional precipitation variation. Using the lake as an outlet, the contributing watershed can be delineated from contour maps, field surveys, aerial photographs, and digital elevation models. If the watershed area contributing to the lake is limited, then surface runoff to fill the lake will be limited. Rainfall is the driving force and the key parameter controlling surface water input to a quarry lake. Figure 5 shows variation in mean annual precipitation over the contiguous U.S. (based on observation data from 1899 to 1939); annual precipitation is on the order of 50 inches or more in the southeast United Sates. More recent and long-term meteorological data from 209 weather stations are available from the National Weather Service (NWS) of the National Oceanic and Atmospheric Administration (NOAA), and this allows us to examine regional precipitation variation at any stone quarry site for potential water supply development.

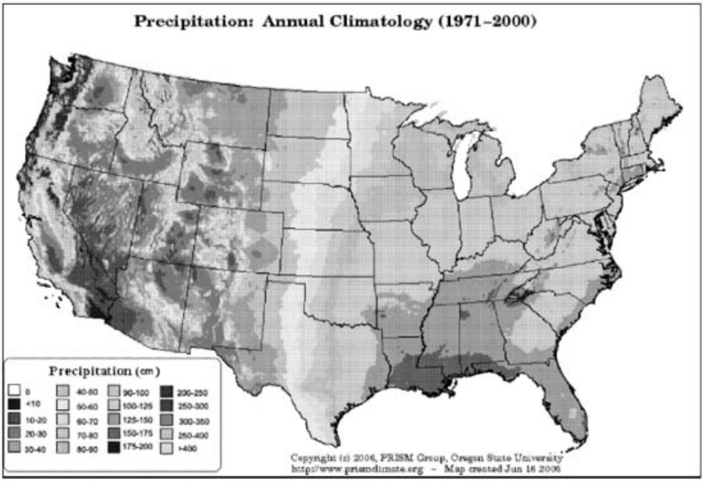

Figure 3.5: Mean annual precipitation in inches, 1971-2000 (Courtesy of Oregon State University).

3.3.4 Groundwater Hydrology of Quarry Lakes

For potential quarry lake sites in karst regions, it is important to identify land-use risks, which can be done with geophysical imaging and hydrological field studies. Imaging the subsurface structure of a quarry and related geologic features such as fractures, solution-generated conduits, and evolving sinkholes is of both practical and academic interest. These geologic features serve to increase the hydraulic conductivity, either as sources of stored water or as preferential pathways for groundwater flow. Geophysical methods can successfully detect void features; previous research projects have used such methods as electrical resistivity and ground-penetrating radar for this purpose (Carpenter et al., 1998; Zhou et al., 2002; Kruse et al., 2006; van Schoor, 2002). Electrical resistivity anomalies associated with karst can be either positive or negative, depending on whether they are air-filled (high resistivity) or water/soil-filled (low resistivity). Knowledge of the water-table depth is thus critical for interpreting the sign of the geophysical anomalies. Multielectrode systems allow for true 3-D inversion of the acquired electrical data; however, it is essential that the survey and measurement design be targeted to the scale of the features expected at each potential quarry lake site. Geophysical imaging methods provide an effective means to locate and characterize important shallow geologic features that may control hydrological transport. For example, the presence of low-permeability layers below the quarry can restrict the leakage of water. The presence of impervious aquitard layers stratigraphically above and surrounding the quarry, however, may retard the artificial recharge process. In cases where such confining layers are absent, water can percolate down-dip in the direction of the hydraulic gradient and recharge the underlying aquifers or deep-seated fractured zone.

Knowledge of water-table depths not only provide constraints for geophysical data interpretation and hydrological modeling, but also marks the presence of areas with high hydrologic conductivity as characterized by troughs and low hydraulic gradients in the water table. Superimposing these data on a discharge hydrograph will provide insight into the internal structure and transportation characteristics of the aquifer (Ryan and Meiman, 1996). Multiple- well aquifer pumping tests (Tiedeman and Hsieh, 2001) can also be conducted near the quarry sites. For example, a hydraulic conductivity meter can be used to record the head changes during a pumping test over time. This aquifer test can reveal the degree of hydraulic connection between boreholes and yield valuable insights into the hydrological characteristics of rocks surrounding the quarry. The relationship between measured drawdown and distance from the pumped well can be compared to that theoretically observed during radial flow in an aquifer, as predicted by the Theis solution (Fetter, 2001). The hydraulic conductivity calculated for both the highly conductive and less-conductive parts of the rock represent high-conduit clusters and the low-conduit matrix, respectively. These field experiments allow the assessment of water-level change in response to quarry water replenishment; such data are important for recharge operation and water resources management. Both the hydrological and geophysical measurements are geared toward understanding how the geologic features of each candidate site might affect its suitability as a water-supply reservoir. A rating system based on this information can then be used to prioritize candidate quarry sites for water supply development.

3.3.5 Environmental Impacts of Quarry Lakes

Simulation models are designed and applied to answer a variety of science and engineering questions, support watershed planning and analysis, and support decision making activities in various areas. During the past thirty years, numerous watershed loading and receiving water quality models have been developed. These models generally fall into two categories: lumped parameter models and physics-based distributed models. In order to evaluate the development of stone quarries as water supply reservoirs, models of surface-water, the unsaturated zone, groundwater, and the watershed should be integrated to quantify water budgets, water quality, thermal stratification dynamics, geochemical evolution and consequences for quarry reservoirs under different environmental and climate scenarios. The following discussion integrates a watershed model (TOPMODEL), geo-environmental model (FRACFLOW), and lake-water-quality model (MINLAKE96) to assess remedial alternatives and integrative applications of stone quarries. The following discussion does not intimate that other watershed and receiving water quality models cannot be integrate; for example, WASH123D (Yeh et al., 2005; Yeh et al., 2006a) and BEST3D (Yeh et al., 2006b; Yeh et al., 2006c; Yeh et al., 2007; Shan et al., 2008) can be integrated as both linked and coupled models to form the unified modeling system for this type of study, but these models are not yet publicly available for research development and engineering applications. The Watershed Analysis Risk Management Framework (WARMF) (EPRI, 2001) provides a decision support system for analyzing watershed management strategies as well as calculating, allocating, and implementing total maximum daily loads (TMDLs). The WARMF integrates one- or two-dimensional reservoir water quality models with watershed and stream network models (both hydrology and water quality) but was designed and applied for large basins or watersheds (such as USGS 8 digit Hydrologic Unit Code watersheds), which may not be suitable for examining small stone quarries.

Watershed Modeling Analysis

A quarry under a water supply feasibility study is treated as an outlet or pour point of the surrounding watershed as simulated by TOPMODEL and connected to the simulation domain for a lake water quality model - MINLAKE96 (Fang and Stefan, 1996). TOPMODEL is a distributed hydrological model and can use a GIS interface (e.g., ArcMap) for the analysis and manipulation of spatial data (i.e., digital elevation topography, spatial distributions of soil characteristics such as hydraulic conductivity, depth to bedrock, depth of the AB soil horizon, and field capacity). Although the model was first developed by Beven and Kirkby (1979), the latest version reflects the current understanding of how precipitation moves over and through watersheds to become streamflow (Wolock, 1993). The model is designed to simulate hydrological fluxes of water (infiltration-excess overland flow, saturation overland flow, infiltration, exfiltration, subsurface flow, evapotranspiration, and channel routing) through a watershed and simulates the variable-source-area concept for streamflow generation. The algorithm may assess explicit groundwater/surface-water interactions by predicting the movement of the water table, which determines whether saturated land-surface areas may have the potential to produce saturation overland flow. It uses time series of precipitation and air temperature as model inputs, making it a suitable platform to examine climatic and drought scenarios on water budgets for quarry development. Consequently, TOPMODEL has been used in a variety of research areas of water resources engineering, such as topographic effects on streamflow (Beven and Wood, 1983; Beven et al., 1984; Kirkby, 1986), topographic effects on water quality (Wolock, 1988; Wolock et al., 1989; Wolock et al., 1990), and effects of climate change on hydrological processes (Wolock and Hornberger, 1991). Since it is a module-based modeling system with publicly available FORTRAN source code, it can be coupled with other modeling components and systems, such as the lake/reservoir water quality model MINLAKE96 (Fang and Stefan, 1996). The hydrographs generated by TOPMODEL at daily time steps are therefore compatible for model integration with a daily-time water quality model of lakes (MINLAKE96, coded in FORTRAN) and to assess water balances for a targeted watershed for water quarry development.

Lake Modeling Analysis

A verified, process-oriented, unsteady and one-dimensional (vertical) year-round lake water quality model (MINLAKE96) was used to study the impact of global climate change on water temperature, dissolved oxygen (DO), and fish habitat for small lakes in the contiguous U.S. (Fang and Stefan, 1999; Stefan et al., 2001). This model is a good candidate to simulate the continuous change of lake stratification and water quality in response to weather, inflow, outflow, and exchange processes at the sediment interface, and in-lake processes. The water quality parameters that can be modeled after model enhancements include temperature, up to three forms of algae (expressed as chlorophyll-a), available phosphorus, ammonia, nitrate-nitrite, detritus as biochemical oxygen demand (BOD), zooplankton, inorganic suspended sediment, and dissolved oxygen. The depth profile of water temperature is computed from a balance between incoming heat from solar and long-wave radiation and the outflow of heat through convection, evaporation, and back radiation. Water loss through evaporation and input from precipitation are included in the model. MINLAKE96 can simulate daily lake water temperature and DO profiles in a continuous mode for multiple years. Thermal stratification is modeled first before the transport and fate of contaminants are tracked. The model requires dividing a lake (e.g., a quarry reservoir) into a series of well-mixed horizontal water layers including a sediment layer below (Fang and Stefan, 1996, 1997; Stefan et al., 1998).

The model was developed for simulations in different types of lakes in a cold region (Minnesota) first and then extended to small lakes (up to 10 km^2 surface area) over the contiguous U.S.A. The model has linked calibration parameters and initial conditions to lake geometry and/or location. For example, the hypolimnetic eddy diffusion coefficient (K_z in cm^2 sec^{-1}) is a function of lake surface area (A_s in km^2) and stability frequency [$N^2 = -(\partial\rho/\partial z)(g/\rho)$ in sec^{-2}, where ρ is density of water, z is water depth, and g is acceleration of gravity] (Hondzo and Stefan, 1993):

$$K_z = 8.17\times10^{-4}(A_s)^{0.56}(N^2)^{-0.43} \quad (1)$$

The maximum vertical hypolimnetic eddy diffusivity, K_{zmax}, occurs when temperature profiles are under weakly stratified conditions that were defined as $N^2 = 7.0 \times 10^{-5}$ sec^{-2} (Riley and Stefan, 1988). The model includes a bulk mixed-layer model in response to wind and convection, and a wind sheltering coefficient adjusts the wind speed for fetch over the lake in the direction of wind and is used to compute turbulent kinetic energy from a wind speed that is typically measured at an off-site weather station at 10-m elevation. The wind sheltering coefficient, $W_{str} = 1.0 - \exp(-0.3 \times A_S)$, is set as a function of lake surface area based on model calibrations using various Minnesota lakes (Hondzo and Stefan, 1993).

The stratification of water temperature and dissolved oxygen in two hypothetical stone quarry lakes were simulated using MINLAKE96. The first lake simulated has a surface area of 1.7 km^2 (420.1 acre), maximum depth of 24 m (78.7 ft), and lake volume of 1.25x10^7 m^3 (3,302.3 MG). Based on the lake volume, this lake has similar size as the second stone quarry that OWASA purchased (Figure 3.3). Water temperature and dissolved oxygen concentrations from 1961 to 1995 were simulated using weather data at Raleigh, NC (more recent weather data can be obtained from NOAA). The numerical simulation model for daily DO profiles in a lake solves the one-dimensional, unsteady transport equation and the vertical DO profiles in the lake are computed from a balance between oxygen sources (surface reaeration and photosynthesis of phytoplankton) and oxygen sinks [sedimentary oxygen demand (SOD), biochemical oxygen demand (BOD), and algal respiration] (Stefan and Fang, 1994):

$$\frac{\partial C}{\partial t} = \frac{1}{A}\frac{\partial}{\partial z}(AK_z\frac{\partial C}{\partial z}) + P_{MAX}\theta_p^{T-20}MIN[L]Chla - \frac{S_b}{A}\frac{\partial A}{\partial z}\theta_s^{T-20}$$
$$- k_b\theta_b^{T-20}BOD - Y_{O2CH}k_r\theta_r^{T-20}Chla \quad (2)$$

where C(z, t) is the DO concentration in mg L^{-1} as a function of depth (z) and time (t), T(z, t) is the water temperature in °C, A(z) is the lake horizontal area in m^2 (change with depth z based on lake bathymetry), K_z(z, t) is the vertical turbulent diffusion coefficient in m^2 day^{-1}, S_b is the coefficient for SOD at 20°C in mg O_2 m^{-2} day^{-1}, P_{MAX} is the maximum specific oxygen production rate by photosynthesis at 20°C under saturating light conditions = 9.6 mg O_2 (mg Chla)$^{-1}$ day^{-1} according to experimental data by Megard et al. (1984), Min[L] is the light limitation determined by Haldane kinetics (Megard et al., 1984), Chla is the chlorophyll *a* concentration in mg L^{-1}, Y_{O2CH} is the yield coefficient that equals to 120 mg O_2 (mg Chla)$^{-1}$, k_b and k_r are the first order decay rate coefficient for BOD and algae respiration, respectively, and equal to 0.1 d^{-1} (Brown and Barnwell, 1987). θ_s = 1.065, θ_p = 1.036, θ_b = 1.047 and θ_r = 1.047 are the temperature adjustment coefficients for SOD, photosynthesis, BOD and algae respiration, respectively. BOD in water bodies can vary greatly with external waste load inputs. For the regional lake simulations it is assumed that there are no external inflows of BOD to a lake (Stefan and Fang 1994), BOD is related to primary productivity represented as chlorophyll *a*, therefore, BOD is set as 1.0 mg L^{-1}, 0.5 mg L^{-1}, and 0.2 mg L^{-1} for eutrophic, mesotrophic, and oligotrophic lakes, respectively. In the model, chlorophyll-a is specified by a mean annual value and a function that calculates typical seasonal chlorophyll cycles (Stefan and Fang, 1994) based on observation data from 56 lakes or reservoirs in Europe and North American (Marshall and Peter, 1989). In the model, the oxygen transfer through the water surface (reaeration) during the open water season is used as an oxygen source or sink term in the topmost water (surface) layer of the lake, and the surface oxygen transfer coefficient is calculated as a function of wind speed (Wanninkhof et al., 1991). For modeling of stone quarries, we will include inputs of nutrients, sediments, and oxygen demand contaminants from watershed modeling.

Figure 3.6 shows simulated time series of water temperature (top panel) and DO concentration (bottom panel) at five different depths from the water surface under 1962 weather conditions. Simulated results for other years are similar to these patterns which are not plotted. The lake geometry ratio $A_s^{0.25}/H_{max}$ (where A_s is the surface area in m^2, and H_{max} is the maximum lake depth in m) is equal to 1.5 for this quarry, and this indicates that the lake is strongly stratified during summer (Fang and Stefan, 1999). Figure 3.6 does clearly show strong stratification in water temperature and DO concentration in this lake from early spring to late fall, and reveals that the lake is more or less well mixed in winter months (November to February). There is very weak inverse stratification of water temperature and slightly stronger stratification of dissolved oxygen (due to biochemical and sediment oxygen demand) during winter months. Starting in early July, dissolved oxygen concentrations at the lake bottom become zero; this anoxic condition last for five months. From August to mid September, the anoxic layer extends to as shallow as 6.5 m. Anoxia throughout a large portion of the lake volume will have profound impacts on the lake ecosystem and dynamics of many other water quality parameters (such as phosphorus and nitrogen cycles). Hypolimnetic aeration may be required to improve water quality before this quarry lake can be used as a source of water supply.

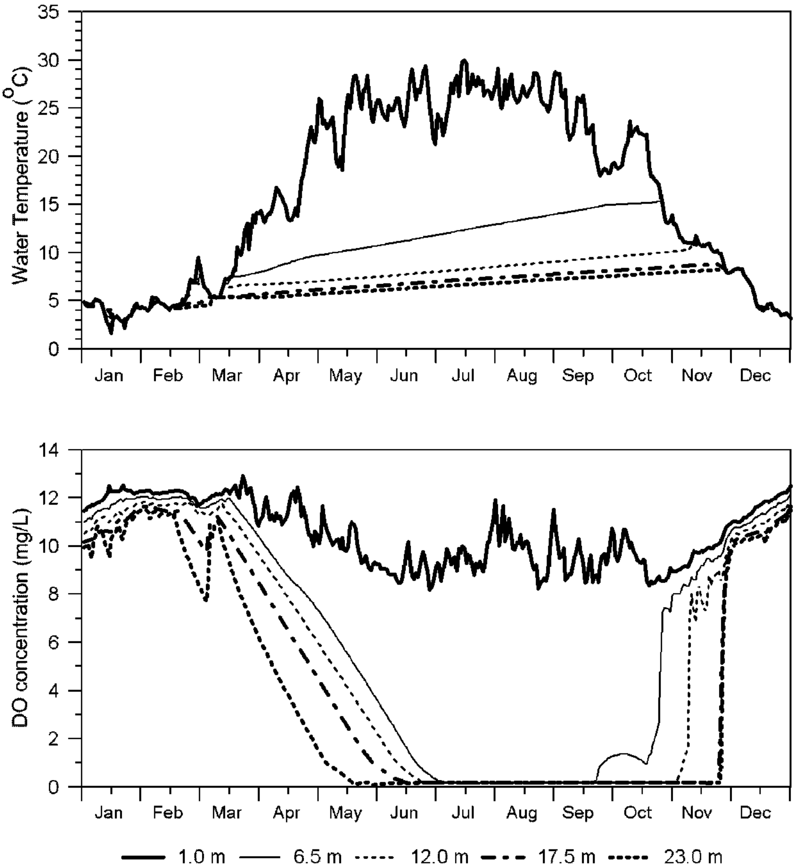

Figure 3.6: Simulated time series of water temperature (top) and dissolved oxygen concentration (bottom) for a 24-m deep eutrophic lake in Raleigh, NC

The lake simulated for Figure 3.6 is assumed to be an eutrophic lake with mean annual chlorophyll-a concentration of 15 μg/l and Secchi depth of 1.2 m. If the lake is oligotrophic (nutrient-poor, biologically unproductive) with less biochemical and sedimentary oxygen demand, no anoxic conditions will develop, as shown in Figure 3.7 for comparison. In an oligotrophic lake high water transparency allows solar energy penetrate to deeper depth and makes photosynthesis of phytoplankton be active and result metalimnetic oxygen maximum as shown higher DO concentrations at 6.5 m than at 1.0 m in Fig. 3.7. If runoff from surrounding watershed result in high turbidity, organic acids, and colored dissolved organic matter in a stone quarry, it can change water transparency that affects thermal stratification and water quality dynamics in the quarry and then affects whether or not the quarry can be a potential source of water supply for an urban center. Under anaerobic conditions, nitrate can be reduced to nitrite and the nitrite converted to free nitrogen by the process of denitrification (Chapra, 1997). High levels of nitrate in drinking water appear to be the cause of methemoglobinemia, or "blue babies", which primarily affects infants less than 6 months old. Such nitrate toxicity and public health problems may be avoided if some parts of the lake volume are anoxic.

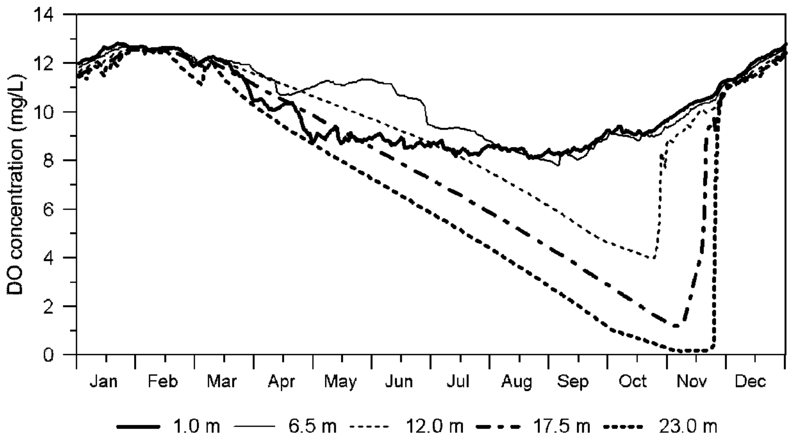

Figure 3.7: Simulated time series of dissolved oxygen concentration for a 24-m deep oligotrophic lake in Raleigh, NC

Geo-environmental Modeling Analysis

Water storage and recharge in the aquifers must be achieved with caution since water quality may be changed by biogeochemical reactions with aquifer materials. Recharge water may be warmer or cooler than native groundwater and it may dissolve certain minerals while allowing others to precipitate. This is because of the temperature-dependent retrograde and prograde solubility of carbonate and sulfate versus silicate mineral groups. Moreover, biochemical reactions may occur in the quarry and surrounding aquifers in response to changing redox conditions. Elevated Fe, Mn, and As concentrations may be derived from oxidation of metal sulfides or bacterial reduction of metal oxides as the redox conditions change (Penny et al., 2003; Lee et al., 2007). A successful operation of artificial recharge in a quarry requires a comprehensive understanding of the changes in groundwater flow, storage, and chemical conditions. When surface water is diverted for artificial recharge in quarries, it may or may not be in thermal, redox, or chemical equilibrium with minerals in quarry rocks. The above modeling effort can be integrated with water-rock interaction modeling. Preventing degradation of the water quality of the injected water and groundwater is important for meeting drinking water standards (Ziegler et al., 1999). In this study we demonstrate the use of a geochemical model, Geochemist's Workbench (Bethke, 2008), as a part of the decision support to investigate the chemical states of surface water and groundwater and quantify how injected water will react with minerals in the quarries. Moreover, typical fractured rocks consist of confining units and water bearing units that permit the rapid transport of water through connected fractures and conduits (National Research Council, 2001; White, 2002, 2006). We have developed a computer software, FRACFLOW (Lee and Wolf, 1998), to simulate such flow characteristics and hydraulic head distribution in fractured/karst rocks bounded by constant-head streams or lakes. The program first removes non-conducting portions (fractures or solution conduits with dead-ends) of a fractured rock and then calculates head distribution. The conduit network could be obtained by field fracture/conduit network mapping, surface and subsurface geophysics, or from a realization structure created by theoretical fracturing propagation models such as CRACK (Renshaw, 1996, 1999). This model can be integrated with the watershed and lake models to account for the impact of karst geology on the quarry reuse.

The numerical tools described above can be integrated to address science questions related to (1) chemical interactions among surface water, groundwater, and quarry or aquifer materials; (2) pathogen fate and transport from possible sources in nearby watersheds if the quarry is included; and (3) hydrological loading and storage capacity of the quarry under different geologic conditions. These considerations can be synthesized to generate index values reflecting the feasibility of the sustainable development of water supply with each candidate site for final decision making. One case study is given below to demonstrate this geo-environmental analysis.

Mobilization of As and other metals during aquifer storage and recovery (ASR) of surface water in carbonate aquifers is an increasing concern for drinking water quality (Arthur et al., 2002; Arthur et al., 2005). For example, increased concentrations of Fe, SO_4^{2-} , and the trace metals As, Co, Ni and Zn have been detected during the recovery of drinking water at the Collier County Manatee Road ASR facilities in Florida (Lucas et al., 2003). The concurrent increase of Fe, SO_4^{2-}, and trace metals at several Florida ASR sites suggests that the oxidation of pyrite (FeS_2) or arsenian pyrite may be the primary source of metal mobilization. These metals are bound to or reside in aquifer minerals such as sulfides, carbonates, and oxides. ASR operations can change physio-chemical conditions and mineral stability, resulting in water-rock interactions and metal release. Public health concerns related to drinking water quality exemplify the need to better understand the possible geochemical interactions that occur during the injection and recovery of surface water in the quarry or storage aquifers. We used the computer model Geochemist's Workbench to simulate interactions between injected surface water and the aquifer minerals calcite and arsenian pyrite. The model traces how a fluid's chemistry evolves and which minerals precipitate or dissolve over the course of reaction processes due to adding reactive minerals into a fluid.

We began by equilibrating injected surface water from Florida as the initial condition in the simulation. The calculation used the water chemical data collected from the Apalachicola-Chattahoochee-Flint (ACF) River Basin (station ID 023358694, Table 3.2). This injected surface water is slightly acidic (pH=6.6) and is saturated with Fe oxyhydroxides ($Fe(OH)_3$) under aerobic conditions. The reaction model (Figure 3.8) traces the chemical consequence of progressively titrating limestone aquifer minerals containing 10 mmol of calcite ($CaCO_3$) and 1 mmol of arsenian pyrite ($FeS_{1.99}As_{0.01}$) into the surface water at 25°C. Thermodynamic data for arsenian pyrite solid solution were compiled into a revised Geochemist's Workbench database *Thermo-As* (Lee et al., 2005; Saunders et al., 2008). This new thermodynamic database is more realistic in characterizing and predicting the arsenic behavior in reducing, iron-bearing groundwater conditions.

The predicted mineral reactions (Figure 3.8) show that ferric oxyhydroxide becomes thermodynamically unstable as oxygen is consumed by oxidation of arsenian pyrite. Arsenian pyrite continues to dissolve until the O_2 (aq) in the injected surface water has been consumed. In the calculation, 1 kg of injected surface water can oxidize about 0.720 mmol of arsenian pyrite until it becomes saturated under reducing condition. The final dissolved arsenic concentration reaches almost 550 μg/kg (or 0.720 mmol/kg), which far exceeds EPA's drinking water standard of 10 μg/L. The pyrite dissolution releases iron, sulfate, H^+, and arsenic into the water according to the reaction:

$$FeS_{1.99}As_{0.01}+1.02H_2O+3.49O_{2(aq)} \rightarrow Fe^{2+}+1.99SO_4^{2-}+0.01As(OH)_4^- +1.99H^+ \quad (3)$$

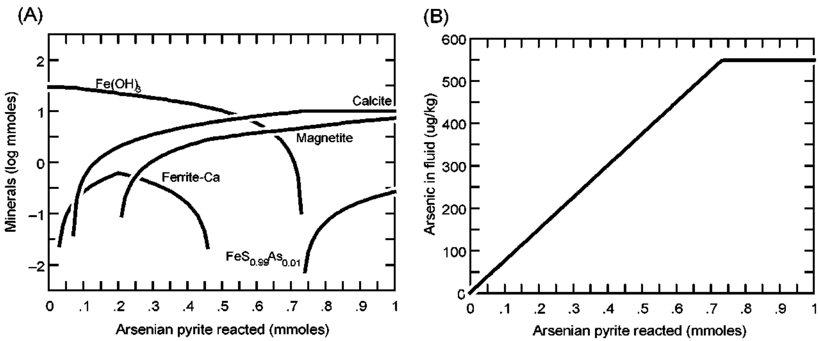

Figure 3.8: (A) Predicted mineralogic reactions resulting from oxidation of arsenian pyrite by injected oxygen-rich surface water. (B) Increase in dissolved arsenic concentration from dissolution of arsenian pyrite

The dissolution of arsenian pyrite predicted by the geochemical model agrees well with the concurrent increase of Fe, SO_4^{2-}, and As observed at several Florida ASR sites. The injected surface water participating in water-rock interactions maintains a near-neutral pH, reflecting the acid-buffering capacity of the calcite. This case study demonstrates that geochemical modeling of water-rock interactions can assist the decision making process by quantifying how naturally-occurring As may be released into stored water when oxygen-rich surface water reacts with limestone minerals in the reservoir's substrate.

Table 3.2: Major ion composition (in mg/kg) of injected surface water and groundwater used in computer simulations of mineral oxidation and fluid mixing reactions

Sample	Alkalinity (HCO_3^-)	Ca	Mg	Cl	K	Na	SO_4	pH
ACF River[1]	15	4.7	1.2	2.9	2.5	5.3	5.7	6.6
PCB 1[2]	190	62	49	196	4.3	104	13	7.6

1. Station ID 023358649, http://ga.water.usgs.gov/nawqa/tables/swlusbwc.maj.html
2. Panama City Beach (PCB) groundwater well #1 (McCartha and Lee, 2003)

Public Health Impacts of Quarry Lakes

Concerns about developing water supply with quarries can include potential chemical/biological contaminants (e.g., heavy metals, nitrate, fecal coliform, and fecal bacteria such as *E. coli*) from surrounding watersheds entering quarry reservoir surface water and groundwater, and water quality dynamics within the quarry reservoir. Heavy metals such as As, Co, Ni and Zn are of concern in the geochemical cycle. These concerns are related to protection of public health by the water supply treatment, distribution and storage system. For example, high nitrogen and phosphorus content in storm water runoff at the watershed scale can impede water reuse potential in the stone quarries and impact ecosystem integrity, human health, and community wealth (Alberta Environment, 2007). Nitrate may be toxic and can cause human health problem such as methemoglobinemia and liver damage. Phosphorus may trigger eutrophication of fresh water bodies, which could result in toxic algae and endanger the source of drinking waters (Chang et al., 2008). Despite these impacts, when urban regions gradually expand due to regional development, centralized sewage collection, treatment, and disposal are often unavailable for both geographic and economic reasons. Thus, decentralized or on-site wastewater treatment systems (OWTS) may be necessary to protect public health. Nationwide, wastewater effluent from OWTS can represent a large fraction of nutrient loads to groundwater aquifers (Chang et al., 2007). Determining the OWTS and combined sewer outflow sources of fecal and nutrient pollution will be important to track down the possible impacts of *Cryptosporidium* spp. and *Giardia* spp. With the aid of state health agencies, the sources of potential chemical/biological contaminants can be investigated to address the health impact. The integrated modeling system described here can be used to study the impacts of including stone quarries as storage reservoirs on public health concerns and to produce public health index values reflecting the potential of each candidate site for final decision making.

Socioeconomic Impacts of Quarry Lakes

The socioeconomic implications of both climate and nonclimate impacts on water supply and demand depend in large part on the ability of water managers and planners to adapt to change (Frederick and Gleick, 1999). A number of socioeconomic factors determine the end use of stone quarries. Specific candidate sites that are relatively more vulnerable in drought events may warrant more attention to assess the quantitative nature of risk. Socioeconomic impact analysis herein seeks to investigate the interplay between institutional factors and the integrative water supply systems due to the inclusion of stone quarries, in which the implementation plans can be facilitated by using an optimization platform. Such a platform covers water system components and includes financial and regulatory modules that can guide the water utility and local government in developing the system. This provides a learning process at the organizational level leading to a better understanding of the potential of each candidate site. For example, revenue from property taxes, sales tax, licenses and permits, and state and federal aid are influenced by population gains and losses. Consequently, it is necessary to investigate the population dynamics region-wide for the demand-side analysis that supports the financial viability determination. The overall structure of federal and state laws and regulations that provide mining-related environmental protection, such as local zoning laws and permitting decisions on the site-specific evaluation process, should be considered too. Monetary benefits of quarry rehabilitation can be estimated before conducting a benefit and cost analysis for each candidate site (Chang and Chang, 1997; Damigos and Kaliampakos, 2003). A benefit/cost ratio may be derived and used for the final decision making.

3.4 CONCLUSIONS

Due to rapid economic development and population growth and migration, critical civil engineering infrastructures have evolved into highly coupled and interacting, or interdependent networks and systems that are connected by complex physical, chemical, biological, natural resource, geographic, human, social, and economic processes. They must be sustainable and resilient, however, so as to cope with any catastrophic events, such as flood and drought, and they must be re-engineered to include recent advances in adaptive water resources management strategies. This study has identified potential problems from global climate change and cyclical droughts to water supply in fast growing urban regions that must be considered to fulfill the resilient and sustainable goals in design, construction, and operation of these interdependent environmental infrastructures. One of the solutions is an integrative approach using stone quarries as potential water supply reservoirs to meet the goal of water infrastructure sustainability. Whether or not a particular abandoned or active quarry can be used as a water supply reservoir depends on many factors.

Some important factors, considering geological, environmental, public health, water resources, and socioeconomic impacts, were analyzed holistically in the context of a preliminary environmental impact assessment. These efforts focused on the concept of sustainability and developing and applying a forward-looking, risk-informed, and cost-effective decision-making analysis that combines not only technical concerns but also social and economic factors with broad-based considerations of potential future environmental impacts. As demonstrated in our case study, water storage and recharge in the aquifers must be achieved with caution since water quality may be changed and toxic metals may be released by biogeochemical reactions with aquifer materials. A quarry reservoir can become thermally stratified during summer if it is deep enough, and the stratification can lead to oxygen depletion in the bottom waters, which may require hypolimnetic oxygenation (aeration) to improve water quality. To cover geoenvironmental, water supply, public health and socioeconomic concerns, the use of a Multiattribute Decision Making (MADM) framework for decision analysis may promote the policy making via understanding, quantifying, and modeling key attributes and criteria that affect developing stone quarry as water-supply reservoirs. Such finding should promote a holistic understanding of the physical world and the built environment via sensing, modeling, analysis, and prediction for water supply systems under normal operation and disaster conditions.

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CHAPTER 4

Saltwater Intrusion Management in Urban Area Aquifers: A Case Study for Savannah, Georgia, USA

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ABSTRACT: Urban development in coastal regions is usually linked to population growth in or near urban centers. Paralleling this growth is the demand on essential needs to sustain this population. Among other needs such as energy, transportation and health care infrastructure, fresh water supplies constitute one of the primary needs of these population centers. The management challenge is to provide sustainable water supplies to these centers through planning without causing adverse effects on the environment, economy and also the public health. In most cases groundwater is the most economical and readily available source of fresh water for coastal regions. Using this resource in providing fresh water to urban centers in coastal areas brings about an additional concern that needs to be addressed which may not be an issue for inland regions. This important concern is the saltwater intrusion problem and the degradation of freshwater aquifers. Saltwater intrusion and thus degradation of fresh groundwater resources is a commonly observed problem in most coastal urban areas and the cause is the increased pumping demand. Resolution of this problem creates a management challenge which can be addressed using sound science initiatives. This problem has also been observed in the Atlantic coast of the State of Georgia, USA. Long-term pumping from the Upper Floridan Aquifer (UFA) in the Savannah, Georgia area has lowered groundwater piezometric heads significantly in the region. This resulted in saltwater intrusion and brackish water contamination of the aquifer at or near Hilton Head Island, S.C. The UFA aquifer is the primary source of drinking and industrial process water in the region, and various categories of users routinely apply to the State of Georgia Environmental Protection Division (EPD) for groundwater withdrawal permits. These users include the municipalities in the region, industrial and agricultural users as well as the individuals residing in the area. In responding to this demand EPD's goal is the development of a long term groundwater management strategy, which will protect the UFA from further saltwater intrusion at the coastline while allowing additional groundwater withdrawals from the aquifer. In this chapter we will review these topics from a scientific perspective and discuss the current initiatives that are considered to address the saltwater intrusion problem in the Savannah region.

4.1 INTRODUCTION

Groundwater has been one of the major sources of drinking water all over the world (Bear, 1979). In the United States, the usage of groundwater is up to fifty percent of all drinking water supplies and this percentage may increase to eighty percents in some rural areas (OTA, 1984). From a management point of view, aquifers near the coast are unique because of potential saltwater intrusion problems which may be initiated due to excessive pumping. Thus, the management criteria for groundwater resources in these aquifers differ from other regions which are not near the coast. Improper water management practices in coastal aquifers is the main cause of saltwater intrusion and thus degradation of fresh water aquifers as reported for twenty coastal states in the USA (Frind, 1982). Once the degradation of aquifers occur it often results in a loss of fresh water resources and requires the need to seek alternative water supplies that tend to be more costly (Cheng, Halhal et al., 2000; Frind, 1982; Huyakorn, Andersen et al., 1987).

The solution of the saltwater intrusion problem is not a simple task. From a technical point of view there are several approaches that are available to us that may be used to analyze this problem. The first is the use of analytical solutions based on the Ghyben-Herzberg relationship (Bear, 1972; Bear, 1979) and the single potential theory approach introduced by Strack (Strack, 1976; Strack, 1989). Using this approach, steady state solutions can be obtained for hydrogeologically homogeneous aquifers. The second is a more complex model that is based on the assumption of sharp interface between freshwater and saltwater and the numerical solution of the governing equations of this conceptualization. In this approach one solves two nonlinear coupled partial differential equations for pressures or hydraulic heads of freshwater and saltwater numerically. The interface position is implicitly defined and arrived at as one of the outcomes of the numerical solution. This is a fairly reasonable approach for aquifers which exhibit narrow mixing zones such that the interface can be inferred from freshwater and saltwater heads. The third alternative is a relatively more complex approach that is based on solution of density dependent flow equations in both spatial and temporal dimensions (Diersch and Kolditz, 2002; Frind, 1982; Herbert, Jackson et al., 1988; Holzbecher, 1998; Kolditz, Ratke et al., 1998; Oldenburg and Pruess, 1995; Oldenburg and Pruess, 1996; Park, 2004; Park and Aral, 2007; Voss and Souza, 1987). Since this conceptualization allows the mixing of freshwater and saltwater in the transition zone of the interface it may represent the saltwater intrusion hydrodynamics in a more realistic way. From an analytical perspective, the use of one of these approaches would be necessary to analyze the saltwater intrusion problem in a coastal aquifer. Obviously, in this mix of available methods, experimental approaches can also be mentioned. However, such an analysis would not provide us solutions to regional problems but instead may give us insight to the occurrence of physical conditions for idealized situations. The choice of the proper methodology to be adopted would depend on the complexity of the aquifer system under study, the accuracy expected of the modeling outcome for the chosen aquifer system, and the budget and time limitations of the overall study.

Two goals can be identified as important issues associated with groundwater resources management in coastal aquifers. These are: (i) freshwater production to supply the demand; and, (ii) environmental preservation. Accordingly, the goals are to maximize freshwater extraction while minimizing or elimination saltwater intrusion into the aquifers. These goals are of great importance and are in conflict with one another in the proper management of coastal aquifers. The solution to this problem can be approached using an optimal management strategy. In this chapter we will briefly review the theoretical basis of these models and their applications in the literature and conclude with the analysis of the saltwater intrusion problem in the Savannah region of the State of Georgia, USA.

4.2 LITERATURE REVIEW

In this section a literature review is provided on the following selected topics: (i) optimization of pumping rates and well placement in coastal aquifers for fresh water extraction and controlled saltwater intrusion; (ii) variable density flow models in variably saturated porous medium; and, (iii) multi-objective analysis of the Savannah region saltwater intrusion problem. The discussion included in this chapter will be limited to these three important topics considering the space limitations of this chapter. Among the three topics identified above the first approach, since it uses analytical solutions, may only provide solutions for an idealized coastal aquifer case. The second approach is very elaborate but very complex and has significant computational burden. Thus, it is very difficult and cumbersome to link the variable density flow algorithms with optimization methods to arrive at the optimal solutions of the saltwater intrusion management problem for site specific cases. The third case involves simpler conceptualizations and thus simpler numerical models but considers uncertainty aspects of the management problem and in doing so may introduce a new perspective to the overall management problem.

4.2.1 Application of Optimization Methods in Saltwater intrusion Management

The use of optimization algorithms in the analysis of groundwater planning and management problems has a long history as demonstrated in the following selected publications (Ahlfeld, Mulvey et al., 1988a; Ahlfeld, Mulvey et al., 1988b; Aly and Peralta, 1999; Aral, Guan et al., 2001; Atwood and Gorelick, 1985; Gorelick, Evans et al., 1983; Gorelick and Remson, 1982; Guan and Aral, 2004; Ling, Rifai et al., 2005; Marryott, Dougherty et al., 1993; Peralta, Solaimanian et al., 1995; Ratzlaff, Aral et al., 1992; Wagner and Gorelick, 1987). Even though the water management problem may be formulated in several unique ways depending on the criteria selected for management and many other site specific conditions, most optimization problems in subsurface applications can be categorized into two groups: (i) extracting safe potable groundwater in the vicinity of a contaminated aquifer or coastal zone, or providing safe yield in a clean aquifer system; and, (ii) containing and/or remediating contaminant plumes.

The first group includes a range of problems including aquifers which are exposed to contaminant sources or coastal aquifers threatened by saltwater intrusion due to excessive water extraction. Thus, for these cases, the optimization analysis focuses on the control issues associated with the protection of aquifer resources. The second group of problems is associated with the remediation of aquifers which are contaminated. In most cases the technique used will be the pump-and-treat method or air sparging technologies and optimal management of these applications. Similar to design of pump-and-treat systems, optimization problems in saltwater intrusion control are unique, in having goals such as the protection of aquifers from saltwater intrusion while providing fresh water to users. Surprisingly, the use of optimization methods in the solution of saltwater intrusion problems are relatively recent and few (Cheng, Halhal et al., 2000; Das and Datta, 1999a; Das and Datta, 1999b; Das and Datta, 2001a; Emch and Yeh, 1998; Finney, Samsuhadi et al., 1992; Hallaji and Yazicigil, 1996; Park and Aral, 2004; Shamir, Bear et al., 1984; Willis and Finney, 1988a). The consideration of different aspects of the saltwater intrusion problem such as inland aquifer intrusion, saltwater up-coning or hammock formation in tidal estuaries and the complexity of the solution of these problems has lead to the formulation of many different optimization models as well as the use of many different techniques to solve these optimization problems. For example, the analytical solution of the saltwater intrusion problem using the sharp interface assumption allows flexibility to the solution of the optimization problem since this approach yields computationally efficient procedures for the solution of the hydrodynamics equations. On the other hand, variable density models, which are more complex but more realistic in conceptualization, yield hydrodynamic models which have limited use in optimization analysis. This is due to the computational burden they have in solving the hydrodynamics problem repeated solution of which is needed in optimization analysis.

Willis and Finney (1988), Finney et al. (1992), Cheng et al. (2000) and Park and Aral (2004) studied coastal aquifer management models for saltwater intrusion control with the assumption of the sharp interface between fresh water and saltwater phases. While most of the management problems addressed saltwater intrusion into wells in an indirect manner, such as constraining drawdown at control points or minimizing the intruded saltwater volume or concentration, Cheng et al. (2000) used an optimization approach to solve for the pumping rates for an existing multiple extraction well system in a coastal aquifer using Genetic Algorithm (GA). In their study, the analytical solution of the sharp-interface saltwater intrusion model was used for simplicity. The use of a more complex model is deferred, in part due to the intensive computation time required by the repetitive use of the model during the optimization process. In their solution, they have discretized the pumping rate and used the Structured Messy Genetic Algorithm (SMGA) approach with the pumping rate selected as the basic design variable. Park and Aral (2004) successfully formulated an optimization model that has continuous pumping rate as the design variable. In their application they have also selected the well locations as continuous decision variables that has to be determined as a part of the optimal management problem. With these applications GAs gained popularity as a robust method in solving saltwater intrusion problems. The GA approach is further extended to progressive GA (PGA) approach to improve efficiency of the solution process (Aral, Guan et al., 2001; Guan and Aral, 1999a; Guan, Kentel et al., 2008; Guan and Aral, 1999b).

4.2.2 Density Dependent Groundwater Flow Models

The origin of density dependent groundwater flow analysis can be traced back to Henry's problem (Henry, 1960), where an analytical solution to the steady state variable density saltwater intrusion problem was provided. Variable density analysis is based on the quantitative description of the dynamic equilibrium between fresh water phase and the saltwater phase in a coastal aquifer (Holzbecher, 1998). Even though there have been critical remarks in the literature on the use of an unrealistic boundary condition on the seaside boundary of this problem and the use of the constant diffusivity in the solution, Henry's problem has become one of the most popular benchmark problems in variable density flow analysis (Frind, 1982; Huyakorn, Andersen et al., 1987; Lee and Cheng, 1974; Pinder and Cooper, 1970; Segol, Pinder et al., 1975). Interestingly, none of the numerical solutions has been able to obtain results that were close to the semi-analytical solution provided by Henry. This may be due to some form of inaccuracy in the original findings of Henry that requires that the contours of equipotential heads are not orthogonal to Darcy velocity vectors (Kolditz, 1994). Nevertheless, it is a trend in variable density flow analysis that the Henry problem is widely chosen as one of the benchmark problems for validation of the existing or newly developed codes prior to the use of the code in more complex applications.

The Elder problem (Elder, 1967), is another classic problem related to density dependent flow analysis. In this problem, Elder used a Hele-Shaw cell to generate a thermal convection-dominated flow by constantly heating part of the base of the medium. The Elder problem can be analogous to density-driven flow problem from the hypothesis of (Cooper, 1959) "differing from it only in that changes in density are produced by changes in concentration rather than by changes in temperature." Similar to the Henry problem, the Elder problem is also used as a benchmark problem for variable density flow analysis with varied success in its numerical solution (Boufadel, Suidan et al., 1999; Frolkovič and de Schepper, 2001; Oldenburg and Pruess, 1995; Park and Aral, 2007; Voss and Souza, 1987).

Salt domes has been known to be a good geological formation for storage of radioactive wastes and solution of this problem is also related to density dependent flow analysis (Holzbecher, 1998; Witherspoon, 1991). In this case the denser fluid is at the bottom of the solution domain and the convection above creates the density dependent nature of the flow. The Hydrologic Code Intercomparison (HYDROCOIN) project was developed in 1984 as multi-country effort for understanding groundwater flow in radioactive waste disposal sites characterized by this type of flow conditions. Numerous researchers has attempted to solve this problem with varied degree of success (Herbert, Jackson et al., 1988; Kolditz, 1994; Konikow, Sanford et al., 1967; Konikow, Sanford et al., 1997; Oldenburg and Pruess, 1966; Wooding, Tyler et al., 1997a; Wooding, Tyler et al., 1997b). It was (Konikow, Sanford et al., 1997) that discovered the erroneous use of boundary conditions which lead to different results in the numerical solution in this problem.

Saltwater up-coning is another important problem that has attracted the attention of researchers. Saltwater up-coning can be described as the movement of saltwater from a deeper saltwater zone into the fresh groundwater zone in response to reduction of pressures due to pumping at a single well or a well filed (Reilly, Frimpter et al., 1987; Reilly and Goodman, 1987). This is a widespread problem that is also observed in coastal aquifers in several states in the USA including Georgia (ASCE, 1969; Krause and Clarke, 2001) as well as globally in other countries. The solution of this problem has been attempted using a sharp interface model (Motz, 1992) or the variable density flow analysis approach as discussed above and also in (Reilly, Frimpter et al., 1987; Reilly and Goodman, 1987). The most recent work on this line of research is by (Oswald and Kinzelbach, 2004).

4.2.3 Multi-objective Analysis of Savannah Saltwater Intrusion Problem

Groundwater resources management problems of coastal aquifers are complex. Local government authorities must decide the future development of groundwater resources and whom to grant permits for groundwater withdrawal given the constraints on the saltwater intrusion problem. Selection of the best management strategy for the groundwater resources development and conservation in a coastal aquifer can be achieved by considering hydrological, political, and economical objectives. Depending on the nature of the objective, for certain cases, it may be more convenient to characterize the objectives and the constraints of the optimization problem as a fuzzy set instead of crisp objectives and constraints. Approaching the solution of this problem in this manner may yield a better management strategy and more informed decisions.

For this purpose a coupled simulation-optimization model, followed by a fuzzy multi-objective decision-making framework, can be utilized in order to satisfy the multiple pumping demands in a coastal aquifer while keeping the saltwater intrusion problem in check. Coupled simulation-optimization models have been developed to manage groundwater resources earlier (Emch and Yeh, 1998; Kentel and Aral, 2007; Mantoglou, 2003a; Mantoglou, 2003b; McPhee and Yeh, 2004; Shamir, Bear et al., 1984; Willis and Finney, 1988a). Fuzzy set theory concepts have also been used for various classes of decision-making problems, such as fuzzy preferences and choice, group decision-making, multi-criteria decision-making, multistage decision-making, optimization, mathematical programming and dynamic programming. Decision-making in fuzzy environment have been discussed in various texts (Bezdek, Dubois et al., 1999; Carlsson and Fullér, 2002; Kacprzyk and Orlovski, 1987; Slowiński, 1998). More recently, this topic has also been covered in several technical papers including (Bender and Simonovic, 2000; Carlsson and Fullér, 1996; Roubens, 1997; Sakawa, Inuiguchi et al., 1993; Stanciulescu, Fortemps et al., 2003; Yager, 2004).

In saltwater intrusion control and groundwater extraction management problem in coastal aquifers various questions may need to be answered. For example: What is the safe pumping rate for existing wells in a coastal aquifer, before saltwater intrusion becomes a problem? How many wells are needed to supply the required freshwater demand to a community before saltwater intrusion becomes a problem? From a more practical point the management question can be posed as: Where should these wells be placed and what should their pumping rates be, before saltwater intrusion becomes a problem? How close can these wells be placed to the coast and what would their maximum withdrawal rates will be before saltwater intrusion becomes a problem? Some of these questions can be answered using the ideal condition of homogeneity of aquifers in the coastal zone. Combining analytical solutions with optimization methods, the analysis of this problem is possible. The solution of this problem is discussed in Section 4.3. For regional problems with heterogeneous aquifers the best approach would be to combine the optimization methods with density dependent simulation models. This has not been attempted in the literature due to the overwhelming computational burden required in the coupled solution methodology. Another approach that is considered is the linkage of the uncertain management decision criteria with optimization methods using simpler groundwater flow models. A discussion of this approach is given in Section 4.4 below.

4.3 USE OF ANALYTICAL SOLUTIONS AND OPTIMIZATION METHODS

The groundwater extraction problem in a coastal aquifer can be formulated as a multi-objective optimization problem where the decision variables are the pumping rates of extraction wells and the well locations with the constraints of prevention of saltwater intrusion as the increasing demand of an urban community is satisfied. The method that can be used for this purpose is the iterative sub-domain GA method, in which the algorithm searches for the optimal solution by perturbing the well locations and pumping rates simultaneously (Park and Aral, 2004). In this case the decision variables of the optimization problem are modeled as continuous independent variables. Sharp interface solution of the homogenous steady state problem is used along with the Dupuit and Ghyben-Herzberg assumptions. In this approach, the direct method of searching for saltwater intrusion points is formulated by comparing the location of the stagnation points of the flow field, and the saltwater intrusion profiles obtained from the single-potential theory solution (Strack, 1976). The search for the optimal solution, within each sub-domain, is conducted using the Genetic Algorithm (GA). The multi-objective problem is formulated to maximize pumping rates while minimizing the distance between critical stagnation point and the reference coastline location, such that the wells are placed as closely as possible to the coastline.

4.3.1 Governing Equations

Analytical solutions used in this approach are valid if the following assumptions can be made for the aquifer under study: (i) a sharp interface exists between the saltwater zone and the fresh water zone, rather than a transition zone; (ii) the aquifer is assumed to be homogeneous, and steady state conditions are considered.; (iii) Dupuit assumption is used to obtain the two-dimensional equations, by averaging the flow equation in vertical direction (Bear, 1972; Bear, 1979); (iv) the interface location is deduced from the Ghyben-Herzberg assumption; and, (v) a single-potential theory approach (Strack, 1976) is adopted to make use of a single governing potential equation across the two zones of the coastal aquifer, Figure 4.1.

Following (Cheng, Halhal et al., 2000; Strack, 1976) a potential ϕ is defined for both confined and unconfined aquifers as follows. For confined aquifers:

$$\phi = Bh_f + \frac{(s-1)B^2}{2} - sBd \qquad zone\,1 \qquad (1)$$

$$\phi = \frac{1}{2(s-1)}\left[h_f + (s-1)B - sd\right]^2 \qquad zone\,2 \qquad (2)$$

For unconfined aquifers:

$$\phi = \frac{1}{2}\left[h_f^2 - sd^2\right] \qquad zone\,1 \qquad (3)$$

$$\phi = \frac{s}{2(s-1)}\left(h_f - d\right)^2 \qquad zone\,2 \qquad (4)$$

where h_f is the freshwater head, d is the elevation of mean sea level above the datum, and B is the confined aquifer thickness. The density ratio of saltwater to freshwater is given as,

$$s = \frac{\rho_s}{\rho_f} \qquad (5)$$

where ρ_s and ρ_f are the saltwater and freshwater densities respectively. Since the potential function satisfies the Laplace equation, $\nabla^2\phi = 0$, the interface location ξ can be defined using proper boundary conditions for both aquifers. For a confined aquifer:

$$\xi = \sqrt{\frac{2\phi}{s-1}} + d - B \qquad (6)$$

For an unconfined aquifer:

$$\xi = \sqrt{\frac{2\phi}{s(s-1)}} \qquad (7)$$

From Figure 5.1, the toe of saltwater can be evaluated at $\xi = d$. Hence, the potential at the toe can be calculated from Equations (6) and (7). For a confined aquifer:

$$\phi_{toe} = \frac{(s-1)}{2}B^2 \qquad (8)$$

For an unconfined aquifer:

$$\phi_{toe} = \frac{s(s-1)}{2}d^2 \qquad (9)$$

The freshwater potential for multiple pumping wells, in an aquifer with uniform flow, can be obtained using the method of superposition.

$$\phi = \frac{q}{K}x + \sum_{i=1}^{n}\frac{Q_i}{4\pi K}\ln\left[\frac{(x-x_i)^2 + (y-y_i)^2}{(x+x_i)^2 + (y-y_i)^2}\right] \qquad (10)$$

Using either Equation (8) or (9) in Equation (10), the toe location for the multiple wells can be solved. The location of multiple stagnation points of the flow field is important to define the maximum pumping rate for the pumping wells. The locations of the stagnation points can be obtained from the following relation,

$$\frac{\partial\phi}{\partial x} = \frac{\partial\phi}{\partial y} = 0 \qquad (11)$$

Differentiating Equation (10):

$$\frac{\partial\phi}{\partial x}=\frac{q}{K}+\sum_{i=1}^{n}\frac{Q_i}{4\pi K}\frac{(x+x_i)^2+(y-y_i)^2}{(x-x_i)^2+(y-y_i)^2}\left[\frac{2(x-x_i)}{(x+x_i)^2+(y-y_i)^2}-2(x+x_i)\frac{(x-x_i)^2+(y-y_i)^2}{\left((x+x_i)^2+(y-y_i)^2\right)^2}\right] \tag{12}$$

$$\frac{\partial\phi}{\partial y}=\sum_{i=1}^{n}\frac{Q_i}{4\pi K}\frac{(x+x_i)^2+(y-y_i)^2}{(x-x_i)^2+(y-y_i)^2}\left[\frac{2(y-y_i)}{(x+x_i)^2+(y-y_i)^2}-2(y+y_i)\frac{(x-x_i)^2+(y-y_i)^2}{\left((x+x_i)^2+(y-y_i)^2\right)^2}\right] \tag{13}$$

Equations (12) and (13) and the condition given in Equation (11) form a set of nonlinear equations. Newton-Raphson method can be used to solve the stagnation points from these equations. These equations need to be further differentiated to make a use of the Newton-Raphson method. The stagnation point location can be used to detect the saltwater well-intrusion condition, by comparing the location of this point relative to the location of the toe on the coastline.

4.3.2 Optimal Solution of the Saltwater Intrusion Problem

One of the objectives in the optimal solution of saltwater intrusion problem is the maximization of the pumping rates. The optimal solution of the saltwater intrusion problem becomes unique, if the extraction wells are forced to be placed as closely as possible to the coastline. This is a typical conflicting condition since the development region in most cases has to be selected close to the coastline. This situation may cause saltwater intrusion problems. If the extraction wells can be placed further inland, extraction wells may be allowed to pump at higher rates without concern of saltwater intrusion. Thus, placement of the pumping well as close as possible to the coastline is the second objective of the optimization problem. This, in turn, restricts the first objective, that is the maximization of the pumping rate. The solution of this multi-objective optimization problem has been attempted using various techniques. (Fonseca and Fleming, 1998) used the GA for solving the multi-objective problems and introduced the concept of dominated and non-dominated solutions. There are other techniques that can be used in the solution of the multi-objective problems, such as fitness sharing, niche approach, etc. GA uses the survival of the fittest concept in selecting the competitive populations. This selection process is linear in the scalar objective value of each population. However, the main difficulty originates from the multi-objective nature of the problem. There may be no simple way to differentiate the relative importance of each objective in the selection of the best other than the determination of the Pareto optimal front. Another way to deal with multiple objectives is to use the weighed sum of each objective function to form a single scalar objective function. This approach brings the difficulty of adjusting the weight of each objective, which can be treated as the managerial decision. However, one should recognize that a single scalar objective function generated is not capable of representing the vector tendency of each objective. Nonetheless, if we use the single scalar objective function approach for simplicity, the two objectives and the combined objective function of the optimization problem can be given as seen below. The first objective is the maximization of the pumping rate for each well.

$$Max\sum_{i=1}^{n} Q_i \tag{14}$$

The second objective is the minimization of the distance between the stagnation points and the reference coastline location, which can be represented as a maximization problem as given in Equation (15).

$$Max\left(-\sum_{i=1}^{n}(x_c^i - x_{ref})\right) \tag{15}$$

where x_c^i and x_{ref} are the inland x-axis coordinates of the stagnation points of the pumping well i and the location of the coastline in x-axis direction respectively, Figure 4.1. If the coastline is not perpendicular to the x-axis direction, and thus represented by a sloping shoreline with respect to x-axis, Equation (15) should be selected to represent the perpendicular distance between the stagnation point and the coastline for a two-dimensional domain. For the geometry considered in Figure 4.1, the combined and normalized objective function can be written as,

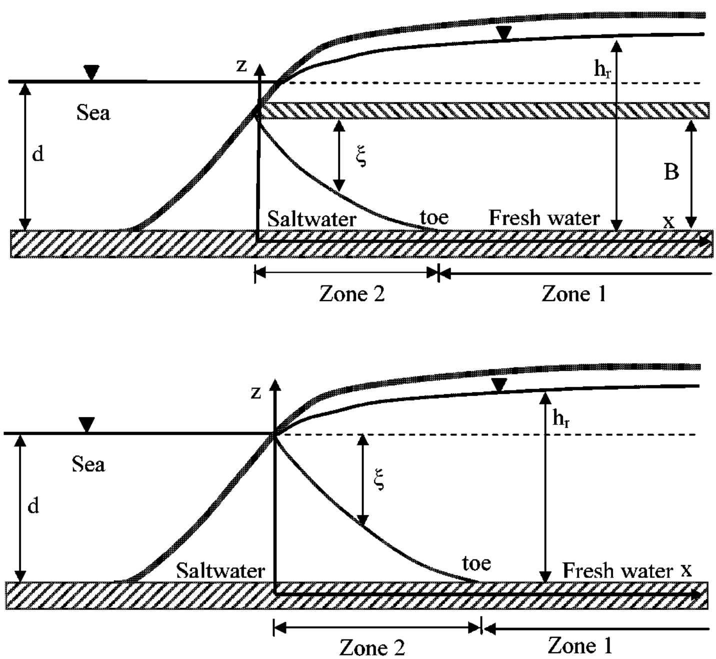


Figure 4.1: Cross-sections of a coastal aquifer in a confined and unconfined aquifer

$$f\left(Q, x_{ref}, x_c^i\right) = \sum_{i=1}^{n}\left(\alpha \frac{Q_i}{Q_i^{\max}} + \beta\left(\frac{x_{ref}}{x_c^i} - 1\right)\right) \tag{16}$$

Subject to the following conditions,

$$x_{toe}^i(Q, X, Y) < x_c^i(Q, X, Y) \tag{17}$$

$$Q_i^{\min} < Q_i < Q_i^{\max},\ x_i^{\min} < x_i < x_i^{\max},\ y_i^{\min} < y_i < y_i^{\max}\ \ i = 1, \ldots, n \tag{18}$$

where Q_i is the pumping rate of well i, x_{toe}^i is the toe location at the well i, and α and β are the objective function weighting parameters. The independent variable vectors, Q, X, Y, which represent the pumping rates and (x, y) coordinates of each well, will be redefined in the modified optimization formulation for the progressive genetic algorithm application (POGA) (Guan and Aral, 1999b).

In solving this problem (Cheng et al., 2000) have used the discretized pumping rate as an independent variable for optimizing pumping rates of fifteen fixed wells in a case study. This independent variable itself requires a large number of simulations for the solution of the optimization problem. For a typical application, it is almost impossible to complete an exhaustive search for the global solution. To avoid this computationally intensive approach, the decision variables of the optimization problem can be changed. Rather than selecting the pumping rates and the well locations as decision variables, one can chose the perturbations of these variables as decision variables. In this manner, GA solution can eliminate a considerable amount of unnecessary simulations at each step of the iterative solution. To achieve this, for each optimization step of each perturbation, a sub-domain should be defined, and the search can be conducted within this sub-domain. No matter where the simulation starts, there is a path from this starting point to the optimal solution. The direction from the current to the next location on the correct path may be determined easily by GA within the pre-determined sub-domain, even if it is far away from an optimal solution. This approach allows the subsequent sub-domains to move in the optimal direction as well, which leads the solution closer to the global optimum. Thus, within each step the solution gets closer to the optimal solution by effectively eliminating the unnecessary paths, as shown in Figure 4.2a.

The optimal solution to saltwater intrusion problem is complex, due to the dependence of the pumping rates and well locations on each other. The sub-domain concept handles this dependence issue effectively and provides good feedback as the simulation proceeds. One other advantage of this method is that the independent variables are treated as continuous variables rather than discrete ones. In this approach, the modified objective function, the constraints and the independent variables of the perturbations can be given as:

$$f\left(Q, x_{ref}, x_c^i\right) = \sum_{i=1}^{n}\left(\alpha \frac{Q_i^0 + \Delta Q_i}{Q_i^{\max}} + \beta\left(\frac{x_{ref}}{x_c^i} - 1\right)\right) \tag{19}$$

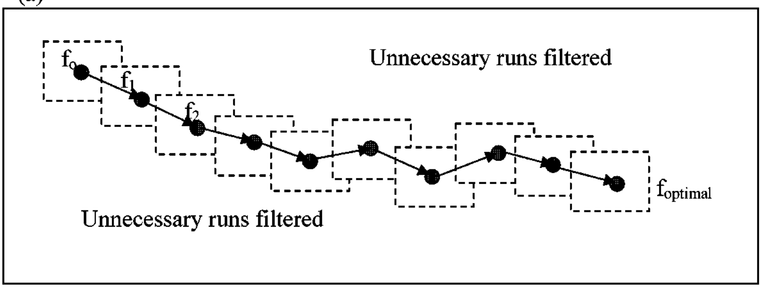
$$\Delta Q_i^{\min} < \Delta Q_i < \Delta Q_i^{\max},\ \Delta x_i^{\min} < \Delta x_i < \Delta x_i^{\max},\ \Delta y_i^{\min} < \Delta y_i < \Delta y_i^{\max}\ \ i = 1, \ldots, n \tag{20}$$

where,

$$\left.\begin{array}{l}\Delta Q_i^{\min} = Q_i^{\min} - Q_i^0;\ \Delta Q_i^{\max} = Q_i^{\max} - Q_i^0 \\ \Delta x_i^{\min} = x_i^{\min} - x_i^0;\ \Delta x_i^{\max} = x_i^{\max} - x_i^0 \\ \Delta y_i^{\min} = y_i^{\min} - y_i^0;\ \Delta y_i^{\max} = y_i^{\max} - y_i^0\end{array}\right\} \tag{21}$$

$Q_i^0, x_i^0, and\, y_i^0$ is the initial starting discharge and the initial starting coordinates respectively, and the bounding sub-domain is determined to be polyhedron having the starting points at the center of the polyhedron. The stagnation point x_c^i of the well *i*, used in the perturbed objective function Equation (19), is an implicit function of the perturbed coordinates of the well.

(a)



(b)

$$Parent\,1: (\Delta Q_1, \Delta x_1, \Delta y_1)(\Delta Q_2, \Delta x_2, \Delta y_2)\cdots(\Delta Q_n, \Delta x_n, \Delta y_n)$$
$$Parent\,2: \left(\Delta Q_1, \Delta\tilde{x}_1, \Delta\tilde{y}_1\right)\left(\Delta Q_2, \Delta\tilde{x}_2, \Delta\tilde{y}_2\right)\cdots\left(\Delta Q_n, \Delta\tilde{x}_n, \Delta\tilde{y}_n\right)$$

$$Child: \begin{pmatrix}\Delta Q_1 & \Delta x_1 & \Delta y_1 \\ or & or & or \\ \Delta Q_1 & \Delta\tilde{x}_1 & \Delta\tilde{y}_1\end{pmatrix}\begin{pmatrix}\Delta Q_2 & \Delta x_2 & \Delta y_2 \\ or & or & or \\ \Delta Q_2 & \Delta\tilde{x}_2 & \Delta\tilde{y}_2\end{pmatrix}\cdots\begin{pmatrix}\Delta Q_n & \Delta x_n & \Delta y_n \\ or & or & or \\ \Delta Q_n & \Delta\tilde{x}_n & \Delta\tilde{y}_n\end{pmatrix}$$

n is the number of wells.

Figure 4.2: (a) The concept of eliminating unnecessary runs using perturbation method; (b) Coding and crossover of design variables.

In order to handle the inequality constraint of Equation (17) properly in the objective function of GA, the slack vector and penalty functions are introduced. Thus, the modified objective function and constraints can be given as:

$$f(Q, x_{ref}, x_c^i) = \sum_{i=1}^{n} \left(\alpha \frac{Q_i^0 + \Delta Q_i}{Q_i^{\max}} + \beta \left(\frac{x_{ref}}{x_c^i} - 1 \right) \right) - \sum_{\forall_i} \gamma s^2 \tag{22}$$

$$x_{toe}^i(Q, X, Y) + S = x_c^i(Q, X, Y) \qquad S \geq 0 \tag{23}$$

where

$$\gamma = \begin{cases} c_1 \ \textit{if} \ s_i < 0 \\ 0 \ \textit{otherwise} \end{cases} \tag{24}$$

and c_1 is large constant. It should be noted that the independent variables of this formulation are now the perturbations, $\Delta Q_i, \Delta x_i, \Delta y_i$. The solution to this optimization problem can be obtained using POGA (Park and Aral, 2003; Park and Aral, 2004).

4.3.3 Applications

A numerical example for three, four and five well placement case is given below as the demonstration of the multi-objective optimization problem. Other applications using this approach can be found in (Park and Aral, 2003; Park and Aral, 2004). The aquifer domain and aquifer parameters used in this application are given in Table 4.1 and Figure 4.3. Although the number of pumping wells for each of these cases is different, the total optimal pumping rates obtained are close to each other. This result indicates that, for a single objective problem, the total optimal pumping rate for a domain can be achieved by a finite number of wells, and adding more wells to the domain will not improve the maximum pumping rate significantly as shown in Figure 4.4. The maximum economic number of wells for this case is 3. As can be seen from this figure, the total pumping rate does not increase significantly if we add more wells to the solution. In this figure, we also show the average pumping rate for the wells. Based on the heuristic stated previously, the global maximum pumping rates for this simple application can be obtained when we perturb the well locations and the pumping rates simultaneously. It should be noted that once the critical number of wells is established within the solution domain, adding more wells often creates saltwater toe delineation similar to the case of the critical number of wells. This can be observed from Figure 4.3d. In this figure, there are three peaks for the toe even though the number of wells is four. Note that this figure represents the intermediate solution obtained from the four well solution. This indicates that the same optimal pumping rate obtained from the three pumping well case can be obtained from four pumping wells yielding a similar pattern of toe delineation. Since Figure 4.3d is an intermediate solution, continuing the simulation will reach the results shown in Figure 4.3b. Obviously, the improvement achieved is minimal as shown in Figure 4.4, since the four pumping wells are already more than the critical number of wells for this case.

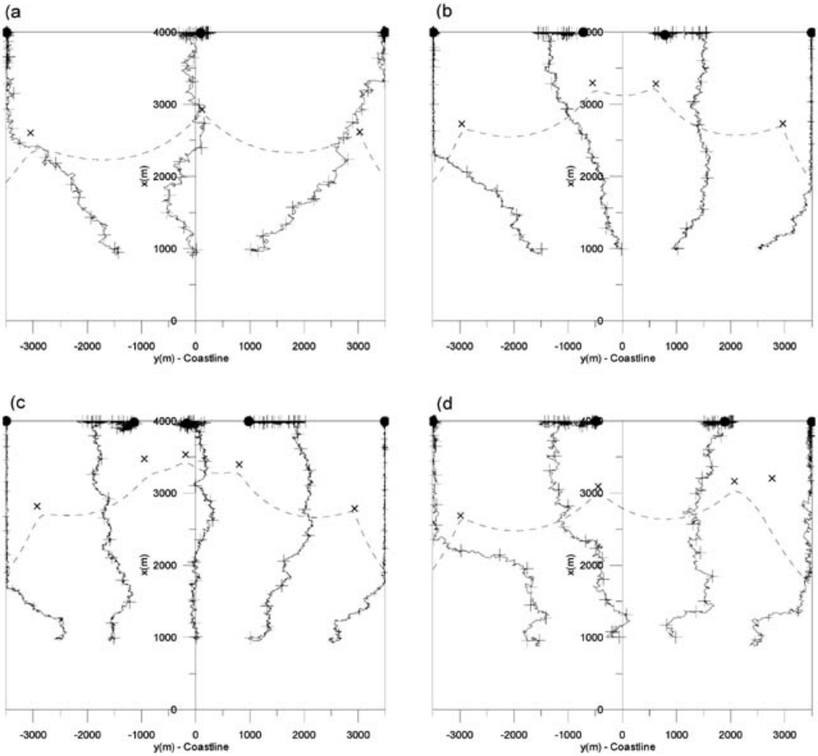


Figure 4.3: (a) Three pumping well placement optimization path; (b) Four pumping well placement optimization path; (c) Five pumping well placement optimization path; (d) Four pumping well placement optimization path (an intermediate solution – similar toe pattern to that given in (a))

Table 4.1: Summary of modeling parameters for three-, four- and five well cases

Aquifer parameters	Value	MOGA parameters	Value
Aquifer type	unconfined	α, β	1/2, 1/2
Saltwater density	1.025 g/cm^3	Convergence	0.001
Uniform flow rate	0.6 m^2/d	Population size	40
Saltwater depth	14 m	Mating probability	0.9
Hydraulic conductivity	100 m/d	Mutation probability	0.1

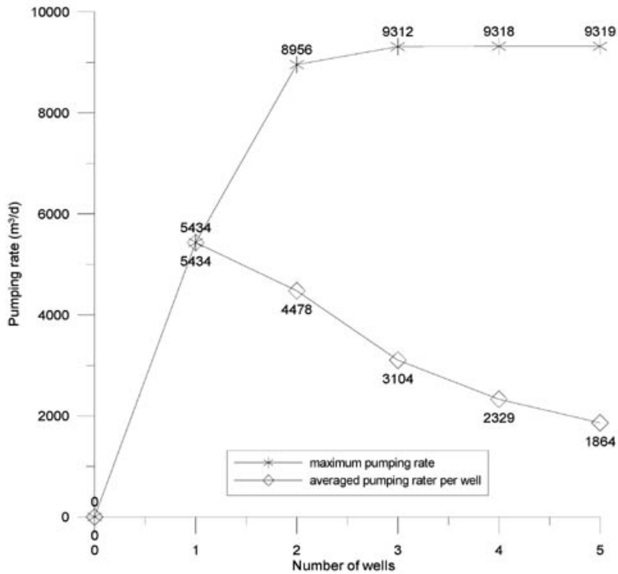


Figure 4.4: The relationship between the number of wells and the maximum pumping rates and thewell efficiency.

4.4 OPTIMAL MANAGEMENT OF COASTAL AQUIFER UNDER FUZZY RULES: THE SAVANNAH CASE STUDY

In this section the use of a coupled simulation-optimization model followed by a fuzzy multi-objective decision-making framework is discussed in order to satisfy the multiple pumping demands and objectives in a coastal aquifer. Coupled simulation-optimization models have been developed to manage groundwater resources (Emch and Yeh, 1998; Mantoglou, 2003b; McPhee and Yeh, 2004; Shamir, Bear et al., 1984; Willis and Finney, 1988b). The decision-making framework proposed here can be considered to be an extension of these applications to evaluate the performances of various alternative management strategies with respect to multiple fuzzy objectives. Fuzzy set theory concepts have been used for various classes of decision-making problems as discussed in Section 4.2.3.

To demonstrate the application of the proposed method, one may consider a set of hypothetical groundwater withdrawal applications from the Floridan aquifer system in the Savannah region of Georgia, USA. Each demand location in the aquifer is represented by a hypothetical well and the coupled simulation-optimization model is formulated to calculate the optimum additional groundwater withdrawal rates from each of these wells assuming that all of them will be extracting groundwater at the same time.

The Floridan aquifer system is composed of two water-bearing aquifers, the Upper Floridan Aquifer (UFA) and the Lower Floridan Aquifer (LFA) which are separated by a semiconfining unit. Pumping from this aquifer at various locations has already lowered groundwater levels resulting in encroachment of seawater into the aquifer at the northern end of Hilton Head Island, South Carolina (Clarke and Krause, 2000; Garza and Krause, 1996) . Saltwater contamination has constrained further development of the UFA in the coastal area and created competing demands for the limited supply of water (Leeth, Clarke et al., 2003). However, since the UFA is the primary source of water in the region, various categories of users routinely apply to the State of Georgia Environmental Protection Division (EPD) for groundwater withdrawal permits. While supplying the water demand in the region, EPD's goal (EPD, 1997; EPD, 2001) is to avoid further aggravation of the saltwater encroachment at Hilton Head Island (hereafter referred as the indicator site).

4.4.1 Study Area, Model Domain and the Hypothetical Case Studied

To assist in evaluating and planning the future water supply in the area, the U.S. Geological Survey (USGS) developed a groundwater simulation model, the Savannah Area Model, for the region (Garza and Krause, 1996). The Savannah Area Model utilizes the ModFlow simulation code (McDonald and Harbaugh, 1988). In the present study, this model and its database is used without modification utilizing Processing ModFlow (PMWIN) computational environment (Chiang and Kinzelbach, 2000). The USGS Savannah Area Model treats the UFA and the LFA as two semiconfined aquifer layers while the unconfined unit above the UFA is treated as a fixed head boundary layer (Garza and Krause, 1996). Steady-state simulations are conducted to best represent the long-term response of the aquifer system. The Savannah Area Model is extensively calibrated with field data to determine the aquifer parameters for the two-layer aquifer system (Clarke and Krause, 2000; Garza and Krause, 1996) and it is a well-accepted management tool for the region.

Development of the Floridan aquifer system as a fresh water supply source began in about 1880. Garza and Krause (1996) recorded a total extraction rate of 5.266 m^3/sec of which 4.784 m^3/sec is attributed to the UFA. More recent pumping rates in the region which are also used in this study are identified as 4.473 m^3/sec from the UFA and 0.486 m^3/sec from the LFA. In the present study, our concern is limited to the assessment of groundwater resources in the vicinity of the Savannah-Hilton Head Island area. The model domain used here, which covers an area of 17301.2 km^2, is the same as the one used in Garza and Krause (1996). In the Savannah Area Model, a 76 element by 88 element finite difference grid with square elements is used to idealize the region. The tranmissivities in the UFA and the LFA range from 80.359 m^2/day to 19,044.5 m^2/day and 185.8 m^2/day to 7627.09 m^2/day, respectively. Other aspects of the numerical model used in this study, the database, and the numerical model used to solve the system are identical to those used for the Savannah Area Model (Garza and Krause, 1996). A detailed summary of the hydrogeology of the region is provided in Kentel et al. (2005a).

Hypothetical Case

It is assumed that there are six groundwater withdrawal permit applications in the Savannah region for the next planning period. These permit applications are at Rincon, Bloomingdale, Marlow, Ridgeland, Denmark, and Hinesville (referred to as demand locations). Bloomingdale, Denmark, and Hinesville apply for additional groundwater withdrawal rates of 0.0876 m^3/sec (2 *MGal/day*) while Marlow and Ridgeland apply for 0.0438 m^3/sec (1 *MGal/day*).

In order to present how the probabilistic water demand of a city may be included into the analysis, it is assumed that the City of Rincon has conducted an extended probabilistic population forecast study and evaluated its additional groundwater demand using a stochastic model. Accordingly, discrete demands and their associated probabilities for Rincon are as follows: 0.0657 m^3/sec, 0.057 m^3/sec, and 0.0482 m^3/sec with probabilities of 0.7, 0.2, and 0.1, respectively. The procedure for calculation of the specific demand at Rincon is explained in the following paragraphs.

In the literature, various probabilistic population forecasting methods have been developed (Alho, 1997; Alho and Spencer, 1985; Lutz, Sanderson et al., 2001; Sanderson, Scherbov et al., 2003). These probabilistic population forecasting methods may be used to determine the probabilistic water demands of a city. When the demand is a discrete stochastic process, each discrete demand is associated with a probability of occurrence and an expected value of the demand needs to be calculated. For a total of q states, let Dem represent the set of possible discrete demand values, $Dem = \{Dem\_1, Dem\_2, ..., Dem\_q\}$. Each element in set Dem, Dem_j, is associated with a probability of occurrence, $p(Dem\_j)$, such that the summation of the probabilities over all the states in the set Dem is equal to one. The expected value of the demand i, Dem_i, is calculated using the following equation:

$$Dem_i = \sum_{j=1}^{q} Dem\_j \times p(Dem\_j) \tag{25}$$

where i represents the demand location where the groundwater demand is provided as a discrete stochastic process. According to the above described procedure, the expected value of the demand at Rincon can be calculated as 0.0622 m^3/sec.

4.4.2 Coupled Simulation-Optimization Model

In coastal aquifers one of the major limitations for groundwater withdrawal is the saltwater intrusion problem. In this study, drawdown at the indicator site, which is a confirmed location of saltwater intrusion, is used as the constraint of the optimization model. In two recent USGS studies (Clarke and Krause, 2000; Garza and Krause, 1996), and in all of the Interim Strategies developed by EPD a drawdown less than 1.524 *cm* (0.05 *ft*) in the UFA at the northern end of Hilton Head Island is considered as acceptable.

Numerous simulations with both a single pumping well and multiple pumping wells are conducted and it is observed that drawdown at the indicator site changes linearly with pumping. Thus, in this discussion the response matrix approach is used to embed the results of the simulation model into the optimization model while handling the drawdown constraint at the northern end of Hilton Head Island. The response matrix approach is based on the principle of superposition. It is applicable when the system response is linear or approximately linear and the boundary conditions are homogeneous (Das and Datta, 2001b). This approach has been used by numerous researchers including Aral (1989), Heidari (1982), Zhou et al. (2003), Gorelick and Remson (1982), Willis and Finney (1985), and Ahlfeld et al. (2005).

The objective of the optimization model is to maximize the additional groundwater withdrawal while providing equal opportunity to each demand location without increasing the risk of saltwater intrusion at the indicator site. Based on this criterion, the proposed optimization model can be described as:

$$\text{Maximize: } \sum_{i=1}^{N}\left[Q_i - w_i \times \left(Q_{ave} - Q_i\right)^2\right]$$

$$\text{Subject to: } \left.\begin{array}{l} s_{HH}\left(Q_i\right) \le s_{\max} \\ 0 \le Q_i \le Q_{\max} \end{array}\right\} \quad i = 1,2,3,\ldots,N \tag{26}$$

where Q_i is the pumping rate in m^3/sec at the well i, N is the total number of hypothetical wells, Q_{ave} is the ideal average pumping rate in m^3/sec, w_i is the scaling factor, s_{HH} is the total drawdown in *cm* at the indicator site due to pumping at hypothetical wells and it is a linear function of Q_i, $s_{\max}$ is the maximum drawdown that is allowed at the indicator site, and $Q_{\max}$ is the maximum pumping rate that can be assigned to a pumping well. In our example, N is 6, $s_{\max}$ is 1.524 cm, and $Q_{\max}$ is 0.438 m^3/sec.

The ideal average pumping rate, Q_{ave}, is approximated as:

$$Q_{ave} = \sum_{i=1}^{N} Q_i \Big/ N \tag{27}$$

The objective function used in Equation (26) is composed of two terms. The first term, Q_i, maximizes total pumping. The second term, $w_i \times \left(Q_{ave} - Q_i\right)^2$, is a penalty term. The function of this term is to penalize any pumping rate which is different from Q_{ave}, so that optimum pumping rates from the hypothetical wells are forced to be as close as possible to each other. As expected, if the penalty term is not used, uniform groundwater extraction rates can not be obtained since the drawdown constraint favors pumping from hypothetical wells which are further inland from the indicator site. In order to have control on the magnitude of penalty imposed, a scaling factor, w_i, is introduced into the penalty term.

In this study, the impact of using a constant value (i.e., $w_i = 1; i = 1,2,3,\ldots,N$) and various functions as scaling factors is also investigated. The scaling factor, w_i is defined as a function of the distance between the hypothetical well and the indicator site. The function proposes for w_i can be defined as follows:

$$w_i = \left(B - \frac{d_i}{d_{\max}} \right) \tag{28}$$

where B is a constant greater than one, d_i is the distance between the hypothetical well i and the indicator site, and $d_{\max}$ is the maximum of all d_i's.

For a given B the scaling factor given in Equation (28) increases as the distance between the hypothetical well and the indicator site decreases. Thus, pumping rates different than Q_{ave} at wells close to the indicator site are penalized more. Due to the drawdown constraint, optimum additional pumping rates from hypothetical wells close to the indicator site are expected to be lower than Q_{ave}. Thus, pumping less than Q_{ave} at wells close to the indicator site will be highly penalized. By choosing different values for B in Equation (28), the scaling factor can be adjusted and uniform pumping rate distributions can be obtained in the region. This is achieved by shifting some of the additional available groundwater from wells located at the outer zones (i.e., inland locations) towards wells that are close to the indicator site. The coupled simulation-optimization model is used to calculate optimum additional withdrawal rates for the hypothetical case in the Savannah region.

4.4.3 Optimum Additional Withdrawal Rates for the Hypothetical Case

The optimization model proposed in Equation (26) is used to determine optimum additional withdrawal rates from six demand locations. It is assumed that all groundwater withdrawal is from the UFA. A discussion of other cases can be found in (Kentel, Gill et al., 2005b). Initially all w_i's are selected as one and optimum additional pumping rates are calculated. GAMS software (Brooke, Kendrick et al., 1998) is used to solve the optimization model. Optimum additional pumping rates, which can be withdrawn from the UFA at six demand locations using $w_i = 1, \forall i$ are given in Figure 5.5. The radius of the circle around the demand location in this figure is proportional to optimum additional pumping rate at those locations and the numbers in each circle are the optimal pumping rates in *m³/sec*. The total amount of additional groundwater withdrawn from the UFA when $w_i = 1, \forall i$ is approximately 0.259 m^3/sec. The combined withdrawal from Denmark and Hinesville accounts for approximately half of this total amount (i.e., 0.123 m^3/sec). The minimum amount of additional groundwater extraction is assigned to Ridgeland whereas the maximum amount is assigned to Hinesville. As can be seen in Figure 4.5, the optimum additional amount of groundwater extraction at a demand location decreases as the distance between the indicator site and the demand location decreases. The results obtained by using $w_i = 1, \forall i$, Figure 4.2, may be interpreted as "unfair" or "non-uniform." In order to investigate the effect of different scaling factors on optimum additional pumping rates, other values of B can be selected $\{B = 2, 4, \text{and}, 6\}$ in

Equation (28). Optimum additional pumping rates from the UFA obtained when these different w_i's are given in Figure 4.6.

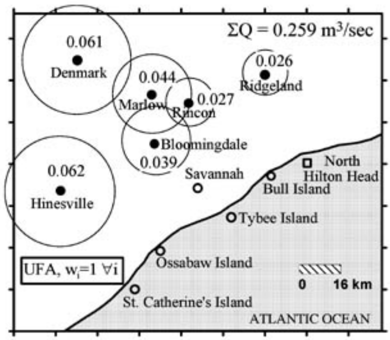


Figure 4.5: Optimum additional pumping rates (m3/sec) from the UFA for six demand locations using $w_i = 1, \forall i$

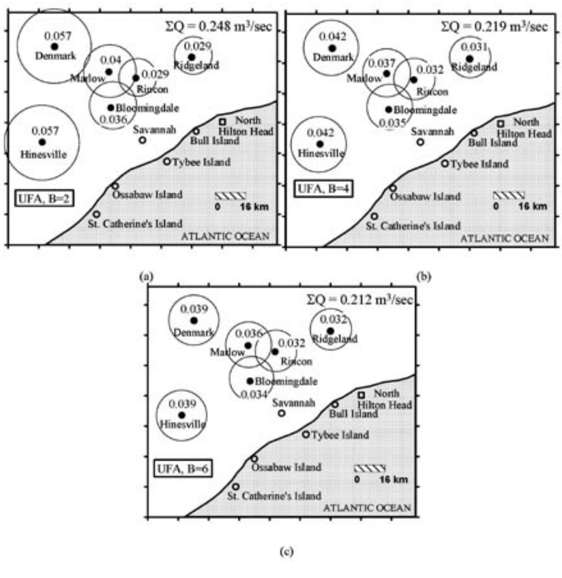


Figure 4.6: Optimum additional pumping rates (m^3/sec) from the UFA for six demand locations using $(a)\, B = 2; (b)\, B = 4; (c)\, B = 6$

4.4.4 Fuzzy Multi-Objective Decision-making Framework

The LFA is identified as one of the alternative water sources in the coastal area (EPD, 2001). Thus, it is possible to identify three management strategies for this study: additional groundwater can be supplied from: (i) the UFA; (ii) the LFA; and, (iii) the UFA and the LFA together. In order to evaluate the impact of the penalty term in the decision-making process, four sub-strategies are considered (Kentel and Aral, 2007). Thus, a total of 12 management strategies, Table 4.2, are evaluated using the coupled simulation-optimization model.

Optimum additional pumping rates for management strategies which consider groundwater withdrawal from the LFA and UFA+LFA are provided in Figures 4.7 and 4.8, respectively. In Figure 4.8, groundwater withdrawal rates from the LFA and from the UFA are plotted separately. A comparison of these results indicates that optimum additional pumping rates from the UFA and the LFA are very similar to each other. This may be due to a number of hydrologic factors including, but not limited to the leaky nature of the confining unit.

Table 4.2: Management strategies

	Aquifer groundwater is extracted from	Penalty term, w_i
MS_1a*	UFA[1]	$w_i = 1;\ i = 1,2,\ldots,6$
MS_1b	UFA	$w_i = (2 - d_i / d_{\max})$
MS_1c	UFA	$w_i = (4 - d_i / d_{\max})$
MS_1d	UFA	$w_i = (6 - d_i / d_{\max})$
MS_2a	LFA[2]	$w_i = 1;\ i = 1,2,\ldots,6$
MS_2b	LFA	$w_i = (2 - d_i / d_{\max})$
MS_2c	LFA	$w_i = (4 - d_i / d_{\max})$
MS_2d	LFA	$w_i = (6 - d_i / d_{\max})$
MS_3a	UFA+LFA	$w_i = 1;\ i = 1,2,\ldots,6$
MS_3b	UFA+LFA	$w_i = (2 - d_i / d_{\max})$
MS_3c	UFA+LFA	$w_i = (4 - d_i / d_{\max})$
MS_3d	UFA+LFA	$w_i = (6 - d_i / d_{\max})$

* Management Strategy_1a; [1] Upper Floridan Aquifer; [2] Lower Floridan Aquifer

As can be observed in Figures 4.5, 4.6, 4.7, and 4.8, optimum additional pumping rates at Rincon and Ridgeland increase while the pumping rates from the other demand locations decrease as *B* increases. However, it should be noticed that the rate of increase at Rincon and Ridgeland is smaller compared to rates of decrease at Bloomingdale, Marlow, Denmark, and Hinesville. For example, increasing *B* from 2 to 4 results in a total increase of 0.006 m^3/sec at Rincon and Ridgeland when water is pumped from the LFA. However, the combined decrease in pumping rate at the other four demand locations is 0.034 m^3/sec. This shows that in order to keep the drawdown at the northern end of Hilton Head Island below 1.524 *cm*, the slight increases in pumping rates at Rincon and Ridgeland must be balanced by substantially larger decreases in the pumping rates at the other four demand locations.

Optimum additional groundwater withdrawal rates at demand locations and the effect of additional withdrawal on hydrological conditions in the region are used to evaluate the individual satisfaction of each management strategy with respect to a set of fuzzy objectives. Then, these individual satisfaction degrees are aggregated into a single overall performance value which will be used in selecting the best management strategy.

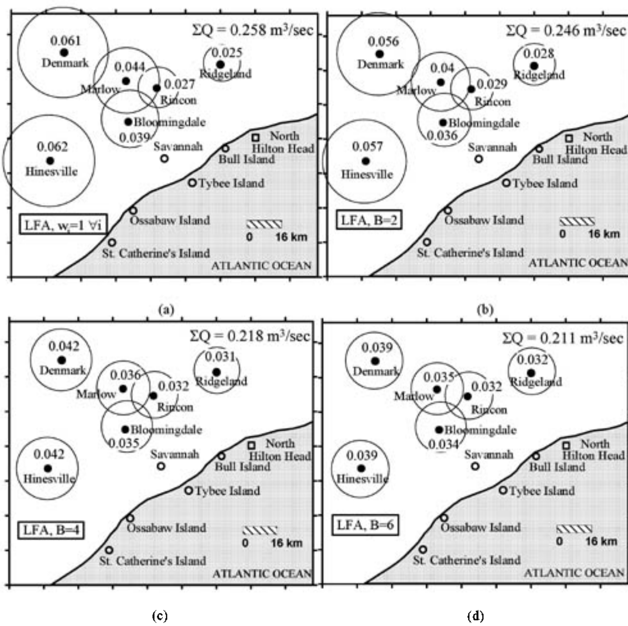


Figure 4.7: Optimum additional pumping rates (m3/sec) from the LFA for six demand locations using $(a)\, w_i = 1, \forall i; (b)\, B = 2; (c)\, B = 4; (d)\, B = 6$

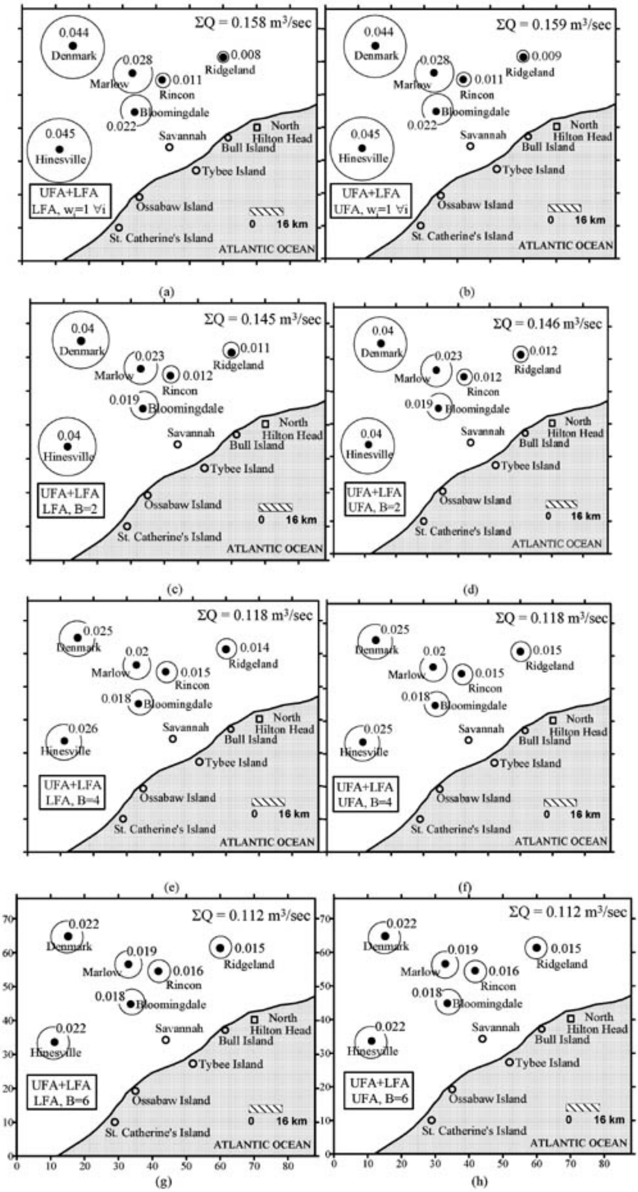


Figure 4.8: Optimum additional pumping rates (m3/sec) from UFA+LFA for six demand locations using

$(a)\,LFA, w_i = 1, \forall i; (b)\,UFA, w_i = 1, \forall i; (c)\,LFA, B = 2; (d)\,UFA, B = 3;$
$(e)\,LFA, B = 4; (f)\,UFA, B = 4; (g)\,LFA, B = 6; (h)\,UFA, B = 6$

The process for selecting the best management strategy can be summarized as follows: Identify fuzzy objectives, i.e., F_k; $k=1,2,3...m$, where m is the total number of fuzzy objectives; Determine the membership value of each management strategy for each fuzzy objective (i.e., individual satisfaction degree of each management strategy for each fuzzy objective) i.e., $\mu_{s,k}, s = 1,2,3,...,r$ and $k = 1,2,3,...,m$ where r is the total number of strategies; Calculate an overall representative degree of performance for each management strategy with respect to all fuzzy objectives, i.e., $D_s, s = 1,2,3,...,r$; and, Choose the best management strategy, i.e., the management strategy with the highest D_s.

Degrees of Satisfaction of the Management Strategies for Each Fuzzy Objective

Management objectives need to be determined by the managers. For this hypothetical study, we have selected the following four fuzzy objectives as the additional critical goals which need to be considered for the Savannah area: Maintain high satisfaction of the sum of individual demands, $HSD=F_1$; Maintain low drawdown at critical locations other than Hilton Head Island (i.e., Tybee Island and Bull Island), $LD=F_2$ (note: critical drawdown condition at the indicator site is already satisfied for all cases based on our optimal solution methodology); Maintain fair groundwater withdrawal (i.e., uniform additional groundwater withdrawal from demand locations), $FW=F_3$; and, Maintain low cost, $LC=F_4$. These fuzzy objectives represent our interpretation of the hydrological, political, and economical concerns for the region. Although the utilization of the proposed analysis is demonstrated by using these four fuzzy objectives, the proposed approach is general and can be applied to other fuzzy objectives with relative ease. The four fuzzy objectives are explained in more detail in the following paragraphs.

i. Maintain high satisfaction of the sum of individual demands: $HSD=F_1$

Each groundwater permit applicant demands a certain amount of additional groundwater, Dem_i, which will be pumped from the Floridan aquifer system. The subscript i refer to a permit applicant. As a result of the coupled simulation-optimization model, the optimum additional groundwater withdrawal, Q_i in m^3/sec for $i = 1,2,3,...,N = 6$ is calculated. As an indicator of the overall satisfaction of the management strategy, the sum of the percent individual satisfactions at each demand location is defined as follows:

$$Satisfaction = \sum_{i=1}^{N} \frac{Q_i}{Dem_i} \times 100 \qquad (29)$$

If the calculated optimum additional amount of groundwater is higher than or equal to the demand (i.e., $Q_i \geq Dem_i$) the percent satisfaction of that demand location is taken as 100%. The satisfaction value calculated by Equation (29) will range between 0 and $(N \times 100)$. Thus the range 0-600 is used as the domain of the fuzzy objective of high satisfaction of the sum of individual demands for the six demand locations used in this study. The membership function of $HSD=F_1$ is given in Figure 4.9(a).

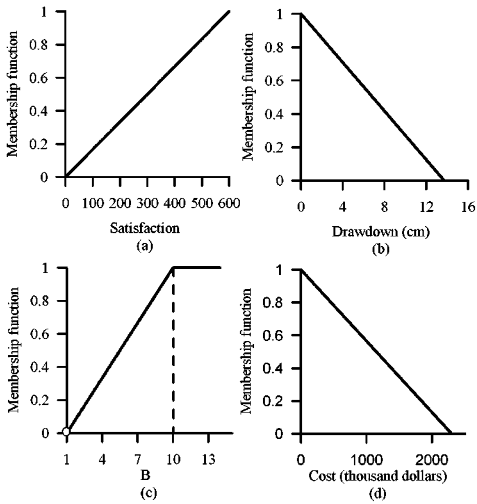


Figure 4.9: Membership functions of (a) high satisfaction, *HSD*; (b) low piezometric head decline, *LD*; (c) fair groundwater withdrawal, *FW*; (d) low cost, *LC*

The expected value of demand at Rincon calculated by Equation (25) as 0.0622 m^3/sec is used in Equation (29) as the $Dem_i, i = \text{Rincon}$. All other demands are provided as deterministic values, $Dem_{\text{Bloomingdale}} = Dem_{\text{Denmark}} = Dem_{\text{Hinesville}} = 0.0876\, m^3/sec$, and $Dem_{\text{Marlow}} = Dem_{\text{Ridgeland}} = 0.0438\ m^3/sec$, which can be used directly in Equation (29).

ii. Maintain low drawdown at critical locations other than Hilton Head Island: $LD=F_2$

The northern end of Hilton Head Island is the confirmed location of saltwater intrusion in the Savannah region (Clarke and Krause, 2000; Clarke and Krause, 2001a; EPD, 1997; Garza and Krause, 1996). Further south, near the eastern end of Bull Island in South Carolina, geological conditions are favorable for saltwater to enter the aquifer as well (EPD, 1997). According to USGS and EPD studies, some wells in this area show higher than expected salinity levels. Based on groundwater modeling, the USGS reports that saltwater may also be entering the aquifer offshore from Tybee Island (EPD, 1997). These two locations, Bull Island and Tybee Island, can also be identified as critical locations at which saltwater intrusion may occur. Thus, the decision maker may prefer a management strategy which provides low drawdown in the UFA at Bull Island and Tybee Island as well. Optimum additional pumping rates at each one of the six demand locations are entered into the simulation model, and drawdown at Bull Island and Tybee Island is calculated with these additional pumping rates, Table 4.3.

Table 4.3: Drawdown (cm) in the Upper Floridan Aquifer at Bull Island and Tybee Island with optimum additional pumping rates at six demand locations

Management Strategy	Drawdown (cm)	
	Bull Island	Tybee Island
MS_1a*	8.84	12.50
MS_1b	8.84	11.89
MS_1c	8.84	11.28
MS_1d	8.53	11.28
MS_2a	8.84	12.50
MS_2b	8.84	11.89
MS_2c	8.84	11.28
MS_2d	8.53	11.28
MS_3a	9.14	13.72
MS_3b	8.84	12.80
MS_3c	8.84	11.89
MS_3d	8.84	11.58

As can be seen from Table 4.3, the largest drawdown is 13.72 *cm*. Thus, the domain for the fuzzy objective for low drawdown is chosen as 0 cm to 13.72 cm, and a linearly decreasing function is used as the membership function for *LD*, Figure 4.6(b). A drawdown of zero *cm* fully belongs to the fuzzy set of *LD* and any drawdown greater than 13.72 *cm* does not belong to the fuzzy set of *LD*.

iii. Maintain fair groundwater withdrawal rates for all users in the region: *FW*

As described earlier, the B value in the scaling factor w_i in Equation (4.28) can be used to adjust the degree of penalizing for non-uniform pumping rates. The higher the B value, the more uniform the optimized pumping rates are in the region. In this study, uniform withdrawal is identified as "fair" management strategy for all permit applications. The impact of various w_i's on optimum additional pumping rates at six demand locations for the LFA, the UFA, and UFA+LFA are given in Figures 4.10 (a), (b), and (c) respectively.

As can be seen in Figure 4.10, for each management scenario the optimum additional pumping rate at each demand location converges rapidly to a common optimal pumping rate, a value slightly less than 0.035 m^3/sec, as B increases to 10. It should be noted that substrategies with $w_i = 1, \forall i$ do not have a scaling factor which include a B value, and are not plotted in Figure 4.10. However, choosing $w_i = 1, \forall i$ results in pumping rates, which are "not fair" when compared to $B = 2, 4, ..., 25$ cases. Thus, we identify the substrategies obtained from $w_i = 1, \forall i$ as the substrategy, which does not belong to the fuzzy set of fair groundwater withdrawal. For the remainder of the substrategies, the B value is used as an indicator for fairness. The membership function for fair groundwater withdrawal is given in Figure 4.9(c).

iv. Maintain low cost: *EC*

Optimum additional pumping rates at each demand location vary for each management scenario. For example, the optimum additional pumping rate for the management strategy MS_1a at the city of Denmark is 0.061 m^3/sec while at the same location it is only 0.039 m^3/sec for MS_1d. To generate "fair groundwater withdrawal" strategies, higher B values are used in the optimization model. This results in lower optimum additional pumping rates at Denmark but more uniform groundwater withdrawal rates for the region. Thus, although it is hydraulically possible to grant a pumping rate of 0.061 m^3/sec to Denmark (MS_1d), lower optimum additional pumping rates are assigned to Denmark for management strategies other than MS_1d. The difference between the maximum additional pumping rate, which can be granted to Denmark and the optimum additional pumping rate granted as a result of one of the other management strategies can be identified as the deficit. This deficit may be used as an indication of the additional cost for water supply at Denmark. Because decision makers have no control on the demand, which is requested by the permit applicant we choose to define the deficit as identified above, and used this definition of deficit to determine the additional cost each management strategy may bear. It should be noted here that the maximum additional amount, which can be granted is calculated as a result of the optimization analysis.

Let us identify the optimum additional pumping rate at a demand location, i, obtained for the management strategy, j, as "Q_i of MS_j". The maximum of optimum additional pumping rates at demand location, i, can be calculated by:

$$Q_i^{\max} = \max_{j=1a,1b,\ldots,3d} \{Q_i \text{ of } MS\_j\} \tag{30}$$

Then the deficit, $Deficit_i$, in m^3/sec at demand location, i, can be calculated by:

$$Deficit_i = Q_i^{\max} - Q_i \tag{31}$$

The deficit has to be supplied by some means other than groundwater at the demand location i, and this will cost additional money for the applicant. Other means may be purchasing water from another municipality or using surface water with necessary water quality treatment, etc. At each demand location, the alternative source of water may be different, and may cost different amounts. If we call the cost of one cubic meter of water from the alternative source at demand location i as c_i, then the total cost in dollars per second for each management strategy can be approximated by:

$$Total\ Cost = \sum_{i=1}^{N} c_i \times Deficit_i \tag{32}$$

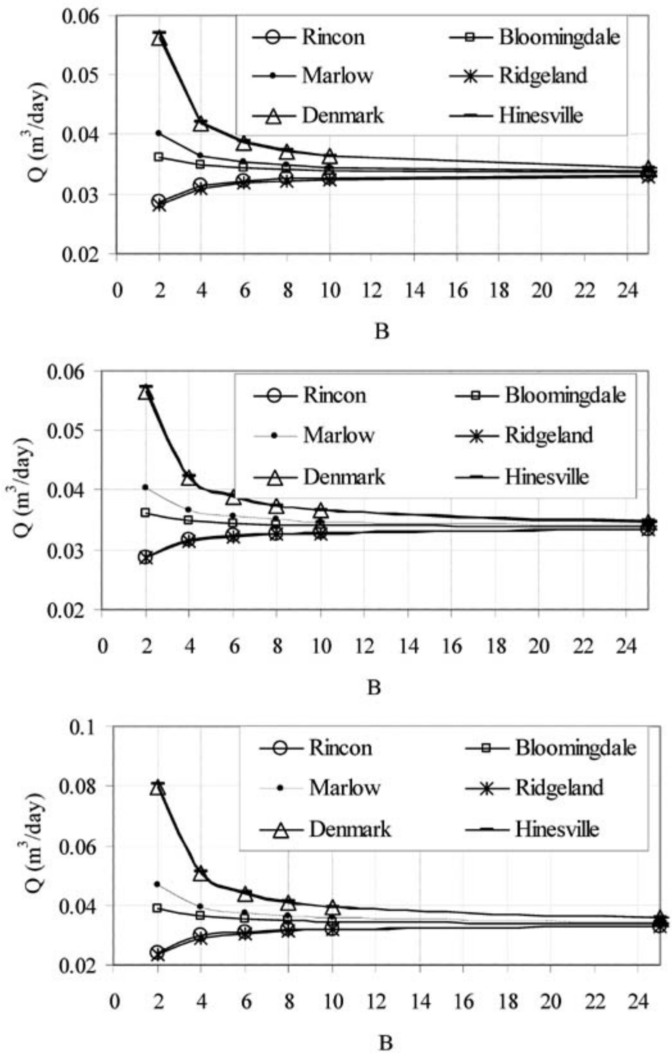


Figure 4.10: Change of optimum additional pumping rates (m3/sec) with w_i for (a) LFA; (b) UFA; (c) UFA+LFA

where $N = 6$. Since we are working with a hypothetical example, the same c_i value of \$0.55 per m^3 of water is used to calculate the cost for all six demand locations. The annual cost for each management strategy is given in Table 4.4.

The highest annual cost is \$ 2,272,180 for MS_2d. Considering the results provided in Table 4.4, a range of 0 to \$ 2,300,000 is used as the domain of the fuzzy set low cost. A linearly decreasing membership function is assigned to the fuzzy set, Figure 4.9(d). The membership function given in Figure 4.9(d) is used to determine the satisfaction degree of each management strategy for the fuzzy objective of maintenance of low cost.

The second step in the decision-making process is to determine the membership values (i.e., individual satisfaction degrees) of the management strategies for each of the fuzzy objectives. The individual satisfaction degrees are determined using previously defined membership functions of the fuzzy objectives, Figure 4.9, and are given in Table 4.5.

Table 4.4: Annual cost for each management strategy

	Total cost (thousand dollars)
MS_1a*	1447.66
MS_1b	1637.64
MS_1c	2133.12
MS_1d	2248.63
MS_2a	1462.10
MS_2b	1658.16
MS_2c	2155.16
MS_2d	2272.18
MS_3a	460.516
MS_3b	845.80
MS_3c	1839.03
MS_3d	2070.80

Table 4.5: Individual satisfaction degrees for each management strategy with respect to the fuzzy objectives, $\mu_{s,k}$

Fuzzy Objectives		Membership function, $\mu_{s,k}$, $s = 1,2,..,6$ and $k = 1,2,3,4$					
		MS_1a* s = 1	MS_1b s = 2	MS_1c s = 3	MS_1d s = 4	MS_2a s = 5	MS_2b s = 6
$k = 1$ (*HSD*)		0.65	0.62	0.57	0.56	0.64	0.62
$k = 2$	Tybee Island	0.09	0.13	0.18	0.18	0.09	0.13
(*LD*)	Bull Island	0.36	0.36	0.36	0.38	0.36	0.36
$k = 3$ (*FW*)		0.00	0.11	0.33	0.56	0.00	0.11
$k = 4$ (*LC*)		0.37	0.28	0.07	0.02	0.36	0.28

Table 4.5 continued

Fuzzy objectives		Membership function, $\mu_{s,k}$, $s = 7,8,..,12$ and $k = 1,2,3,4$					
		MS_2c s = 7	MS_2d s = 8	MS_3a s = 9	MS_3b s = 10	MS_3c s = 11	MS_3d s = 12
$k=1$ (*HSD*)		0.57	0.55	0.75	0.71	0.60	0.58
$k=2$ (*LD*)	Tybee Island	0.18	0.18	0.00	0.07	0.13	0.16
	Bull Island	0.36	0.38	0.33	0.36	0.36	0.36
$k=3$ (*FW*)		0.33	0.56	0.00	0.11	0.33	0.56
$k=4$ (*LC*)		0.06	0.01	0.80	0.63	0.20	0.10

* Management Strategy_1a; *HSD*: Maintain high satisfaction of the sum of individual demands; *LD*: Maintain low drawdown at critical locations; *FW*: Maintain fair groundwater withdrawal *LC*: Maintain low cost.

Selecting the Best Management Strategy

Evaluation of the overall performance of multiple fuzzy objectives can be performed using the aggregation approach. Detailed information about fuzzy sets and aggregation operators can be found in Zimmerman (1985), Dubois and Prade (1988), Kaufmann and Gupta (1988) and Słowiński (1998). The aggregation process can be realized by using various aggregation operators identified as conjunctive, disjunctive, or averaging operators. Here, we chose to use an averaging operator, the ordered weighted averaging (OWA) operator, to aggregate individual satisfaction degrees provided in Table 4.5 into a single overall performance value for each management strategy. A brief explanation of the OWA operator is provided in the Appendix.

The quantifier guiding the aggregation is selected as "most" and it is defined by $Q(r) = r^2$. This translates into "the decision maker desires to satisfy most of the fuzzy objectives." Evaluation of management strategies when all fuzzy objectives are considered is provided below (maintenance of low drawdown at Tybee and Bull Islands are considered as two separate fuzzy objectives):

$$\begin{aligned}
v_1 &= Q\left(\frac{1}{5}\right) - Q\left(\frac{1-1}{5}\right) = Q(0.2) - Q(0) = 0.04 \\
v_2 &= Q\left(\frac{2}{5}\right) - Q\left(\frac{2-1}{5}\right) = Q(0.4) - Q(0.2) = 0.12 \\
v_3 &= Q(0.6) - Q(0.4) = 0.2 \\
v_4 &= Q(0.8) - Q(0.6) = 0.28 \\
v_5 &= Q(1.0) - Q(0.8) = 0.36
\end{aligned} \tag{33}$$

The overall performances are calculated as follows:

$$D_{1a} = 0.04 \times 0.65 + 0.12 \times 0.37 + 0.2 \times 0.36 + 0.28 \times 0.09 + 0.36 \times 0.0 = 0.17$$

similarly

$$\begin{array}{llll} D_{1b} = 0.20 & D_{1c} = 0.21 & D_{1d} = 0.22 & D_{2a} = 0.17 \\ D_{2b} = 0.20 & D_{2c} = 0.20 & D_{2d} = 0.22 & D_{3a} = 0.19 \\ D_{3b} = 0.23 & D_{3c} = 0.24 & D_{3d} = 0.24 & \end{array} \tag{34}$$

As can be seen from Equation (4.34), the best management strategies are MS_3c and MS_3d. However MS_3b perform almost as good as MS_3c and MS_3d. Actually, all the management strategies have overall satisfactions around 0.2. This may not always be the case. OWA operator does not always aggregate individual satisfactions of various alternatives into single overall performances which are close to each other. The membership functions we chose for the fuzzy objectives have a significant impact on the individual satisfaction degrees. For site specific situations, these membership functions have to be determined by experts considering environmental and political requirements, and time and money constraints.

Comparison of Results for Conjunctive, Disjunctive, and Averaging Operators

In order to demonstrate how the overall performance values change with respect to various aggregation operators, the analysis provided in the previous section is conducted for a conjunctive operator, "and" and a disjunctive operator, "or." The results are provided in Figure 4.11.

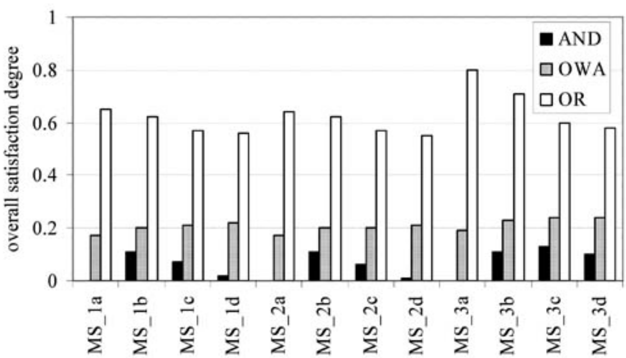


Figure 4.11. Comparison of overall satisfaction degrees when "and", "or", and "OWA" are used as aggregation operators

The conjunctive operator aggregates the criteria by a logical "and." Thus, the overall performance is high if and only if all the individual performances are high. The conjunctive operator does not have a compensation mechanism. If one of the individual performances is low, then the overall performance of the management strategy is low. Thus, the conjunctive aggregator does not provide an average overall performance but rather it provides the worst degree in which a management strategy will respond to a set of fuzzy objectives. On the other hand, disjunctive operators perform aggregation where criteria are combined by a logical "or." In this case, the overall performance is high when at least one of the individual performances is high. The overall performance for aggregation with disjunctive operator is low if and only if all the individual performances are low. Unlike the conjunctive operator, the disjunctive operator does not punish the management strategy for low individual satisfaction as long as one of the fuzzy objectives has high individual satisfaction. Thus, disjunctive operators are useful when ruling out alternatives. To summarize, the conjunctive operator is useful for situations in which all the criteria should be satisfied, while the disjunctive operator is useful for cases in which the satisfaction of any of the criteria is sufficient.

Overall satisfaction degrees obtained by using "and" as the aggregation operator are respectively low while those obtained using "or" are much higher. Since OWA operator compensates the low individual satisfactions with the higher ones while aggregating individual satisfaction degrees of fuzzy objectives, the overall performances obtained by OWA operator lie in between the results obtained by "and" and "or" aggregation operators. In evaluating groundwater resources management alternatives with respect to hydraulically, politically, and economically motivated objectives, OWA operator which utilizes some sort of averaging mechanism produces more reasonable results when compared to "and" and "or" as aggregation operators.

4.5 CONCLUSIONS

In a coastal aquifer management problem, the two main objectives are maximizing the pumping while placing the pumping wells as close as possible to the coast as the solution minimizes the saltwater intrusion to the aquifer. In this chapter the current literature on this aquifer management problem is reviewed. The review indicates that solution of this problem can be obtained using a wide range of methodologies. Each approach has its deficiencies as well as advantages. The choice of the use of these techniques will depend on the purpose of the study at hand.

In the first part of this chapter a solution to this management problem is discussed, where the two objectives are combined into a single scalar objective function and analytical models are employed in the solution of the problem. The use of the proposed methodology is demonstrated on a hypothetical coastal aquifer. The results obtained for maximizing pumping rates and minimizing the distance between the well locations and the shoreline are encouraging and the proposed algorithm has relatively low computational burden. The proposed model is capable of performing multi-objective analysis as demonstrated in the literature cited.

In the second methodology, a robust, objective, and systematic approach to determine the best groundwater management strategy among alternatives is discussed. The use of this approach is demonstrated for the evaluation of the optimum additional groundwater supply potential in the Savannah region for simultaneous pumping at multiple demand locations. The proposed approach allows various forms of uncertainties to be included in the decision-making process that employ "fuzzy" algebra. In this approach, hydrological, political, and economical goals which are more conveniently represented by fuzzy sets can be used in the decision-making process. The final evaluation of the overall performance with respect to a multiple fuzzy objectives alternatives can be performed using various aggregators which result in different "best management" solutions. Here the final choice among the alternatives obtained is the decision of the manager, but the choice would be transparent and based on sound evaluation of uncertain "fuzzy" goals. The use of an OWA operator to aggregate the fuzzy objectives of the groundwater management problem considered in this study seems beneficial and leads to more informed outcomes.

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CHAPTER 5

The Impact of Urbanization of Wekiva Springshed on Groudwater

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ABSTRACT: Springs have historically played an important role in Florida's history and the Wekiva River is a spring-fed system associated with many, possibly 23 springs connected to the Floridan aquifer. Maintaining groundwater recharge to the aquifer is a key factor of the viability of the regional water supply as well as the Wekiva ecosystem. A first principle, physics-based watershed model WASH123D (A Numerical Model Simulating Water Flow and Thermal, Salinity, Sediment, and Water Quality Transport in WAterSHed Systems of 1-D Stream/River Network, 2-D Overland Regime, and 3-D Subsurface Media) has been applied to conduct a study of the Wekiva springshed, which is the recharge area of the spring. We first introduce the basic hydrogeologic characteristics of the study area. The mathematical basis and numerical approximations of WASH123D are presented in succession. The development of the Wekiva Springshed Model using WASH123D is detailed followed by a discussion of hydrologic data input. The Wekiva WASH123D model was run to evaluate the average, steady state 1995 hydrological conditions. The distribution of simulated Floridan aquifer system groundwater levels using WASH123D shows very good agreement with the field observations at corresponding locations. Also identified are the areas of recharge to and discharge from the Floridan aquifer system as well as decreases of the spring discharge due to the urbanization.

5.1 INTRODUCTION

As Florida's population continues to grow, the underlying Floridan aquifer and connected springs are facing increasing pressures. This growth brings an inevitable rise in water use, as well as extensive land use changes. Water shortages could become a controlling factor in the location and timing of new development. Each year, lands within springsheds are developed, altering the quality and quantity of water flowing to the springs. Springs serve as windows into the quality of our groundwater, which continues to decline as development pressures increase.

The spring-fed system, Wekiva River and its tributaries, along with the St. Johns River and associated lands in Central Florida have long been recognized as one of the most valuable natural assets of the state. These areas, which include most of the Central Florida portion of St. Johns River Water Management District (SJRWMD), were designated based upon the likelihood of future water resource problems due to projected 2010 groundwater withdrawals.

The objective of this chapter is to develop a numerical modeling tool that will be capable of estimating the hydrologic characteristics of the fresh groundwater flow system in the Wekiva springshed region and the effect of urbanization on groundwater recharges. In this chapter, we first briefly introduce the basic hydrogeologic characteristics of the study area. The mathematical concepts of WASH123D (Yeh et al., 2005) are presented in succession. The hydrologic data input are then discussed followed by the development of the numerical model. The Wekiva WASH123D model was run to evaluate the average, steady state 1995 hydrological conditions. The distribution of simulated Floridan aquifer system groundwater levels using WASH123D shows very good agreement with the field observations at corresponding locations. Also identified are the areas of recharge to and discharge from the Floridan aquifer system. Decreases of the spring discharge due to the urbanization are discussed, and a relationship between distance and percentage of groundwater flow contribution to Rock Spring discharge is analyzed.

5.2 REGION OF STUDY

The region of study is essentially the same as the East Central Florida (ECF) model (McGurk and Presley, 2002) developed by the SJRWMD. It is centered upon Seminole and Orange counties but includes most of Brevard, Lake, and Osceola counties plus parts of Marion, Polk, and Volusia counties (Figure 5.1). The important climatic, topographic, and hydrogeologic characteristics of the ECF region, organized in a hydrogeologic framework, are discussed in this section.

5.2.1 Climate

The study area climate is humid and subtropical, with warm, relatively wet summers and mild, relatively dry winters (Tibbals, 1990). Most years have at least several days when the temperature drops below freezing, but minimum temperatures are rarely below –7°C and maximum temperatures are rarely above 38°C. Rainfall represents the largest input of water to the hydrologic system, and it is unevenly distributed. Approximately 60% of the annual rainfall occurs from June through October. Normal annual rainfall amounts measured within the region range from around 117 cm/yr (centimeter per year) to 142 cm/yr approximately.

Although evapotranspiration (ET) represents the largest water loss from the hydrologic system, there are few data available that represent direct ET measurements. Estimates of the upper and lower limits of average annual ET rates in the region have been made by Tibbals (1990). The upper limit is approximately equal to the rate at which water can evaporate from an open body of water. This limit ranges from 117 cm/yr in the Northeastern part of the ECF region to 124 cm/yr in the southwestern part (Tibbals, 1990). Estimates of minimum annual ET rate vary from 64 cm/yr to 89 cm/yr (Tibbals, 1990).

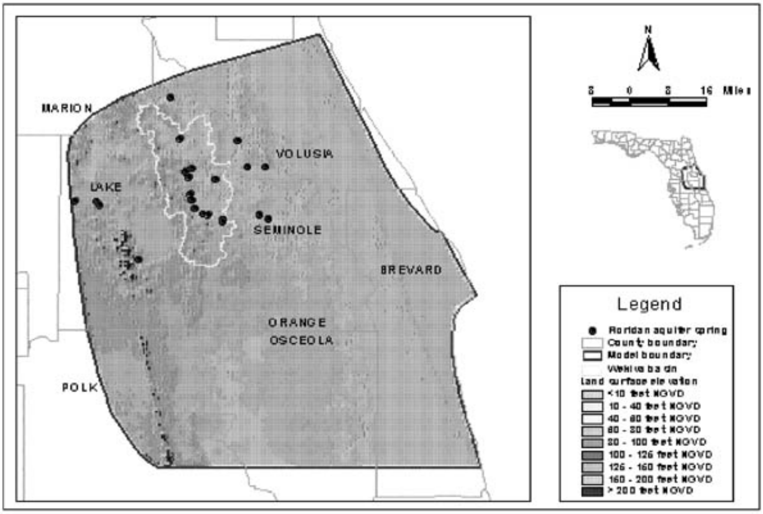

Figure 5.1: Land Surface Elevations and Floridan Aquifer Springs in the Modeling Area

5.2.2 Topography and Surface Water Features

Topographic relief and the nature of surface water features affect the distribution of recharge and discharge within the groundwater flow system. They are briefly described in this subsection.

The study region is approximately 26,000 square kilometers in area. Land surface elevations range from sea level at the coast to greater than 61 m (equals about 200 ft) above the National Geodetic Vertical Datum of 1929 (NGVD, formerly called mean sea level) at hilltops in Lake and Polk counties. In general, the topography increases in elevation in a step-wise fashion westward from the coast to highland areas in Lake, Polk, and western Orange counties (McGurk and Presley, 2002). Generally, the major topographic features are oriented in a coast-parallel or northwest to southeast direction (Figure 5.1).

The major surface water bodies within this area include rivers and their tributaries, canals, coastal lagoons, over 50 large lakes, numerous small storage ponds, 23 Floridan aquifer springs and over 5,000 wells. Long term flow measurement records indicate that the St. Johns, Ocklawaha, and Kissimmee rivers account for approximately 85% of the total surface water discharge within the region (USGS 1998).

There are hundreds of lakes that are not connected to the major surface water drainage systems and have no surface streams or canals flowing in or out of them. These seepage lakes are most numerous in the highland areas of Lake County, eastern Marion County, western Orange and Seminole counties, eastern Polk County, and western Volusia County. They range in size from less than 4,000 square meters to approximately several square kilometers and receive water from direct rainfall, overland runoff, and discharge from the surficial aquifer system. Seepage lakes are often sinkhole depressions that have filled with water. Water level fluctuations tend to be greater in seepage lakes located in upland areas than in other lakes because inflow from runoff and groundwater is relatively less constant.

5.2.3 Hydrogeological Features

The caustic and carbonate sediments beneath the area can be grouped into three aquifers (Surficial aquifer system, Upper Floridan aquifer, Lower Floridan aquifer) bounded by three confining layers (Intermediate confining unit, Middle semiconfining unit, Lower confining unit). These hydrostratigraphic units (see Figure 5.2) apply throughout the domain and their characteristics are described in this subsection.

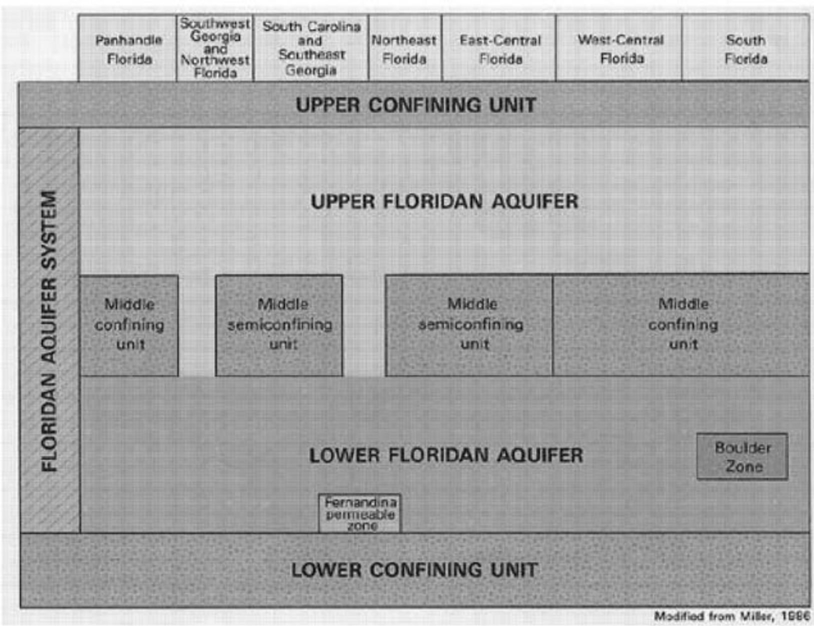

Figure 5.2 The Floridan Aquifer System

5.2.3.1 Surficial Aquifer System

The uppermost unit is the surficial aquifer system with the thickness ranging from less than 6 m to as much as 45 m. The top of this unit (the water table) is located from within about a meter to over ten meters below the land surface. The surficial aquifer system receives recharge mainly from rainfall, irrigation water, and the Floridan aquifer while the discharge occurs mainly due to the ET from the water table, seepage to surface water bodies and pumpage. Reported horizontal hydraulic conductivity of the surficial aquifer system-sediments ranges from 0.9 cm/day to 60 m/day. This layer consists of Pleistocene to Recent (Holocene) age sand, silt, clayey sand, and shell beds.

5.2.3.2 Intermediate Confining Unit

The intermediate confining unit separates the surficial aquifer system from the underlying Floridan aquifer system. The generalized thickness of the intermediate confining unit is from less than 15 m to over 60 m, increasing from north to south. This unit is believed to receive recharge from the surficial layers and supply discharge to the Floridan aquifer where the water table is higher than Floridan aquifer potentiometric surface. The estimated leakance (ratio of vertical conductivity to thickness of the intermediate confining unit) derived from aquifer tests ranges from 10^{-6}/day to 0.8/day. This layer consists of unconsolidated sand, silt, clay, and shell and consolidated beds of shell, limestone, and dolomite of Pliocene and Miocene age.

5.2.3.3 Floridan Aquifer System

The Floridan aquifer system contains the thickest and most extensive aquifer layers in Florida. Estimation of changes in regional-scale groundwater flow patterns due to widespread pumping increases in the Floridan aquifer system is the focus of this study. The Floridan aquifer system is composed of permeable Paleocene-age and Eocene-age carbonate rocks. The geologic formations that compose the Floridan aquifer system are, from bottom to top: the Cedar Keys Formation, the Oldsmar Formation, the Avon Park Formation, and the Ocala Limestone (Table 5.1). These formations consist of interbedded limestone, dolomite, and dolomitic limestone in which the amount of primary porosity, secondary porosity, and secondary infilling of pores or fractures is highly variable with depth. Throughout the ECF region, the Floridan aquifer system has been subdivided into three hydrostratigraphic subunits on the basis of relative hydraulic conductivity (Tibbals 1990): the Upper Floridan aquifer, the middle semiconfining unit, and the Lower Floridan aquifer.

Total thickness of the Upper Floridan aquifer ranges from less than 60 m to more than 198 m in the study area, generally increasing from the northwest to the southeast. Reported transmissivities of the Upper Floridan aquifer are between 111 m^2/day and 49,238 m^2/day. It consists of the Ocala Limestone and approximately the upper one-third of the Avon Park Formation (Table 5.1).

Total thickness of the revised middle semiconfining unit ranges from approximately 45 m to 198 m and also generally increases in a southward direction. The leakances of the middle semiconfining unit range from less than 0.00005/day to more than 0.001/day. This layer consists of relatively soft, micritic limestone and dense, dolomitic limestone with little secondary porosity compared to the aquifer units above and below.

Table 5.1 Geologic and Hydrostratigraphic Units within the Model Area

SYSTEM	SERIES	STRATIGRAPHIC UNIT		HYDROGEOLOGIC UNIT	
QUATERNARY	HOLOCENE PLEISTOCENE	UNDIFFERENTIATED SAND AND CLAY DEPOSITS		SURFICIAL AQUIFER SYSTEM	
TERTIARY	PLIOCENE	HAWTHORN GROUP	PEACE RIVER FORMATION	INTERMEDIATE AQUIFER SYSTEM OR INTERMEDIATE CONFINING UNIT	
	MIOCENE		ARCADIA FORMATION	FLORIDAN AQUIFER SYSTEM	UPPER FLORIDAN AQUIFER
	OLIGOCENE	SUWANNEE LIMESTONE			
	EOCENE	OCALA LIMESTONE			
		AVON PARK FORMATION			MIDDLE CONFINING UNIT
	PALEOCENE	OLDSMAR AND CEDAR KEYS FORMATIONS			LOWER FLORIDAN AQUIFER

Total thickness of the Lower Floridan aquifer ranges from approximately 305 m to greater than 610 m and gradually increases in a southward direction. Reported transmissivities of the Lower Floridan aquifer are between 18,580 m^2/day and 62,245 m^2/day. Estimated rates of natural recharge range from less than 10 cm/yr to greater than 30 cm/yr through the Floridan aquifer system. Natural discharge occurs as diffuse upward leakage to the surficial aquifer system and as spring flow, approximately 42% of which comes from the springs of Wekiva River Basin. The geologic units comprising the Lower Floridan aquifer are the lower part of the Avon Park Formation, the Eocene Oldsmar Formation, and the upper part of the Paleocene Cedar Keys Formation.

5.2.4 Hydraulic Characteristics

The data available concerning the hydraulic characteristics of the Floridan aquifer system are derived from aquifer tests to include information on the Upper and Lower Floridan aquifer transmissivities and specific-capacities as well as normalized well yield data. Reported transmissivity of the Upper Floridan aquifer ranges from approximately 111 m^2/day to 49,238 m^2/day from 84 tests (Table 5.2). Lower Floridan aquifer transmissivity estimates ranged from 18,580 m^2/day to 62,245 m^2/day based on 10 aquifer performance tests. The relatively few Lower Floridan tests that have been conducted to date were located within or near the Orlando area. Field estimates of vertical hydraulic conductivity of the middle semiconfining unit have been made at two sites. At the Bull Creek Wildlife Management Area in eastern Osceola County, estimates ranged from 0.15 cm/day to 61 cm/day.

Table 5.2: Ranges of Aquifer Parameter Values Reported from Aquifer Performance Tests Conducted in the East-Central Florida Region

Hydrostratigraphic unit	Parameter	Minimum Reported Value	Maximum Reported Value	Approximate Number of Tests	Sources*
Surficial aquifer system	Horizontal hydraulic conductivity	0.9 cm/day	61 m/day	50	1,2,4,6
Surficial aquifer system	Transmissivity	8.4 m^2/day	1858 m^2/day	30	2,5,6
Intermediate confining unit	Leakance	1 * 10^{-6}/day	0.8/day	38	5
Upper Floridan aquifer	Transmissivity	113 m^2/day	49,238 m^2/day	84	3,5
Lower Floridan aquifer	Transmissivity	18,630 m^2/day	63,960 m^2/day	10	5,7

*1 = McGurk et al. (1989); 2 = Phelps (1990); 3 = Shaw and Trost (1984); 4 = Spechler and Halford (2001); 5 = Szell (1993); 6 = Williams (1995); 7 = St. Johns River Water Management District consumptive use permitting files (Source: Technical Publication – SJ2002-3; SJRWMD)

5.2.5 Potentiometric Levels

Figure 5.3 provides the average 1995 potentiometric surface of the Upper Floridan aquifer. The elevations of the estimated contours are from less than 3 m NGVD to approximately 40 m NGVD and this is consistent with the terrain features. Different from the ECF model, the western, southwestern and northern boundaries were assumed to provide a zero-flux condition based on the measured potentiometric contours.

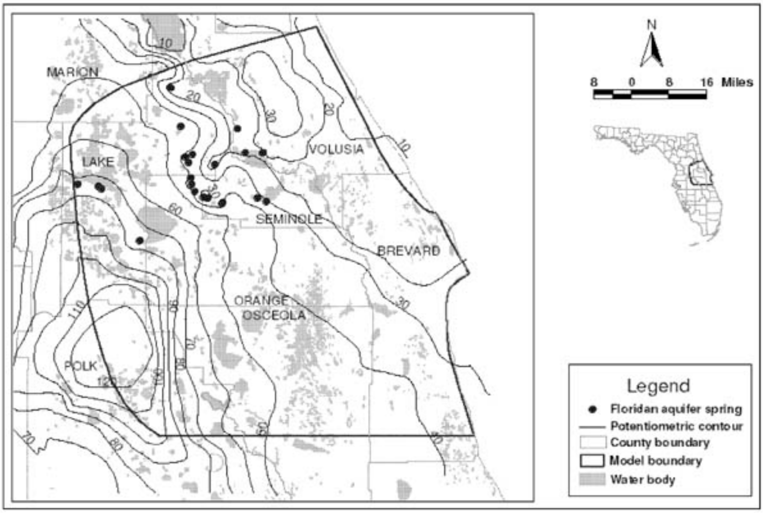

Figure 5.3: Estimated average 1995 potentiometric surface of the Upper Floridan Aquifer (adapted from Knowles et al. 1995 and O'Reilly et al. 1996, unit: ft)

5.3 COMPUTATIONAL MODELS – WASH123D

A physics-based, distributed watershed model WASH123D (A Numerical Model Simulating Water Flow and Contaminant and Sediment Transport in WAterSHed Systems of 1-D Stream/River Network, 2-D Overland Regime, and 3-D Subsurface Media, Yeh et al., 1998, 2005, 2006) was chosen for application to conducting the Wekiva springshed study because of its design capability and flexibility for further studies in the future. This computational model is described in this section.

5.3.1 Multimedia and Multiprocesses

WASH123D was developed to cover dentric river/stream/canal networks, overland regime (land surface), and subsurface media including vadose and saturated (groundwater) zones. It incorporates natural junctions and control structures such as weirs, gates, culverts, levees, and pumps in river/stream/canal networks. It also includes management structures such as storage ponds, pumping stations, culverts, and levees in the overland regime. In the subsurface media, management devices such as pumping/injecting wells, drainage pipes, and drainage channels are also included. Numerous management rules of these control structures and pumping operations have been implemented.

WASH123D was designed to deal with physics-based multi-processes occurring in watersheds. These include density dependent flow and thermal and salinity transport over the entire hydrologic cycle. The processes include (1) evaporation from surface waters (rivers, lakes, reservoirs, ponds, etc) in the terrestrial environment; (2) evportransipiration from plants, grass, and forest from the land surface; (3) infiltration into vadose zone through the land surface and recharges (percolations) to groundwater through water tables; (4) overland flow and thermal and salinity transport in surface runoff; (5) hydraulics and hydrodynamics and thermal and salinity transport in dentric river networks; and (6) subsurface flow and thermal and salinity transport in both vadose and saturated zones.

To enable the modeling of any number of water quality measures including sediments, a general paradigm of reaction-based approaches was taken in WASH123D. As a result of this generic approach, WASH123D can easily be employed to model bigogeochemical cycles (including nitrogen, oxygen, phosphorous, and carbon cycles, etc.) and biota kinetics (including algae, phyotoplankton, zooplakton, caliform, bacteria, plants, etc.). In fact, once one is able to transform the biogeochemical processes into reaction networks and come up with rate equations for every reaction, then WASH123D can be employed to model his/her system of reactive transport in surface runoff, surface water, and subsurface flows on watershed scales.

5.3.2 Mathematical Formulations

The theoretical bases of fluid flows and transport processes built in WASH123D are based on the conservation laws of fluid, momentum, energy, and mass with associated constitution relationships between fluxes and state variables and appropriately formulated equations for source/sink terms. Various types of boundary conditions based on physics reasoning are essential to supplement the governing equations. Adequate initial conditions are either obtained from measurements or with simulations of steady-state versions of the governing equations.

Governing Equations

For fluid flows in river/stream/canal networks, one-dimensional St Venant Equations modified to include the effects of density due to temperature and salinity are employed, which are in fact the cross-section area averaged Navier-Stokes equations. For surface runoff over the land surfaces, two-dimensional St Venant Equations modified to take into account the effects of temperature- and salinity-dependent density. The two-dimensional St Venant Equations are in fact the vertically averaged Navier-Stokes equations. The salinity and thermal transport equations were derived based on the conservation principle of mass and energy (Yeh et al., 2006). The particular features in WASH123D are the inclusion of three approaches to model surface flow in a watershed system: the kinematic, diffusive, and dynamic wave models. The dynamic wave models completely describe water flow but they are very difficult to solve under some conditions (e.g., when the slope of ground surface is steep), regardless of what numerical approach is employed. On the other hand, the diffusion and/or kinematic models can handle a wide range of flow problems but are inaccurate when the inertial terms play significant roles. Thus, three options are provided in the model: the kinematic wave model, the diffusion wave model, and the dynamic wave model to accurately compute water flow over a wide range of conditions. The complete set of governing equations for river/stream networks and surface runoff is not presented here because only the subsurface flow module is employed for this study, and it can be found elsewhere (Yeh et al., 2006).

The subsurface flow is described with the modified Richards equation. The modification incorporates the effect of density due to temperature and salinity effects. The governing equation is derived based on continuity of fluid, continuity of solid mass, incompressibility of solids, and Darcy's law (Yeh et al., 1994; Lin et al., 1997). It can be written as (Yeh et al., 2006)

$$\frac{\rho}{\rho_o} F \frac{\partial h}{\partial t} = -\nabla \cdot \left(\frac{\rho}{\rho_o} \mathbf{V} \right) + \frac{\rho^*}{\rho_o} q \tag{1}$$

where ρ is the density of water; ρ_o is the reference density of water; F is the generalized storage coefficient [L^{-1}]; h is the pressure head [L]; t is the time [T]; $\mathbf{V}$ is the Darcy's velocity or specific discharge [L/T]; ρ^* is the density of source water; and q is the source and/or sink [$L^3/L^3/T$]. The generalized storage coefficient, F, and the Darcy's velocity, $\mathbf{V}$, are given as

$$F = a'\frac{\theta_e}{n_e} + \beta'\theta_e + n_e\frac{dS}{dh} \text{ and } \mathbf{V} = -\mathbf{K} \cdot \left(\frac{\rho_o}{\rho} \nabla h + \nabla z \right) \tag{2}$$

where a' is the modified compressibility of the medium [1/L], θ_e is the effective moisture content [L^3/L^3], n_e is the effectively porosity [L^3/L^3], β' is the compressibility of water [1/L], S is the degree of saturation, $\mathbf{K}$ is the hydraulic conductivity [L/T], and z is the potential head [L].

The principles of mass balance were employed to derive the modified advective-dispersive/diffusion transport equations governing the temporal-spatial distribution of water quality, suspended sediment, and bed sediment. For sediment transport, phenomenological equations for erosions and depositions were used. For biogeochemical transport, reaction rate equations can be provided based on mechanisms (pathways) or based on empirical formulations using experimental data for every slow reaction. Examples of mechanisms-based reaction rates includes forward-backward rate equations based on the collision theory, Monod-type rate equations based on the enzymatic kinetic theory (Segel, 1975), etc. Empirical rate equations include zero-order, first order, n-th order, Freundlich kinetics, etc. For every fast reaction, either the mass action equation based on the thermodynamic approach or a user's defined algebraic equation can be used.

Boundary Conditions

To enable the simulation of as wide a range of problems as possible, many types of boundary conditions that can be anticipated in real-world problems were provided in WASH123D. These include global boundaries, internal boundaries and internal sources/sinks, and media interfaces. On internal boundaries such as natural junctions and control structures of weirs, gates, culverts, levees, mass or energy balance is explicitly enforced by solving a set of flux continuity and state variable continuity (or flux) equations. For the internal sources/sinks, pumping and operation rules are simulated to ensure mass conservation. On the media interfaces, continuity of fluxes and continuity of state variables or formulations of fluxes when state variables are discontinuous are imposed.

On global boundaries, three types of boundary conditions can be prescribed for surface water: (1) specified water depth, (2) specified flow rates, and (3) rating curves relating discharges to water depth. For subsurface flows, five types of boundary conditions can be prescribed: (1) specified pressure head, (2) specified flux, (3) specified pressure gradient, (4) variable conditions in which the model will iteratively determine head or flux conditions (this type of boundary conditions is normally specified at the atmospheric boundary), and (5) radiation conditions where the flux is proportional to the difference in head between the media and surface waters such as rivers or lakes/reservoirs/ponds. The description of boundary conditions for surface water flow and transport can be found elsewhere (Yeh et al., 2006). The boundary conditions for subsurface flows are described below.

<u>Dirichlet Boundary Condition</u>

This boundary condition is used when pressure head can be prescribed on the boundary. It can be expressed as

$$h = h_d(\mathbf{x},t) \qquad on \quad B_d(\mathbf{x}) = 0 \tag{3}$$

where $h_d(\mathbf{x},t)$ is the Dirichlet head on the boundary surface $B_d(\mathbf{x}) = 0$. This type of boundary condition is normally applied to hydraulically-connected surface water bodies where the water depth is known.

<u>Neumann Boundary Condition</u>

This boundary condition is employed when the flux results from pressure-head gradient is known as a function of time. It is written as

$$-\mathbf{n} \cdot \mathbf{K} \cdot \frac{\rho_o}{\rho} \nabla h = q_n(\mathbf{x},t) \qquad on \quad B_n(\mathbf{x}) = 0 \tag{4}$$

where $\mathbf{n}$ is the outward unit vector normal to the surface, $q_n(\mathbf{x},t)$ is the Neumann flux and $B_n(\mathbf{x})=0$ is the Neumann boundary surface. This type of boundary condition is normally applied to locations where the pressure gradient is known. For example, it can be applied to natural drainage boundaries where the pressure gradient may be zero and the drainage is induced mainly by gravity.

Cauchy Boundary Condition

This boundary condition is employed when the flux resulting from a hydraulic-head gradient is known as a function of time. It can be written as

$$-\mathbf{n}\cdot\left(\mathbf{K}\cdot\frac{\rho_o}{\rho}\nabla h+\mathbf{K}\cdot\nabla z\right)=q_c(\mathbf{x},t) \qquad on \;\; B_c(\mathbf{x})=0 \tag{5}$$

where $q_c(\mathbf{x},t)$ is the Cauchy flux and $B_c(\mathbf{x})=0$ is the Cauchy boundary surface. This type of boundary condition is normally applied to locations where the infiltration rate is known, e.g., at lakes, reservoirs, and/or storage ponds that are not hydraulically connected.

River or Radiation Boundary Condition

This boundary condition is employed when there is a thin layer of medium separating the river and the subsurface media. Mathematically, it is as follows

$$-\mathbf{n}\cdot\mathbf{K}\cdot\left(\frac{\rho_o}{\rho}\nabla h+\nabla z\right)=-\frac{K_R}{b_R}(h_R-h) \qquad on \;\; B_r(\mathbf{x})=0 \tag{6}$$

where K_R is the hydraulic conductivity of the thin layer [L/T], b_R is the thickness of the thin layer [L], h_R is the water depth in the river, and $B_r(\mathbf{x})=0$ is the surface between the river and subsurface media.

Variable Boundary Conditions

This boundary condition is usually employed for the air-media interface when the overland flow is not present. If the overland flow is present, the coupling boundary condition between subsurface and surface flows should be applied (Yeh et al., 2006). In the absence of overland flow, the variable boundary conditions are stated as follows. During precipitation periods, the variable boundary condition is mathematically stated as

$$-\mathbf{n}\cdot\mathbf{K}\cdot\left(\frac{\rho_o}{\rho}\nabla h+\nabla z\right)=q_p(\mathbf{x},t) \;\; iff \;\; h\le h_p(\mathbf{x},t) \;\; on \;\; B_v(\mathbf{x})=0 \tag{7}$$

or

$$h = h_p(\mathbf{x},t) \quad iff \quad -\mathbf{n}\cdot\mathbf{K}\cdot\left(\frac{\rho_o}{\rho}\nabla h + \nabla z\right) \geq q_p(\mathbf{x},t) \quad on \quad B_v(\mathbf{x}) = 0 \tag{8}$$

During no-precipitation periods, it is expressed as

$$-\mathbf{n}\cdot\mathbf{K}\cdot\left(\frac{\rho_o}{\rho}\nabla h + \nabla z\right) = q_e(\mathbf{x},t) \quad iff \quad h \geq h_m(\mathbf{x},t) \quad on \quad B_v(\mathbf{x}) = 0 \tag{9}$$

or

$$h = h_m(\mathbf{x},t) \quad iff \quad -\mathbf{n}\cdot\mathbf{K}\cdot\left(\frac{\rho_o}{\rho}\nabla h + \nabla z\right) \leq q_e(\mathbf{x},t) \quad on \quad B_v(\mathbf{x}) = 0 \tag{10}$$

or

$$h = h_p(\mathbf{x},t) \quad iff \quad -\mathbf{n}\cdot\mathbf{K}\cdot\left(\frac{\rho_o}{\rho}\nabla h + \nabla z\right) \geq 0 \quad on \quad B_v(\mathbf{x}) = 0 \tag{11}$$

where $h_p(\mathbf{x},t)$ (positive values) is the ponding depth, $q_p(\mathbf{x},t)$ (negative values) is the flux due to precipitation, $h_m(\mathbf{x},t)$ (negative values) is the minimum pressure head, and $q_e(\mathbf{x},t)$ (positive values) is the potential evaporation rate on the surfaces of the variable boundary condition $B_v(\mathbf{x}) = 0$. Equations (7) and (8) implicitly state that, when the potential infiltration is greater than precipitation, the infiltration is limited by precipitation. When the simulated infiltration under ponding conditions is less than precipitation, it is accepted as the correct solution. Equations (9) and (10) implicitly state that, when the evaporative capacity of the medium is greater than potential evaporation, the evaporation is limited by potential evaporation. When the simulated evaporation under minimum boundary pressure is less than potential evaporation, it is accepted as the correct solution. In other words, the actual infiltration into the media must be less than or equal to the precipitation and the actual evaporation from the medium must be less or equal to potential evaporation imposed by the atmospheric conditions. Equation (11) states that seepage conditions prevail.

Only one of Equations (7) through (11) is used at any point on the variable boundary at any time. The variable boundary conditions are used to determine the recharge (infiltration) or discharge (seepage) areas by iteratively solving the boundary conditions. This is unlike most of the existing numerical models in which a flux boundary condition (Cauchy boundary condition) has to be specified at the air-media interface, equivalent to saying that the recharge or discharge areas are considered *a priori*.

For scalar transport, four types of boundary conditions can be prescribed: (1) specified state variables (concentrations or temperature), (2) specified fluxes of state variables, (3) specified gradient fluxes of state variables, and (4) variable conditions in which fluxes are specified when the flow is coming into the region or the mass/energy is transported out of the region by advection when the flow is going out of the region. In addition, at the atmosphere-media interface, heat and mass budget balance must be satisfied for thermal transport. Mathematical statements of these boundary conditions can be found elsewhere (Yeh et al., 2006).

Numerical Solutions

To provide robust and efficient numerical solutions of the governing equations, many options and strategies are provided in WASH123D so a wide range of application-dependent circumstances can be simulated. For surface flow problems, the semi-Lagrangian method (backward particle tracking) was used to solve kinematic wave equations. The diffusion wave models were numerically approximated with the Galerkin finite element method or the semi-Lagrangian method. The dynamic wave model was first mathematically transformed into characteristic wave equations. Then it was numerically solved with the Lagrangian-Eulerian method. The subsurface flow-governing equations were discretized with the Galerkin finite element method. The dynamic wave model for surface water flows in conservative forms will be discretized with finite element methods in a future update of WASH123D.

For scalar transport equations including thermal, salinity, sediment, and reactive chemical transport, either finite element methods or hybrid Lagrangian-Eulerian methods were used to approximate the governing equations. Three strategies were employed to handle the coupling between transport and biogeochemical reactions: (1) fully implicit scheme, (2) mixed predictor-corrector and operator-splitting methods, and (3) operator-splitting schemes. For the fully implicit scheme, one iteratively solves the transport equations and reaction equations. For the mixed predictor-corrector and operator-splitting method, the advection-dispersion transport equation is solved with the source/sink term evaluated at the previous time in the predictor step. The implicit finite difference was used to solve the system of ordinary equations governing the chemical kinetic and equilibrium reactions in the corrector step. The nonlinearity in flow and sediment transport equations is handled with the Picard method, while the nonlinear chemical system is solved using the Newton-Raphson method.

Several matrix solvers were provided to efficiently solve the system of linear algebraic equations resulting from the discretization of the governing equations and the incorporation of boundary conditions. These include direct band matrix solvers; basic point iteration solvers such as Gauss-Seidel iteration or successive over relaxation; basic line iteration solvers; preconditioned conjugate gradient methods with point iterations, incomplete Cholesky decomposition, and line iterations as preconditioners; and multigrid methods.

5.3.3 Design Capability of WASH123D

WASH123D includes seven modules: (1) one-dimensional river/stream network module, (2) two-dimensional overland module, (3) three-dimensional subsurface module, (4) coupled 1D and 2D module, (5) coupled 2D and 3D module, (6) coupled 3D and 1D module, and (7) coupled 1D, 2D, and 3D module. Each module can be used to simulate flows alone, sediment transport alone, water quality transport alone, or flow and sediment and water quality transport simultaneously. When both flow and transport are simulated, the flow fields are computed first. Then the transport is calculated using the computed flow fields at respective times. Temperature- and salinity-dependent flow is considered. A slightly different version of WASH123D also included 0-dimensional water, energy, and mass budget to simulate the changes of stages, temperature, and concentrations of sediment and any biogeochemical species for well mixed surface water bodies such as small lakes, reservoirs, storage ponds, etc. This 0D module has been coupled to one-dimensional canal networks and it can be coupled with two-dimensional overland regime or three-dimensional subsurface media.

WASH123D has the following main features that make it flexible and versatile in modeling a wide range of real-world problems: (1) "True" rather than "quasi" three-dimensional subsurface problems can be simulated; (2) Irregular elements facilitate the representation of complex geometry; (3) Both heterogeneous and anisotropic media, as many as desired, can be taken into account; (4) On the ground surface, infiltration rates are determined by the WASH123D model rather than imposed as an input parameter by users of MODFLOW; (5) Vadose zone can be incorporated to more realistically simulate the infiltration; (6) Density dependent flow is available to more realistically model coastal aquifers; and (7) Many options are available to both compose and solve matrix equations.

The FORTRAN code WASH123D iteratively solves the three-dimensional groundwater flow equations. Input to the program includes the geometry of the system, the properties of the media, and the initial and boundary conditions. Output includes the spatial distribution of pressure head, total head, velocity fields, and moisture contents, all as functions of time.

5.4 MODEL SETUP

5.4.1 Domain Discretization

The use of WASH123D requires the modeling domain be divided into discrete elements. The numerical equations of groundwater flow are solved iteratively for each node to produce simulated water levels, or head values and Darcy's velocity field. As shown in Equation (1), the groundwater flow between elements depends on the head gradient as well as the conductivities assigned to each element. The model domain was discretized as shown in Figure 5.4.

The domain profile was divided into six layers along the vertical direction (Figure 5.4). The discretization coincides with the ECF model except that the intermediate confining unit and the middle semiconfining unit were incorporated in the simulation. The six layers are stated as following: (1) ECF Layer 1, known as the surficial layer (indicated as yellow in Figure 5.4); (2) The intermediate confining unit (indicated as upper red layer in Figure 5.4); (3) ECF Layer 2, known as the upper zone of the Upper Floridan aquifer (indicated as blue in Figure 5.4); (4) ECF Layer 3, known as the lower zone of the Upper Floridan aquifer (indicated as gray in Figure 5.4); (5) The middle semiconfining unit (indicated as lower red layer in Figure 5.4); and (6) ECF Layer 4, known as the Lower Floridan aquifer (indicated as green in Figure 5.4).

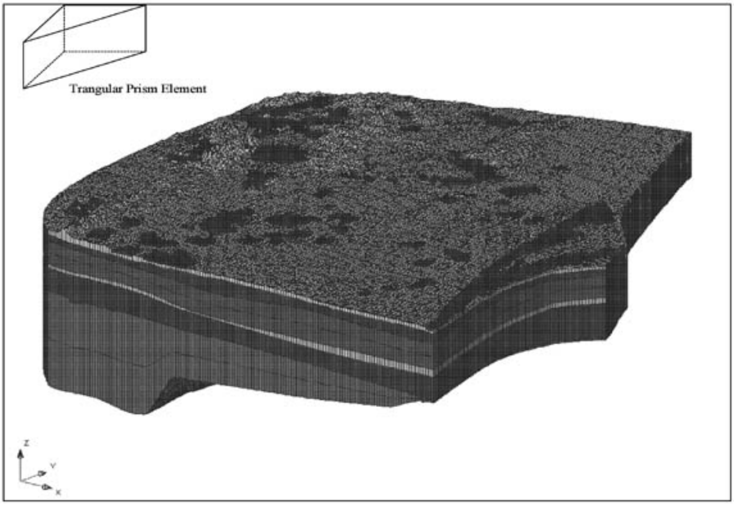

Figure 5.4: 3-D Finite Element Mesh of the Modeling Domain

Numerically, the modeling domain was discretized with a total of 437,576 triangular prism elements (see upper left of Figure 5.4) connected at 249,057 nodes. The interior elements have the equal size 290,322 m^2 while the boundary elements have the approximate size of one third square kilometer due to the irregularity. Furthermore, because of the large thickness of ECF Layer 2 and Layer 4, each was divided into two sub-layers of elements with the same media parameters. Therefore, eight numerical layers are included in the simulation. Several types of input hydrologic data are required for the model. These include information needed to assign boundary conditions, applied stresses, and properties of each numerical layer.

5.4.2 Boundary Conditions

Boundary conditions were estimated and applied at the sides of the model domain for the Floridan aquifer system layers and confining units, at springs, at water bodies such as lakes, and at the air-media interface. Choices for boundary condition assignments can be classified into three types: (1) prescribed potentiometric levels (heads), (2) prescribed flow rates, and (3) head-dependent flux.

The base of the model is a prescribed a zero-flux boundary condition. Since clearly defined hydrogeologic boundaries do not exist within the Floridan aquifer system in the modeling domain, realistic conditions should be set up and applied along the lateral sides of the domain to represent flow that occurs across these artificial boundaries. A potentiometric surface map of the Upper Floridan aquifer (Figure 5.3) was used to locate the model boundaries and to help in defining these conditions. On a regional scale, flow directions within the Upper Floridan aquifer will be perpendicular to the potentiometric contours shown in Figure 5.3. Therefore, the northern, southwestern, and western sides of the domain are prescribed zero-flux boundary conditions. While the head values are defined along the southern and the seaward boundary, these head values are mainly from the input for general-head boundary (GHB) package of the ECF model. At springs and lakes, constant elevations were assumed and the boundary conditions for them were assigned as prescribed levels (heads), where these values are also defined in the ECF model input.

Because several stresses were applied to the model, including well withdrawals from different depths within the Floridan aquifer system, recharge to the Upper Floridan aquifer through drainage wells and recharge to the surficial aquifer system caused by rainfall and ET, the air-media interface is usually a boundary on which the subsurface flow direction is not predetermined and needs to be set up so that consistent computational results can be obtained. WASH123D is such designed as: when a boundary is flux-type for the rainfall period, a complete adsorption of throughfall water is assumed, while a potential ET is simulated if it is for the evaporation period. The ponding-type boundary is to simulate the accumulation of water above ground surface while the minimum pressure-type boundary is to describe the allowed minimum pressure associated with the soil being considered. The ECF model input dataset for the ET package provides such parameters, such as ponding depth and minimum pressure.

5.4.3 Applied Stresses

The most important input stress to the model is the recharge applied to the surficial aquifer system, including precipitation, flow to rapid infiltration basins, septic tank effluent, the ET from the unsaturated zone, applied irrigation as well as the overland runoff. The recharge rates were estimated by developing an algorithm that incorporates the appropriate portions of the steady state water budget for the surficial layer in the ECF model and these values are used as air-media boundary condition input as discussed above and can be found in the ECF model input for the recharge (RCH) package.

A total of 5,097 wells were applied to different depths within the modeling domain. These wells are classified as four types: (1) withdraw wells; (2) drainage wells; (3) self-supplied domestic wells; and (4) free-flowing wells. The withdraw wells introduces the majority of the water consumed. The ECF model provides much of the information used to prescribe well rates in the well (WEL) package input. During the simulation with WASH123D, these wells are treated as point sources or sinks as indicated by the q term in Equation (1). Withdraw wells, self-supplied domestic wells, and free-flowing wells have negative rates and each is treated as a point of sink while each of the drainage wells is treated as a point of source in WASH123D.

5.4.4 Aquifer and Confining Unit Characteristics

Input data representing the model geometry or hydrostratigraphy, such as aquifer layer and confining unit top and bottom elevations, were obtained from the calibration data of the ECF model. Horizontal isotropy was assumed for all eight numerical layers (i.e., horizontal hydraulic conductivity was assumed to be equal along the x- and y- directions). The calibrated vertical conductivities and leakances of the intermediate confining and semiconfining units of ECF model were employed to estimate the hydraulic characteristics of the model layers represented by the material types input of WASH123D. Due to the scarcity of large-scale hydraulic conductivities estimates for the surficial layer, a homogenous horizontal hydraulic conductivity equal to 6 m/day is assumed throughout this system. While all other seven numerical layers have unique material types defined at each element. Moreover, the media within the vicinity of the springs usually have large conductivities to drive the groundwater upward; a particular material type was given for each element of the 23 springs in the modeling domain. In total, 273,509 material types were defined in the simulation.

5.5 CALIBRATION - SIMULATION RESULTS

With minimum calibration, the model matches observations very well as described below.

5.5.1 Potentiometric Levels

The Wekiva WASH123D model was run to evaluate the average, steady state 1995 hydrological conditions. As shown in Figures 5.5 through 5.7, the distribution of simulated Floridan aquifer system groundwater levels using WASH123D shows very good agreement with the field observations at corresponding locations. One can also realize that the simulated 1995 water levels mimic the topography on a regional scale.

The simulated 1995 layer 2 potentiometric surface compares favorably with the average 1995 Upper Floridan aquifer potentiometric surface (Figure 5.5). The simulated layer 3 potentiometric surface is similar to the layer 2 surface, differing only along the St. Johns River valley and near where layer 3 is inactive due to the location of the saltwater interface (Figure 5.6). The simulated 1995 layer 4 (Lower Floridan aquifer) potentiometric surface is a subdued reflection of the Upper Floridan aquifer potentiometric surface. Layer 4 water levels are lower than layer 2 and layer 3 (Upper Floridan aquifer) water levels in the southwestern corner of the model and in central Volusia County. The simulated layer 4 water levels match the observed well data fairly well as shown in Figure 5.7. The simulated potentiometric contours also verify the zero-flux boundaries which were set for the western, southwestern, and northern sides of the modeling domain.

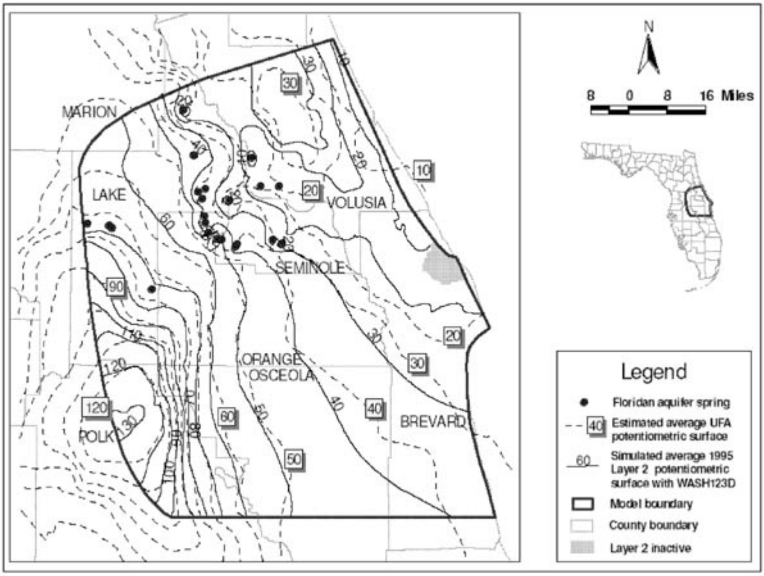

Figure 5.5: Average 1995 Upper Floridan Aquifer (UFA) potentiometric surface and simulated Layer 2 1995 potentiometric surface with WASH123D (unit: ft)

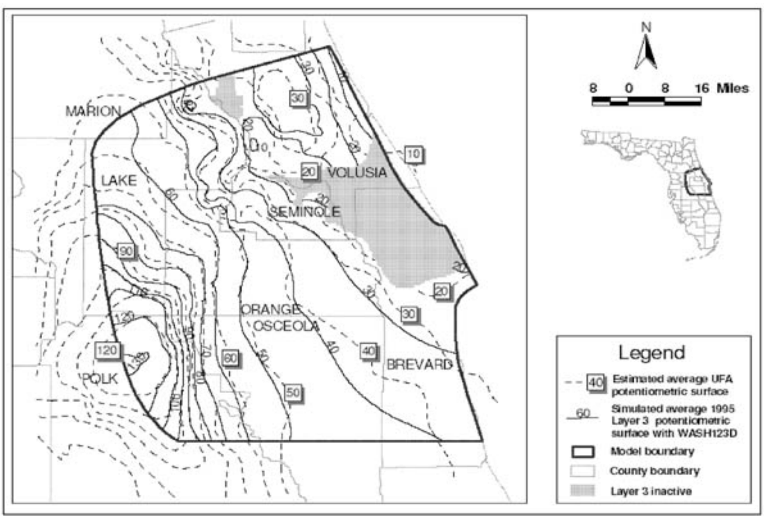

Figure 5.6: Average 1995 Upper Floridan Aquifer (UFA) potentiometric surface and simulated Layer 3 1995 potentiometric surface with WASH123D (unit: ft)

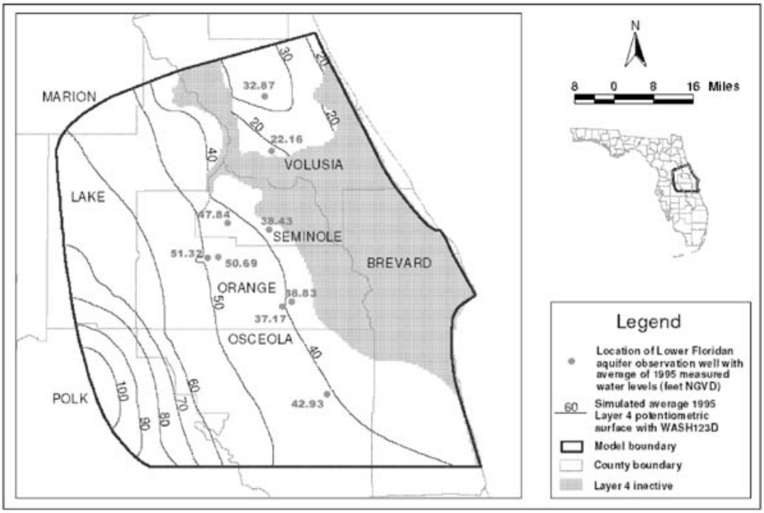

Figure 5.7: Simulated Layer 4 potentiometric surface with WASH123D and observed Lower FloridanAquifer water levels, average 1995 conditions (unit: ft)

5.5.2 Groundwater Flow

One can observe the groundwater flow patterns based on the simulated velocity fields as well as the potentiometric surfaces. Three cross-sections along Rock Spring are selected as shown in Figure 5.8.

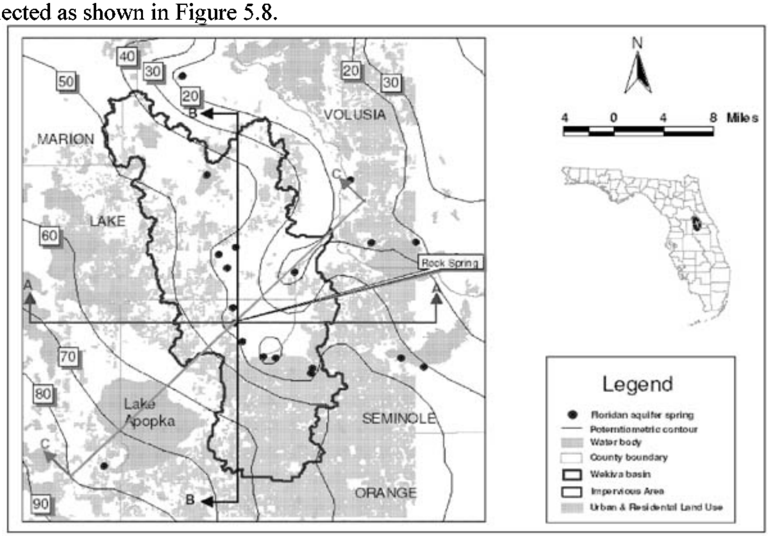

Figure 5.8: Three cross-sections along Rock Spring

It is seen from Figure 5.9 that the potentiometric head difference drives the groundwater moving within the subsurface aquifer system. In a regional scale, the velocities are perpendicular to the head contours. Due to the large conductivity within the vicinity of the spring, the Darcy's velocities are large upwards resulting in the spring discharge. One can also see that the majority of groundwater recharge to spring flow comes from the relatively shallow aquifer within the nearness of the spring. Where the velocities are relatively high, less time is needed for the groundwater moving to the spring. Other contributions to recharge are from the deeper aquifer as well as seepage from the surface.

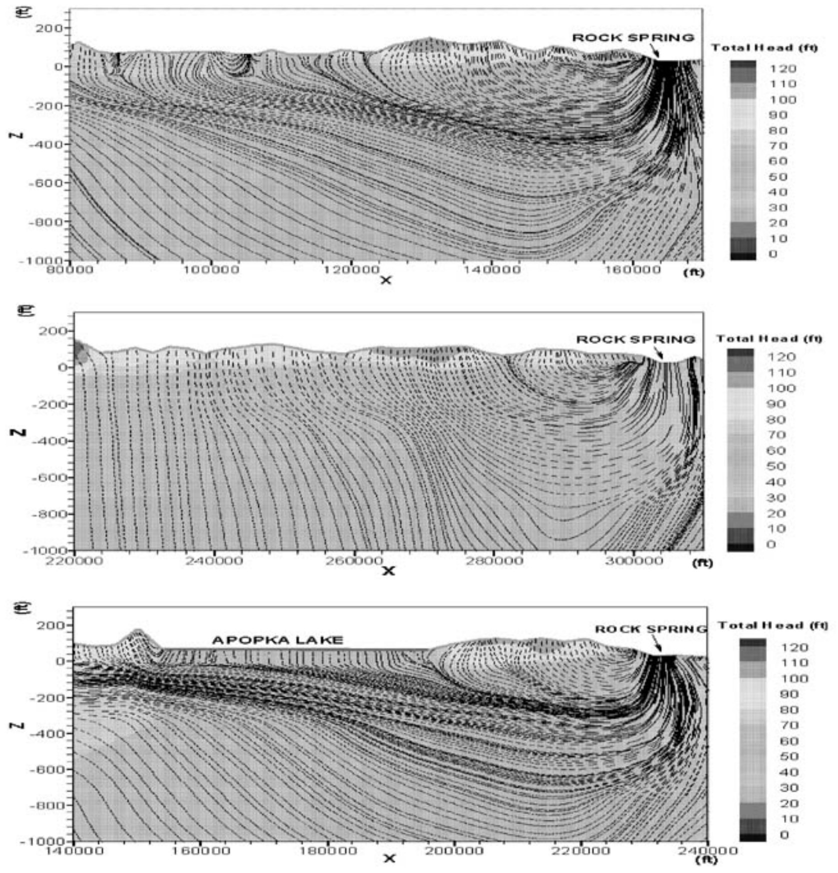

Figure 5.9: Groundwater flow along three cross-sections of Rock Spring: upper (A-A); middle (B-B); lower (C-C)

5.5.3 Areas of Recharge and Discharge

Areas for recharge and discharge from the Floridan aquifer system are identified. This simulation result is consistent with that simulated with the ECF model very well. It also shows good agreement with the reported areas for recharge and discharge by Wekiva Basin Task Force and by Boniol et al. (1993). Natural discharge from the Floridan aquifer system occurs as diffuse upward leakage to the surficial aquifer system and as spring flow (McGurk and Presley, 2002). Simulated rates of natural recharge ranges from less than 10 cm/yr to greater than 30 cm /yr. Water leaks upward to the surficial aquifer system through the intermediate confining unit wherever the Floridan aquifer potentiometric level is higher than that of the surficial aquifer system, as delineated as discharge areas in Figure 5.10. Areas where the surficial aquifer potentiometric level is higher than that of the Floridan aquifer system are defined as the recharge areas in Figure 5.10. High-rate recharge areas coincide with high lands characterized by sandy ridges with deep water table soils and karst topography and where there are few perennial streams to collect overland runoff (McGurk and Presley, 2002), within the areas where the head gradient between the surficial aquifer and Upper Floridan aquifer is large and where the intermediate confining layer is thin or more permeable. Conversely, low-rate recharge zone appears in the low or flat areas where the water table is near the land surface thus enhancing the ET from the saturated zone, where the head gradient is small, and where the intermediate confining layer is thick or having low permeability.

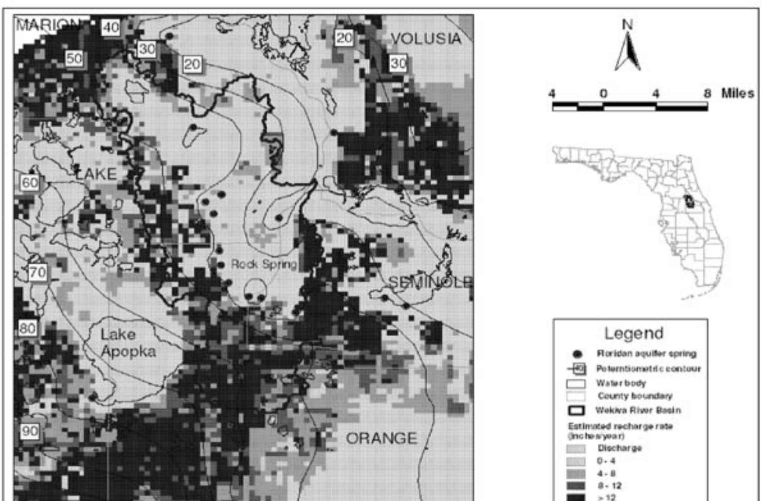

Figure 5.10: Areas of recharge to and discharge from the Floridan aquifer system

One can see that high-rate recharge areas are concentrated within the Wekiva River Basin, and over half of the Lake Apopka provides a source of recharge to the Wekiva springs. There are also high and moderate areas that extend farther south and west and also to the east within Seminole County. The identification of the recharge area is particularly important in preserving the ecosystem of the Wekiva River Basin.

5.6 APPLICATIONS

Once the model is calibrated, it can be used to investigate the effects of anthropogenic changes on groundwater, the contributing areas to spring, etc.

5.6.1 Urbanization Effects

Since the early 1980s, the Central Florida region has continued to experience tremendous growth that has resulted in increasing demands on the region's transportation system and rising development pressures on the land surrounding the Wekiva River Protection Area (WBATF, 2003). The intensive urbanization has been introducing increases of the impervious surface (such as streets and parking areas), thus increasing runoff while decreasing recharge to the Floridan aquifer system. It is possible that the volume of groundwater moving toward discharge from the Wekiva River spring systems has diminished over time due to the loss of recharge as a consequence of land development.

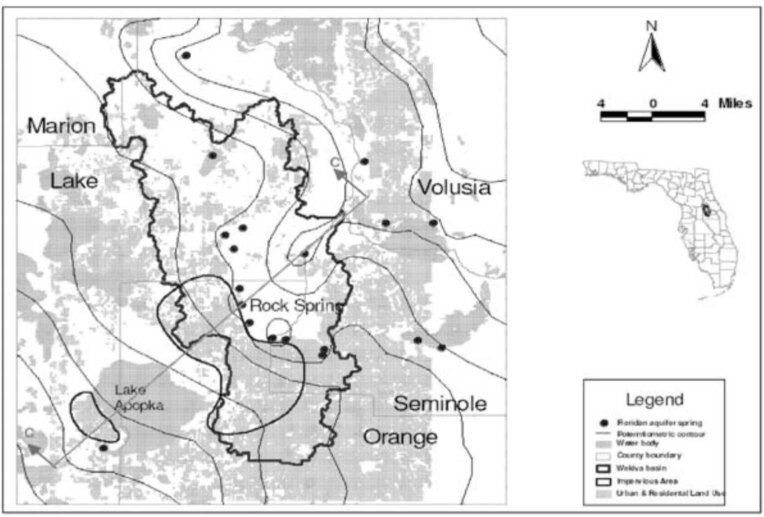

Figure 5.11: Cross-section C-C along Rock Spring

During the study of the Wekiva springshed, calculated were the discharges of the springs based on the simulated velocity fields. Rock Spring is an example, where the simulated discharge is approximately 1.9 cms (cubic meter per second), which is compared to the measured 1.8 cms, under the steady state condition of the year 1995. Using this simulation below ground set of conditions, the impervious area was increased by about 155 square kilometers, as indicted in Figure 5.10 within the dark-red lines. This impervious area is about 20% of the total springshed area estimated to contribute water to Rock Spring. The simulation results on Rock Spring flow after the increase of impervious area indicated a decrease of approximately 10-15 percent in Rock Spring flow. While the application presented herein has focused on the discharge occurring at Rock Spring, it is most likely that other spring flows will also be affected and decreased.

In Figure 5.12, the groundwater flow simulations are presented for the before and after the increase of the impervious areas. Used for the presentation was the cross-section C-C as shown in Figure 5.11. It is noted that the areas between the Lake Apopka and the Rock Spring have the different potentiometric contours. The increase of the impervious area introduces the smaller head gradient, and thus lower Darcy's velocities are generated, resulting in the decrease of the spring flow.

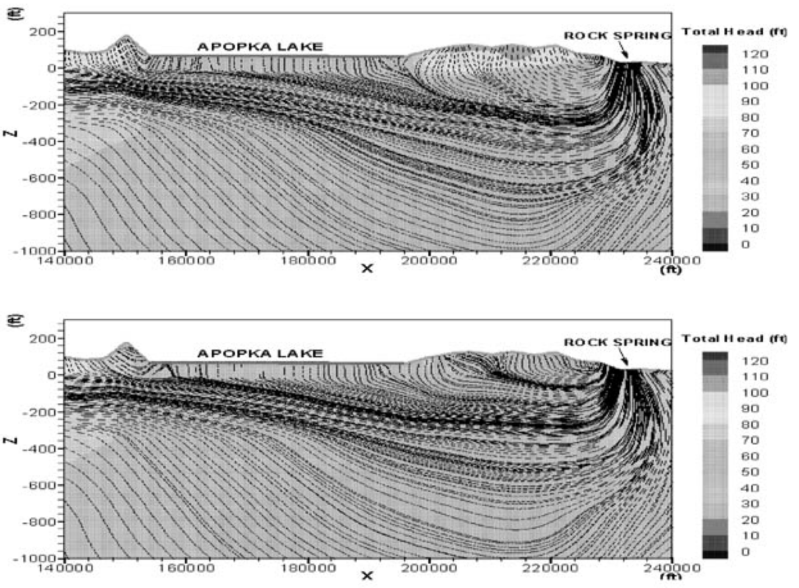

Figure 5.12: Groundwater flow along cross-section C-C before (upper) and after (lower)

The U.S. Geological Survey has defined "Most Effective Recharge Areas" as areas having greater than 25 cm of recharge per year. As discussed before, high-rate recharge areas are concentrated within the Wekiva Springshed area (Figure 5.10).

Protecting the high recharge areas that furnish water to the springs is very critical. Therefore, high-impact land use such as mining, industrial, heavy commercial and urban uses with extensive impervious surface should be of interest to protect the existing recharge potential within the springshed.

5.6.2 Springflow Relationship to Distance

The relationship between distance and the percentage of groundwater contribution for springs can be estimated using the reverse particle tracking method. The WASH123D model does not assume the recharge area but calculates the rate and extent. As can be imagined from Figure 5.12, particles within the vicinity of the Rock Spring travel faster than those faraway, and thus less time is needed for them to appear in the spring flow. For Rock Spring as shown in Figure 5.13, it is estimated the around 70 percent of the spring flow is contributed from within a 13-kilometer radius. Lake Apopka contains both recharge and discharge regions and contributes around 5 percent of the spring's discharge. In addition, more than 95 percent of the spring flow comes from within 22 kilometer of the vicinity.

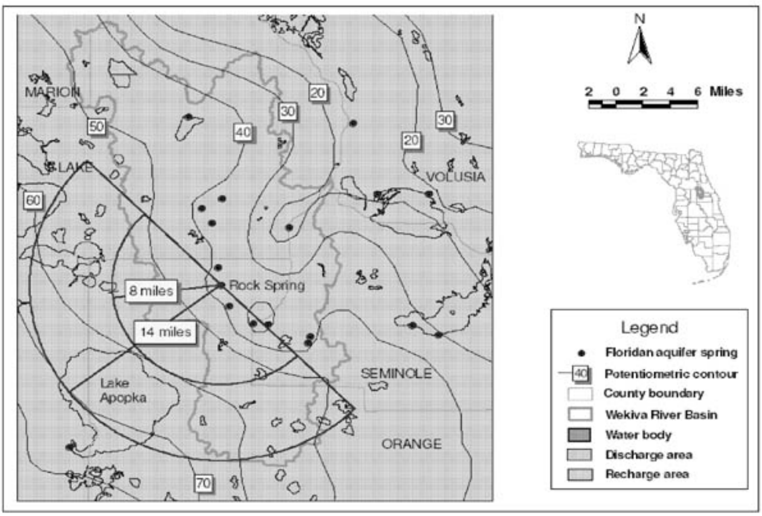

Figure 5.13: Distance and percentage of groundwater contribution for Rock Spring

An estimation of the springshed area has been made using the WASH123D model. The recommended Wekiva study area is approximately 1,244 square kilometers. The estimated area contributing groundwater to springs modeled using WASH123D of Wekiva is approximately 1,166 square kilometers, which is, by the authors' calculation, larger by only 52 square kilometers than the original assumed recharge area (WBATF, 2003). In Figure 5.14, the recommended Wekiva study area and estimated springshed are delineated. It is estimated that 60 percent of the springshed is located within the recommended Wekiva study area. Within the recommended Wekiva study area, there is about 40 percent high recharge zone (defined as the recharge rate larger than 20 cm/year). Also, approximately 60-65 percent of the estimated springshed is in high recharge zone.

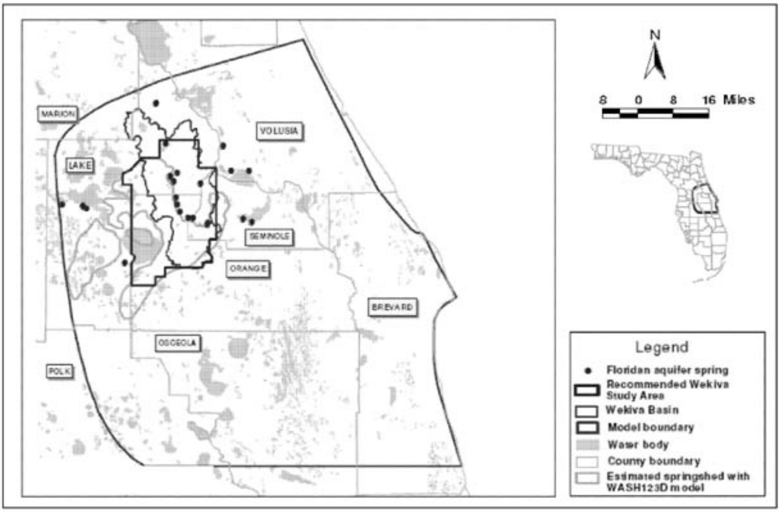

Figure 5.14: Recommended Wekiva study area and estimated springshed with WASH123D

5.7 CONLCUSIONS

A first principle, physics-based watershed model WASH123D has been applied to conduct the groundwater analysis for Wekiva Basin. The region of study is centered upon Seminole and Orange counties but includes most of Brevard, Lake, and Osceola counties plus parts of the Marion, Polk, and Volusia counties. Numerically, the modeling domain was discretized into 437,576 triangular prism elements, connected at 249,057 nodes, and eight numerical layers were included in the simulation. Input hydrologic data include designation of boundary conditions, applied stresses, and properties of each numerical layer.

The Wekiva WASH123D model was run to evaluate the average, steady state 1995 hydrological conditions. The distribution of simulated Floridan aquifer system groundwater levels using WASH123D shows very good agreement with the field observations at corresponding locations. In addition, the simulated 1995 water levels mimic the topography on a regional scale.

Simulated rates of natural recharge range from less than 10 cm/yr to greater than 30 cm/yr. High-rate recharge areas (larger than 20 cm/year) are concentrated within the Wekiva River Basin, and over half of the Lake Apopka provides a source of recharge to the Wekiva springs. There are also high and moderate areas that extend farther south and west and also to the east within Seminole County. The estimated springshed area is approximately 1,166 square kilometers and the recommend Wekiva study area is approximately 1,244 square kilometers. It is estimated that 60 percent of the springshed is located within the recommended Wekiva study area, which is occupied by about 40 percent high recharge zone. Also, approximately 60-65 percent of the estimated springshed is in high recharge zone.

The impervious area was increased by about 155 square kilometers due to urbanization. This impervious area is about 20% of the total springshed area estimated to contribute water to Rock Spring. The simulation results on Rock Springs flow after the increase of impervious areas indicated a decrease of approximately 10-15 percent in Rock Spring flow. Therefore, high-impact land use such as mining, industrial, heavy commercial and urban area with extensive impervious surface should be of interest to protect the existing recharge potential within the springshed. The relationship between distance and the percentage of groundwater contribution for Rock Spring is estimated. Around 70 percent of the spring flow is contributed from within a 13-kilometer radius. Lake Apopka contains both recharge and discharge regions and contributes around 5 percent of the spring's discharge. In addition, more than 95 percent of the spring flow comes from within 22 kilometers of the vicinity.

A model is a device that represents an approximation of field conditions (Anderson and Woessner, 1992). The Wekiva WASH123D model shows decent simulation results compared with the observation data even without calibration. However, the model results are limited by the simplification of the conceptual model upon which the numerical model is based, the element size, the inaccuracies of measurement data, and incomplete knowledge of the spatial variability of input parameters. For example, it is doubtful that the laminar flow assumption is valid within the vicinity of the springs. It is also doubtful that the elevations of the lakes are not functions of time since interactions between surface water bodies and the subsurface need to be considered to ensure mass conservation. Furthermore, element refinement around springs, wells, and lakes may be required to increase the simulation accuracy.

All input stresses in this study represent average, steady state conditions. In the designation of WASH123D, both sources/sinks and all types of boundary conditions can be considered spatially- and/or temporally-dependent plus transient simulations can be processed based on the appropriate initial conditions. In closing, the Wekiva WASH123D model can be further applied to examine the potential long-term, transient impact due to changes of stresses.

ACKNOWLEDGEMENT

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CHAPTER 6

Groundwater Contamination Potential from Infiltration of Urban Stormwater Runoff

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ABSTRACT: Infiltration of urban stormwater runoff is becoming more common in many areas of the United States and throughout the world. For infiltration to be successful as a stormwater management strategy, however, it is imperative that suitable sites be identified and their lifespan and maintenance requirements be predicted. Infiltrating in unsuitable areas can result in groundwater contamination and associated impairment of beneficial uses. This paper reviews several categories of urban stormwater pollutants and their groundwater contamination potential. Case studies, both from the field and the laboratory, then are presented to summarize information regarding the potential factors affecting pollutant transport through the vadose zone. Finally, two levels of prediction models to assess the susceptibility of groundwater contamination from urban stormwater infiltration are presented. However, the current use of these models is limited by the lack of supporting data, especially the lack of fundamental research into the processes controlling stormwater treatment in the vadose zone. Preliminary methods to address these deficiencies are presented, but, similar to the models, these are screening tools only. Until these deficiencies are addressed (for example, as outlined at the end of the chapter), the models should be seen as providing guidance and as preliminary screening tools, not as providing absolute answers.

6.1 INTRODUCTION

The U.S. Environmental Protection Agency (US EPA) reported that less than 60% of the rivers and streams in the United States fully support their beneficial uses. A wide variety of pollutants and sources cause these impaired uses, but runoff from urban, including transportation, and agricultural sources dominate (Field and Turkeltaub, 1981; Pitt and Bozeman, 1982; Pitt and Bissonnette, 1984; Pitt, 1994, which includes an extensive literature review). The infiltration of stormwater has become an increasingly popular option in attempts to restore surface water hydrographic characteristics to relatively natural conditions and as a means of reducing the discharge of pollutants to the surface receiving waters (literature summarized in Welty et al., 2008; Clark et al., 2006). Stormwater managers/engineers therefore are faced with the complex task of planning runoff control systems that prevent receiving water degradation from both flow and pollution to both groundwaters and surface waters. The stormwater manager must decide whether the "disposal" of runoff should be through infiltration, surface discharge, evapotranspiration, or a combination of the three (Pitt and Clark, 2008). Traditionally, runoff has been managed by detaining water throughout the watershed in storage basins. The assumption was that receiving water flooding problems (the only issue originally considered in drainage system designs) were due to the peak flow rates in the urban streams. Recent research has shown, however, that the use of detention by itself has not solved stream problems. With detention-only, peak flow rates may be attenuated (assuming careful placement of the detention facilities throughout the watershed), but the total runoff volume has not been markedly reduced, and pollutant discharges have increased with the increased development. In addition, the use of detention has not addressed two major problems due to urbanization – the reduction of groundwater levels by removing/building on many natural groundwater recharge areas and groundwater contamination due to pollutant discharges in any stormwater that is infiltrating. A commonly accepted current objective in stormwater management is to attempt to match many aspects of pre and post-development hydrology; mimicking not only peak flow rates, but also runoff volume and the flow-duration curve. Many agencies also require pollutant discharge reductions (in concentration and/or mass discharges), compared to conventional development.

Stormwater management is now addressed through the implementation of control practices at the site in question or at a location in the conveyance system prior to discharge to an above- or below-ground receiving water body. These practices may be selected based on site-specific conditions and discharge requirements, but often they are selected based on past history of what has worked in the municipality, the experience of the planners with specific practices, and regulatory requirements on the federal, state and local level. As more states require treatment of runoff prior to discharge to remove chemical pollutants (such as suspended solids or phosphorus) and require mimicking of pre-development hydrology to ameliorate adverse effects such as groundwater depletion, filtration and/or infiltration are of greater interest to stormwater managers. However, groundwater contamination potential, service lifetime, and maintenance requirements rarely are considered by stormwater managers selecting stormwater infiltration options.

6.2 GROUNDWATER IMPACTS FROM STORMWATER INFILTRATION

The groundwater contamination potential of urban runoff pollutants has been addressed through several literature reviews, including Andleman et al. (1994), Pitt et al. (1996), and the Water Environment Research annual reviews may be seen at a web site (Clark et al., 2006).

6.2.1 Nutrients

Many researchers have concluded that the potential exists for nitrate contamination of groundwater due to stormwater runoff (Hampson, 1986; Schiffer, 1989; German, 1989; Bannerman, et al., 1993), and have linked nutrient movement to soil characteristics (Robinson and Snyder, 1991; Butler, 1987; Wilde, 1994; Gold and Groffman, 1993; Crites, 1985; White and Dornbush, 1988; Ragone, 1977). Most report nitrate contamination of groundwaters in urban areas have been associated with poorly located septic tanks and leach fields, and not from stormwater, as nitrates are usually in relatively low concentrations in stormwater, reducing their contamination potential to groundwaters. Phosphorus is also a nutrient of concern in stormwater and can be present in problematic concentrations. They are typically associated with fertilizers (Lauer, 1988a, b), detergents, and road runoff (Schiffer, 1989). Depending on soil conditions, phosphorus is much less mobile than nitrates in the vadose zone and is effectively removed through ion exchange.

6.2.2 Pesticides

Groundwater contamination of pesticides from stormwater runoff has been documented by several researchers, including German (1989), Domagalski and Dubrovsky (1992), Wilson, et al. (1990). However, Gold and Groffman (1993) reported leaching losses from residential lawns to be low for dicamba and 2,4-D (<1 μg/L), when using application rates recommended for residential lawn care, while during one study in California, none of the ten chlorinated pesticides and herbicides investigated were found (Nightingale, 1987a,b; Salo, et al., 1986).

Heavy repetitive use of mobile pesticides on irrigated and sandy soils likely contaminates groundwater. Fungicides and nematocides must be mobile in order to reach the target pest and hence, they generally have the highest groundwater contamination potential. Pesticide leaching depends on patterns of use, soil texture, total organic carbon content of the soil, pesticide persistence, and depth to the water table (Shirmohammadi and Knisel, 1989). Estimates of pesticide mobility can be made based on volatilization, sorption, and solubility of the compound (Armstrong and Llena, 1992). Jury, et al. (1983) demonstrated that the potential for pesticide removal in the soil is related both to the pesticide's ability to sorb to the soil and the potential for biodegradation prior to reaching the groundwater.

6.2.3 Other Organic Compounds

Many researchers have documented groundwater contamination due to toxic organic compounds in runoff (German, 1989; Wilson, et al., 1990; Ku and Simmons, 1985; Wilde, 1994). Many of these were associated with source areas where toxic organic pollutant concentrations exceeded typical stormwater concentrations. Infiltration is therefore inappropriate at critical source areas without adequate runoff pretreatment. Groundwater contamination from these organics, like from other pollutants, occurs readily in areas with pervious soils, such as sand and gravel, and where the water table is near the land surface (Troutman, et al., 1984). The removal of these pollutants from the soil and recharge water can occur by one or more methods: volatilization, sorption, and/or degradation (Crites, 1985). Mobility classes similar to those listed for pesticides have been developed since the removal mechanisms are similar to those of pesticides. These rankings are directly related to the compound's water-to-organic carbon partitioning coefficient's K_{oc} value, where a high K_{oc} indicates that the pollutant is able to sorb to the soil's organic matter (Jury, et al., 1983).

6.2.4 Pathogens and Indicator Organisms

Microbial groundwater contamination from urban runoff, typically through the use of direct recharge devices, has been documented (see Clark, et al., 2006 for a review). Removal of these organisms is dependent on the soil chemical properties that promote adsorption and retention. Viral adsorption in soils is promoted by increasing cation concentration, decreasing pH, and decreasing soluble organics (U.S. EPA, 1992) and is controlled by both the efficiency of short-term virus retention and the long-term behavior of viruses in the soil (U.S. EPA, 1992; Crites, 1985). The downward movement and distribution of viruses are controlled by convection and sorption, along with hydraulic dispersion mechanisms. Since the movement of viruses through soil to groundwater occurs in the liquid phase and involves water movement and associated suspended virus particles, the distribution of viruses between the adsorbed and liquid phases determines the viral mass available for movement.

The major bacterial removal mechanisms in soil are straining at the soil surface and at intergrain contacts, sedimentation, and sorption by soil particles (Crites, 1985). Factors such as temperature, pH, metal concentration, nutrient availability and other factors affect the ability of a bacterial colony to survive in the water or soil (Ku and Simmons, 1985). Bacteria survive longer in acid soils and when large amounts of organic matter are present. Bacteria and larger organisms in wastewater are usually removed during percolation through a short distance of soil (U.S. EPA, 1992). The concern for groundwater contamination is that viruses and bacteria have been shown to migrate in the soil profile. Infiltrating stormwater may collect previously-deposited microorganisms and transport them to the groundwater. Bacterial transport depth appears to be related to total dose on the soil and fluid velocity through the soil (Unice and Logan, 2000; Camesano and Logan, 1998). It also is related to the chemical characteristics of the soil. Clark (2000) studied pollutant transport in intermittently-saturated soil filter columns. All soils (peat-sand, compost-sand, sand, and loam) could remove E. coli, with removals in the peat-sand column approaching 90%. Enterococci removal was statistically significant only in the peat-sand column. Microbially-contaminated sediments often function as a reservoir in which microorganisms can persist (Jensen, 2002).

6.2.5 Metals

In general, studies of recharge basins receiving large stormwater metal loads show that most heavy metals are removed either in the basin sediment or in the uppermost layers of the soil beneath the recharge basin (Ku and Simmons, 1985; Hampson, 1986). Metals' removal by soil may be through one of several processes, including:

soil surface association, precipitation, occlusion with other precipitates, solid-state diffusion into soil minerals, and biologic system or residue incorporation (Crites, 1985). Most of these removal processes are pH-dependent, as is the solubility of most metals. In general, a metal's solubility increases as the solution's pH decreases (Wilde, 1994).

Most of the heavy metals in stormwater are associated with particulates and can be readily strained-out through filtration as the water infiltrates into the soil (Pitt, et al. 1995). Therefore, these direct physical removal mechanisms are likely more important than chemical removal mechanisms for most heavy metals. Similar to that generated for the pesticides, mobility class rankings have been generated for the filtered forms of the metals, as shown in Table 3 (Armstrong and Llena, 1992).

6.2.6 Salts

Soil is not very effective at removing most salts. Once salt contamination begins, the movement of salts into the groundwater can be rapid. The salt concentration may not lessen until the source of the salts is removed (Huling and Hollacher, 1972; Rail, 2000). For example, in Maryland, the nearby use of deicing salts and their subsequent infiltration to the groundwater shifted the major-ion chemistry of the groundwater to a chloride-dominated solution. Although deicing occurred only three to eight times a year, increasing chloride concentrations were noted in the groundwater throughout the 3-year study, indicating that groundwater systems are not easily purged of conservative contaminants, even if the groundwater flow rate is relatively high (Wilde, 1994).

6.3 CASE STUDIES: FIELD INFILTRATION DEVICES TREATING URBAN STORMWATER RUNOFF

The historic case studies of stormwater infiltration focused primarily on characterizing the water quality below infiltration basins and providing fundamental descriptions of the soil based on physical characteristics, such as soil texture. Several historical and current field studies are summarized below. Additional studies have been performed on the infiltration of reclaimed water. Those studies (such as Schneider et al., 1987; Lloyd et al., 1988; Ehrlich et al., 1979a, b; and the summary in Andleman et al., 1994), while not focusing on stormwater, also may provide indications as to the pollutant removal potential of vadose zone soils.

6.3.1 Case Study #1: Water Quality below Stormwater Infiltration Basins in Long Island, NY (Ku and Simmons 1985).

To determine whether urban stormwater infiltration posed a threat to groundwater quality, forty-six storms were monitored at five recharge basins in different land-use areas (strip commercial, shopping-mall parking lot, major highway, low-density residential, and medium-density residential). The results of the analyses for selected EPA priority pollutants in both the stormwater and groundwater are shown in Figure 6.1. The indicator bacteria concentrations (fecal coliforms, fecal streptococci and total coliforms) also were monitored.

With the exception of the medium density residential site and lead, the soils typically were able to remove the pollutants. For chloride and total nitrogen, lags were seen between the high stormwater concentrations and the groundwater concentrations (time series data not shown). In terms of median concentrations over the monitoring period at each basin, the results were inconsistent. This may be an artifact of the lack of groundwater data during a few storm events for these parameters (typically 6 – 10 storms for each site). For the indicator bacteria, stormwater median concentrations ranged from 10^8 to 10^{10} MPN/100 mL. No indicator bacteria were detected in the groundwater beneath any of the recharge basins.

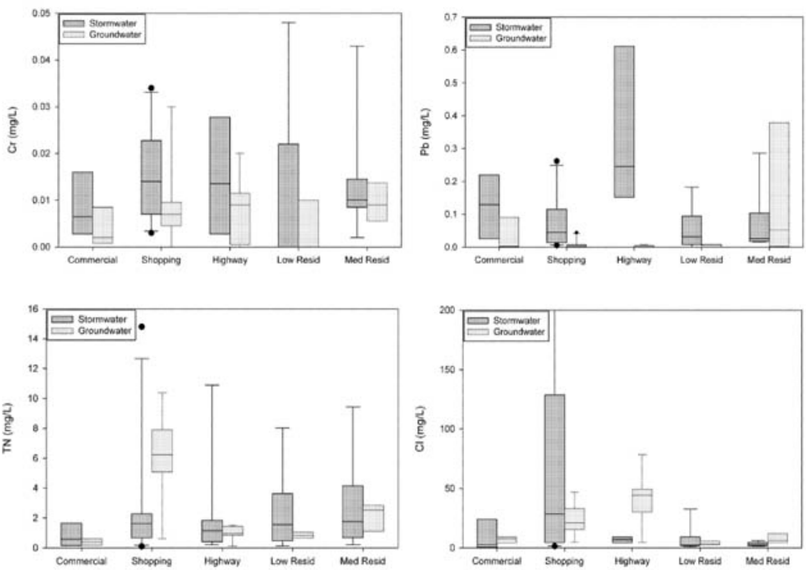

Figure 6.1: Effects of stormwater infiltration on groundwater quality (adapted from Ku and Simmons, 1985)

6.3.2 Case Study #2: Centre Routier in Lyon, France – Soil Accumulation over twenty-plus years of operation (Dechesne et al., 2004; Barraud, et al., 2005)

Soil samples were collected at ten locations throughout this infiltration basin. The soil was separated into depth profiles with each depth interval analyzed individually. These soil analyses allowed for comparisons of pollutant accumulation by depth and location in the basin (Figures 6.2 through 6.5). The general results showed that the highest concentrations of many pollutants were found in the soils near the lowest elevation in the infiltration basin (area 7), likely because of the extended (comparatively) water ponding in that part of the basin. The other area of contamination was near the inlet (area 2). For certain pollutants, other areas of the basin (1, 4 and 9) also had elevated results, likely due to the interactions of those pollutants with the soil, as represented by the CEC and TOC. In addition, the buildup of runoff sediment was very visible and was also seen by the significant difference in the particle size distribution in the near surface soils compared to the three other soil depths analyzed.

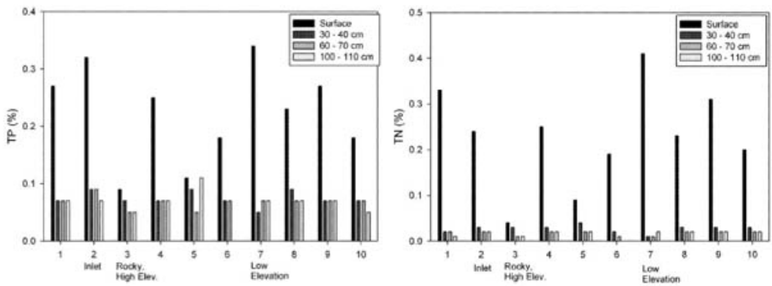

Figure 6.2: Nutrient Concentration by Soil Depth beneath a 20+-year old Infiltration Basin (adapted from Dechesne et al., 2004)

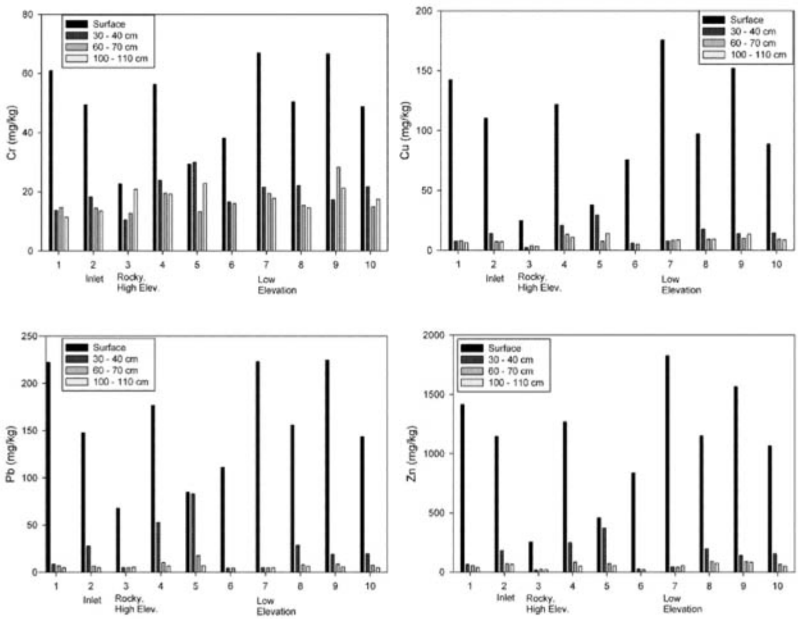

Figure 6.3: Metal and Major Ion Concentrations in Soils beneath a 20+-year Infiltration Basin (adapted from Dechesne et al., 2004)

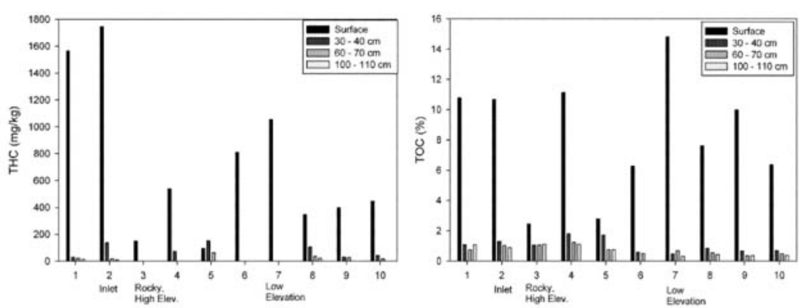

Figure 6.4: Total Hydrocarbon and Organic Carbon Concentrations in Soils beneath a 20+-year Infiltration Basin (adapted from Dechesne et al., 2004)

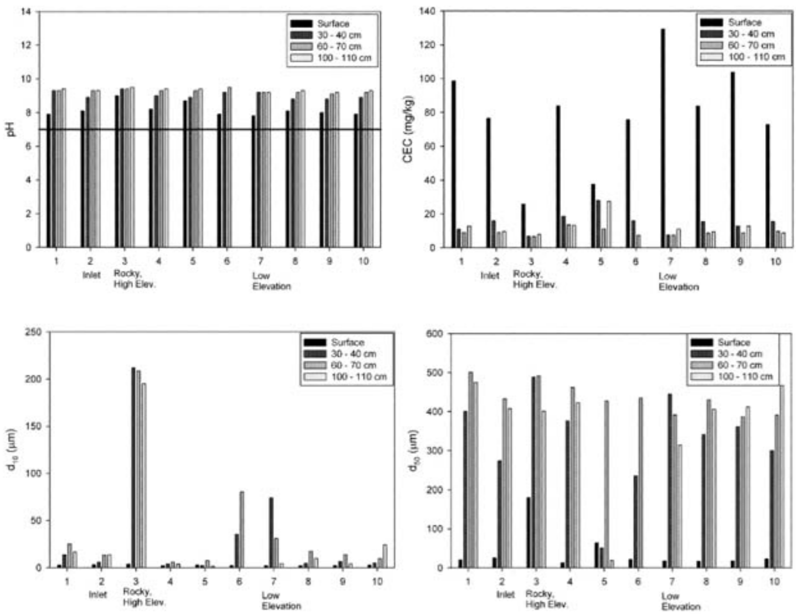

Figure 6.5: Analysis of Soil Characteristics by Depth beneath a 20+-year-old Infiltration Basin (adapted from Dechesne et al., 2004)

The surface soil layer consisted of finer particles and had a higher organic content. Soil organic matter contains surface groups that retain many stormwater pollutants, including organics and positively-charged metals. Therefore, pollutant retention was greatest in the surface layer with little accumulation in the lower layers. No water monitoring data was available to indicate whether pollutants were passing through the system; however, the low levels of pollutants in the lower soil layers suggests that the surface layer to 30 – 40 cm layer treated the infiltrating water.

6.3.3 Case Study #3: North Carolina Field Bioretention Facility Assessment (Hunt et al. 2006)

Three North Carolina bioretention facilities were monitored for one year. The results described below focus on two of the facilities. The bioretention facilities had with depths of 4 feet; one was built on a Madison clay loam soil with a post-construction saturated hydraulic conductivity of 7.6-38.1 cm/hr (3-15 in/hr) and one on a White Store-urban complex soil of clay, clay loam, and silty clay with a post-construction saturated hydraulic conductivity of 3.3-7.8 cm/hr (1.3-3.1 in/hr). The clay loam soil had a pH of 6.2 – 6.6; a CEC of 5.6 – 7.0 meq; and a P-index of 86 – 100. The urban complex soil had a pH of 5.4 – 6.0; a CEC of 1.9 – 2.4 meq; and a P-index of 4 – 12. The P-index (phosphorus index) describes the relative risk of phosphorus being lost from a native soil. Eleven storms were monitored over the course of the year with an average rainfall of 3.3 cm (1.32 inches) per storm. Influent and effluent loads were compared for the two basins (Figure 6.6 with clay loam labeled "high" and urban complex labeled "low").

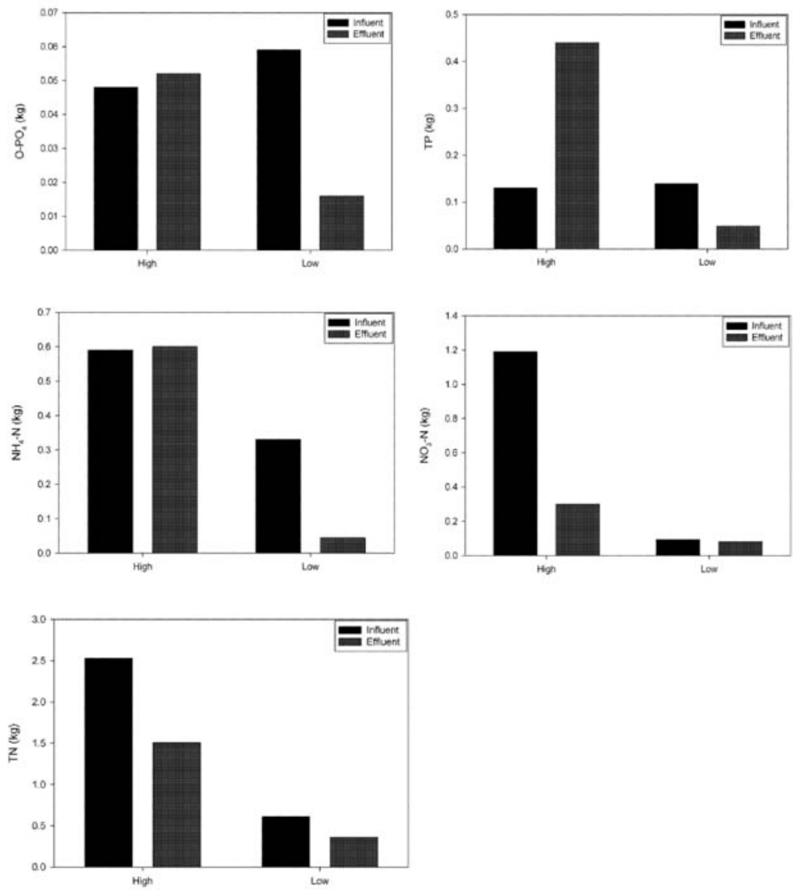

Figure 6.6: Influent and effluent nutrient loads in one-year of monitoring at two bioretention facilities (adapted from Hunt et al. 2006). NOTE: Clay loam soil = High; Urban complex soil = Low

Both bioretention units produced a 40% reduction in total nitrogen, but NO_3-N reduction varied from 13-75%; the higher NO_3-N removal capacity of the Clay Loam unit may have been caused by anaerobic conditions present there but not in the Urban Complex unit. Total phosphorus reduction seemed to be related to the P-index of the bioretention unit's soil; the unit with a P-index of 86-100 caused a 240% increase of total phosphorus while the unit with a P-index of 4-12 caused a 65% reduction of total phosphorus. The clay-loam soil cells caused increases in TKN, NH_4-N, TP, and ortho-P that were significant ($p < 0.05$). The outflow volumes were 50% less than the inflow volumes of the bioretention units. Outflow volume was significantly ($p < 0.05$) related to seasonal weather; the winter months had a lower reduction in inflow to outflow volumes. Pollutant mass removals are correlated with the volume reductions, and not with substantial concentration reductions.

6.3.4 Case Study #4: Effects of Compost Amendments in Hydraulically Poor Quality Soils (adapted from Pitt et al. 1999)

This research project, funded by the U.S. EPA and performed by the College of Forestry Resources (CFR) at the University of Washington (UW), measured the benefits of amending urban soils with compost. It was anticipated that compost-amended soils would improve infiltration characteristics of these problematic soils, along with removing pollutants by filtration/sorption. Most residential housing developments in the Seattle area are sited on the Alderwood soil series due to their wide distribution and inherent stability. The Alderwood soils are characterized by a compacted subsurface layer that restricts vertical water flow. When disturbed (and particularly when disturbed with cut and fill techniques as with residential or commercial development), uneven water flow patterns develop due to restricted permeability. The testing used the Urban Water Resource Center design of large plywood beds for containing soil and soil-compost mixes. The test sites were at the University of Washington and two public schools. The plots were planted with commercial turfgrass (at the University in 1994 and at the schools in 1997). Field application of compost material that mimics the techniques used in this study would be done by applying 4 inches of compost onto the surface of the soil and tilling to a total depth of 30.5 cm (12 inches), including the compost amendment (8 inches into the soil).

Both surface runoff and subsurface flow were separately collected following seven rainfall periods during the months of December 1997 through June 1998. The use of compost-amended soil resulted in significantly increased infiltration rates compared to soil alone. These infiltration rate increases (from 1.5 to 10 times the untreated rates) would substantially decrease the runoff volumes and flow rates during rain storms from turf areas. The native soil structure below the amended depths likely would prevent much of this increased infiltrating water from reaching deeper groundwaters, but the compost amendments would still improve surface water flow characteristics (see also Harrison, et al. 1997).

Surface runoff and subsurface flows were also monitored over several extended periods at the test plot sites. The newer test plots at the schools showed significant decreases in surface runoff for the compost-amended test plots, compared to the soil-only test plots. These improvements also would be important from the perspective of reducing pollutant loads. In addition, the evapotranspiration rates all increased with compost-amended soils, although by only a very small amount at one of the UW test plot pairs. The increase in evapotranspiration ranged from about 33 to 100 percent at the newer school sites.

Table 6.1 summarizes the average concentrations for surface runoff and subsurface flow samples separated by "soil-only" test plots and "soil plus compost" test plots. This table shows the average observations along with the coefficient of variations (unit ratios of the standard deviation divided by the average value). The table only shows data for tests having both surface runoff and subsurface flow samples. The subsurface flows in the soil-only test plots mostly had lower concentrations than the associated surface runoff. For the compost-amended test plots, there were more constituents with higher concentrations in subsurface flows compared to the surface runoff. In addition, the increases were generally larger (as much as 2.5 times greater) than for the increases observed at the soil-only test plots.

The surface runoff from the compost-amended soil sites had greater concentrations for almost all constituents, compared to the surface runoff from the soil-only test sites. Interestingly, the exceptions were for the cations Al, Fe, Mn, Zn, and Si, plus toxicity, which were all lower in the surface runoff from the compost-amended soil test sites. The concentration increases in the surface runoff flows from the compost-amended soil test site were large, typically in the range of 5 to 10 times greater. Subsurface flow concentration increases for the compost-amended soil test sites were common and of similar magnitude. Toxicity tests indicated reduced toxicity with filtration at both the soil-only and at the compost-amended test sites.

Table 6.1: Mean (and COV) values for surface runoff and subsurface flows

	Soil-only plots		Soil plus Compost Plots	
Constituent (mg/L, unless noted)	Surface	Subsurface	Surface	Subsurface
PO_4-P	0.27 (1.4)	0.17 (2.0)	1.9 (1.0)	1.8 (1.2)
TP	0.49 (1.0)	0.48 (2.2)	2.7 (0.9)	2.5 (1.1)
NH_4-N	0.65 (1.7)	0.23 (1.3)	4.1 (1.8)	3.5 (3.0)
NO_3-N	0.96 (1.4)	1.2 (2.5)	3.0 (1.6)	6.2 (2.8)
TN	2.5 (0.9)	1.9 (0.7)	8.4 (1.5)	10 (2.1)
Cl	2.4 (1.0)	2.1 (0.9)	6.7 (1.1)	5.0 (1.6)
SO_4-S	0.68 (1.1)	0.95 (2.0)	1.5 (0.9)	2.4 (1.4)
Al	11 (1.8)	1.7 (2.1)	0.7 (1.6)	2.4 (1.6)
Ca	12 (1.5)	17 (0.7)	18 (1.1)	35 (1.1)
Cu	0.01 (0.8)	0.01 (1.6)	0.02 (1.2)	0.02 (0.9)
Fe	4.6 (1.4)	2.8 (1.6)	1.2 (1.5)	2.6 (0.9)
K	5.4 (1.0)	4.6 (0.8)	30 (1.3)	34 (1.6)
Mg	3.9 (0.8)	5.0 (0.6)	5.8 (1.2)	10 (1.1)
Mn	0.75 (2.9)	0.41 (2.8)	0.36 (1.9)	0.80 (2.4)
Na	3.8 (0.9)	3.4 (0.5)	3.2 (0.8)	4.6 (1.2)
S	1.1 (0.8)	1.3 (1.5)	2.5 (0.8)	4.7 (1.6)
Zn	0.2 (1.2)	0.05 (2.2)	0.14 (1.1)	0.03 (1.8)
Si	26 (1.7)	8.9 (0.5)	4.2 (1.1)	11 (0.7)
10^{th} percentile size (μm)	2.9 (0.7)	3.1 (0.4)	2.8 (0.3)	3.5 (0.6)
50^{th} percentile size (μm)	12 (1.0)	13 (0.6)	15 (0.4)	14 (0.7)
90^{th} percentile size (μm)	45 (0.5)	41 (0.5)	46 (0.4)	47 (0.6)
Toxicity (% light reduct.)	25 (0.7)	13 (0.5)	16 (0.8)	10 (1.1)

The mass discharges of water and nutrients were calculated for each sampling period. As noted previously, compost-amended soils caused increases in concentrations of many constituents in the surface runoff. However, the compost amendments also significantly decreased the amount of surface runoff leaving the test plots, at least for monitoring period. Table 6.2 summarizes these expected changes in surface runoff and subsurface flow mass pollutant discharges associated with compost-amended soils, using the paired data only. The paired data concentration increases were multiplied by the runoff reduction factors to obtain these relative mass discharge changes. The newer sites had greater runoff reductions, while the older sites had smaller concentration increases associated with the added compost. All of the surface runoff mass discharges are reduced (by 2 to 50 percent of the unamended discharges). However, many of the subsurface flow mass discharges may increase, especially for ammonia (340% increase), phosphate (200% increase), plus TP, NO_3-N, and TN (all with 50% increases). Most of the other constituent mass discharges in the subsurface flows decreased.

Table 6.2: Pollutant mass discharges from surface runoff and subsurface flows at new compost-amended sites compared to soil-only sites (Ratios of Amended to Unamended Soil)

Constituent	Surface Runoff Discharges	Subsurface Flow Discharges
Runoff Volume	0.09	0.29
Phosphate	0.62	3.0
Total phosphorus	0.50	1.5
Ammonium nitrogen	0.56	4.4
Nitrate nitrogen	0.28	1.5
Total nitrogen	0.31	1.5
Chloride	0.25	0.67
Sulfate	0.20	0.73
Calcium	0.14	0.61
Potassium	0.50	2.2
Magnesium	0.13	0.58
Manganese	0.042	0.57
Sodium	0.077	0.40
Sulfur	0.21	1.0
Silica	0.014	0.37
Aluminum	0.006	0.40
Copper	0.33	1.2
Iron	0.023	0.27
Zinc	0.061	0.18

The compost likely has significant sorption capacity and ion exchange capacity that is responsible for pollutant reductions in the infiltrating water. However, the compost also leached large amounts of nutrients to the surface and subsurface waters. These results clearly show that amending soil with compost alters soil properties known to affect water relations of soils, including the water holding capacity, porosity, bulk density, and structure, as well as increasing soil C and N, and probably other nutrients as well. The mobilization of these constituents probably led to observed increases in P and N compounds in surface runoff compared to unamended soil plots. Results of the earlier Redmond-sponsored tests (Harrison, et al., 1997) were somewhat different than obtained from the current tests. Some of these differences were likely associated with the age of the test plots, plus different rainfall conditions, and other site characteristics. The results of the earlier study clearly showed that compost amendment is likely an effective means of decreasing peak flows from all but the most severe storm events, even following very wet antecedent conditions. The increases in water holding capacity with compost amendment shows that storms up to 0.8 inches total rainfall would be well buffered in amended soils and not result in significant peak flows, whereas without the amendment, a storm about 0.4 inches total rainfall would be similarly buffered.

In conclusion, adding large amounts of compost to marginal soils enhances many desirable soil properties, including improved water infiltration (and attendant reduced surface runoff), increased fertility, and significantly enhanced aesthetics of the turf.

Unfortunately, the compost also increased the concentrations of many nutrients in the runoff, especially when the site was newly developed, but with the increased infiltration of the soil, the nutrient mass in the surface runoff likely was decreased significantly, inferring that it was either retained by the soil and its biomass or transported to the groundwater. This is especially likely when the need for continuous fertilization to establish and maintain the turf is reduced, if not eliminated, at compost-amended sites. Further research is needed to determine if it is possible that less amounts of compost could be added to an urban soil and still obtain much of the observed benefits, but would have reduced problems associated with leaching. Hunt et al. (2006) highlighted the need to use compost with a lower P content in order to minimize or eliminate P leaching from the soil compost amendment.

Recent case studies have focused on newer infiltration-based treatment units, although several European researchers (Bardin et al., 2001; Barraud et al., 1999; Barraud, et al., 2005; Mikkelsen, 1997), plus Welker et al. (2006), have been investigating soil accumulation of pollutants in infiltration facilities that are several decades old. Research is ongoing at field installations and laboratories both in the U.S. and in other countries. Active researchers in the water-quality impacts of, and controlling factors for, infiltration through both engineered and natural soils include several researchers in Florida, Villanova University, the University of New Hampshire, North Carolina State University, the University of Maryland, the U.S. EPA Urban Watershed Management Branch (investigations in both New Jersey and Kansas City, Missouri), France, Germany, Norway, Australia, and Switzerland (Davis, 2008; Dechesne et al., 2005; Dechesne et al., 2001; Dierkes et al., 2006; Emerson et al., 2008; Ferguson, 1990; Fischer et al., 2003; Guo et al., 2006; Hathhorn and Yonge, 2005; Hatt et al., 2007; Hatt et al., 2008; Hsieh and Davis et al., 2007a,b; Hunt et al., 2008; Jayasuriya et al., 2007; Kumar et al., 2006; Kwiatkowski et al., 2007; Li and Davis, 2008a, b; Lucas and Greenway, 2008; Marcos et al., 2002; McKenzie and Irwin, 1988; Mikkelsen et al., 1997; Muthanna et al., 2007a,b; Nogaro et al., 2007; Osenbruck et al., 2007; Sample and Heaney, 2006; Sansalone and Glenn, 2007; SRF Consulting Group, 2004; Sun and Davis, 2007; Thomas, 2000; Van Cuyk and Siegrist, 2007; Warnaars et al., 1999; Welker et al., 2006; Zhang and Ross, 2007; Zheng et al., 2006; Zomorodi, 2006).

6.4 LABORATORY INVESTIGATIONS INTO SOILS' POLLUTANT REMOVAL ABILITY

The case studies described above indicated, first, that groundwater quality can be impacted from stormwater infiltration. They also indicate that certain soil properties, such as higher organic content, higher CEC, and low P-index, may be among the controlling characteristics of soils that decrease pollutant transport through the vadose zone. Research on stormwater filters (literature summarized in Clark 2000) has shown that the following characteristics, in addition to those described above, can be linked to improved pollutant removal ability: specific surface area of media, surface organic acid concentration, humic and fulvic acid concentration, anion exchange capacity (AEC), depth of media, increased contact time (typically through deepening the filter media), and maintenance of aerobic conditions. Other researchers have shown that pollutant removal is increased in vegetated infiltration facilities, when compared to non-vegetated facilities. Because these field tests could not explicitly determine which factors controlled the pollutant removal, several research groups, using laboratory- and pilot-scale setups, are investigating the soil characteristics that affect pollutant removal, with two goals: first, producing guidance that will assist stormwater planners in determining where infiltration is acceptable in terms of groundwater protection, and second, creating an optimized bioretention/infiltration basin media. This section highlights the activities of two research groups.

6.4.1 Laboratory Study #1: Optimizing Bioretention Media (Hsieh and Davis 2005)

The pollutant removal capabilities of eighteen soil columns were measured and compared with the results from six field bioretention units. The test water was a synthetic stormwater (pH = 7.0; Total Suspended Solids (TSS) = 150 mg/L; Total Dissolved Solids (TDS) = 120 mg/L; Phosphate (PO_4) = 3 mg/L as P; Nitrate (NO_3) = 2 mg/L as N; Ammonium (NH_4) = 2 mg/L as N; Lead (Pb) = 0.1 mg/L). Soil columns were composed of mulch (leaf litter and lawn clippings), two sands of differing particle size, and three soils of varying chemical properties and grain sizes. This overall infiltration media was created from various combinations of these sands, soils, and mulch. The soils showed high removals of oil and grease (O/G), TSS, and lead for most media combinations. One soil increased TSS during passage through the column; this occurred when this soil was the dominant portion of the media. Several media combinations that did not include soil resulted in lower Pb removal rates. In the synthetic stormwater, 56% of Pb in solution bonded to suspended solids, suggesting that removal of total suspended solids would also remove a large portion of Pb, but for two tests, TSS was removed but a corresponding decrease in Pb was not seen. Total phosphorus removal ranged from 4-99%. Mulch did not assist in phosphorus removal; soil and sand had mixed impacts. Nitrate removal ranged from 1-43% with mixed results from various media combinations with higher removals in mulch-containing combinations and potentially lower removals in the sandier mixes. Ammonium removal was low, <26%, for all media combinations and no trends were seen. The field observations reflected column study results; but, unlike the columns studies, organic matter was correlated to phosphorus removal. Also, total suspended solids levels reflected the age of the units, with newer units having lower removal of TSS.

6.4.2 Laboratory Study #2: Assessment of Sand and Soil-based Stormwater Filter/Bioretention Media (Hatt et al., 2008)

The purpose of project was to evaluate combinations of sand, soils and potential soil amendments for improving hydraulic and pollutant removal behavior in filtration and bioretention systems. The influence of time, cumulative sediment loading, cumulative water volume, wetting and drying cycles, and compaction on hydraulic conductivity was investigated. The test media included the following (designations relate to Column 1 in Table 6.3):

- Sand (S)

- Sandy Loam (SL)
- 80% Sandy Loam, 20% Hydrocell™ (a synthetic soil amendment) (SLH)
- 80% Sandy Loam, 10% Vermiculate, 10% Perlite (SLVP)
- 80% Sandy Loam, 10% Compost, 10% Mulch (SLCM)
- 60% Sandy Loam, 20% Compost, 20% Mulch on a Charcoal Drainage Layer (SLCMCH)

A summary of the results is given in Table 3 (adapted from Hatt et al., 2008). The standard deviations are based on three replicates of each media. Negative reductions (designated as NR in the table) indicate contributions from the media itself.

Table 6.3: Percent pollutant reductions in six soil and soil combination columns (Hatt et al. 2008)

Media	TSS	TP	TN	TOC	Cu	Mn	Pb	Zn
S	99 ± 1	97 ± 1	38 ± 1	59 ± 8	97 ± 1	94 ± 1	99 ± 1	99 ± 1
SL	93 ± 4	NR*	NR	NR	97 ± 1	NR	99 ± 1	99 ± 1
SLH	92 ± 3	NR	NR	NR	96 ± 1	NR	99 ± 1	98 ± 1
SLVP	90 ± 3	NR	NR	NR	94 ± 2	NR	95 ± 2	96 ± 4
SLCM	92 ± 4	NR	NR	NR	94 ± 1	NR	97 ± 1	96 ± 1
SLCM CH	96 ± 1	NR	NR	NR	93 ± 1	NR	97 ± 1	96 ± 1

*NR = Pollutant was not removed in these test columns. Pollutant was released from the media into the infiltrating water, resulting in effluent concentrations greater than the influent.

These results suggested that the primary cause of hydraulic failure was formation of a clogging layer at the filter surface. Sediment and heavy metals were effectively captured in the media; however, the soil-based filters leached nitrogen and phosphorus. The authors state that the release of manganese during testing may be an indication of changing chemical conditions in the media. Oxidized manganese would be expected to be well-retained; however, the development of anaerobic conditions could lead to release of certain pollutants. Similar results were seen by Clark (2000) and Clark and Pitt (2009a) for organic stormwater filter media.

After the testing was completed, the media columns were analyzed for pollutant accumulation by depth. The pollutant profiles revealed significant accumulation of all pollutants in the top 20% of the filter profile (80 cm media depth), suggesting that elevated discharges of nutrients was due to leaching of native material, rather than failure to remove incoming pollutants. The release of pollutants from the media itself indicates a need for a drainage or lower media layer in engineered infiltration systems that can capture those pollutants. One possibility is the installation of an expanded mineral drainage layer that has been chemically or heat-activated to improve its pollutant removal; Long (2007) investigated several expanded minerals for their pollutant removal ability.

6.4.3 Laboratory Study #3: Pollutant Removals by Horizon in a Silty Loam and Loamy Sand (Treese et al., 2008)

Two Pennsylvania soils, a Wharton silty loam and Leetonia loamy sand were examined for their ability to remove pollutants from stormwater runoff. Twenty columns of each soil were collected intact in the field. In the laboratory, five columns were left undisturbed (with the exception described below), five columns retained only the O horizon, five retained the A horizon and five retained the A and B horizons for the loamy sand and the A1 and A2 horizons for the silty loam. Some of the tested columns also represented a disturbed soil, such as would be created when amending a natural soil to improve infiltration. The equivalent of a 3-inch rain was applied to each column approximately every week and the effluent analyzed. After every eight infiltration events, one column of each soil of each horizon was sacrificed for soil analysis. The results from the initial water testing are shown in Figures 6.7 through 6.10.

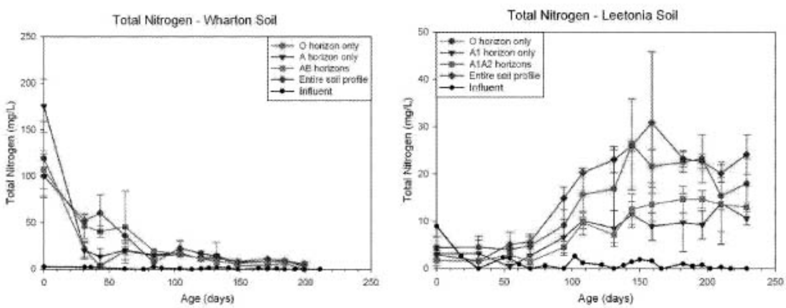

Figure 6.7: Total nitrogen: Wharton silt loam and Leetonia loamy sand

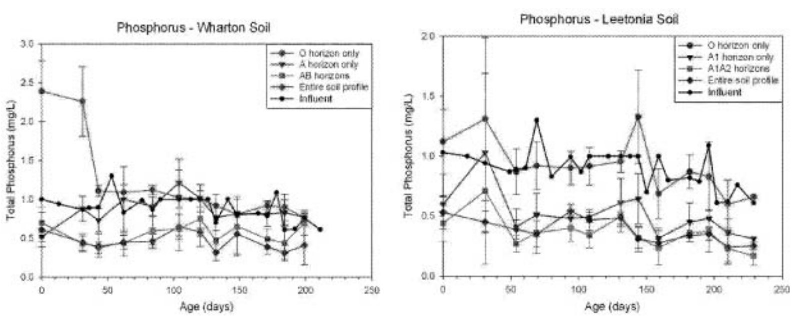

Figure 6.8: Total phosphorus: Wharton silt loam and Leetonia loamy sand

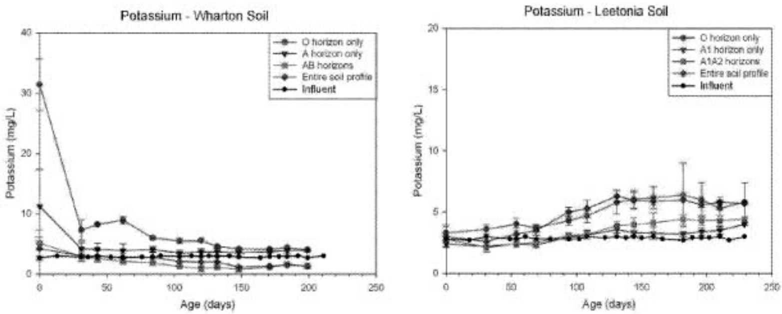

Figure 6.9: Potassium: Wharton silt loam and Leetonia loamy sand

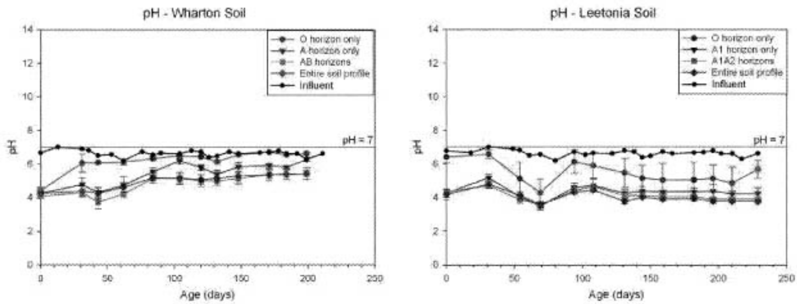

Figure 6.10: pH: Wharton silt loam and Leetonia loamy sand

These results showed leaching of total nitrogen and removal of total phosphorus by all soil horizons of both soil types, except for the early release of phosphorus from the silty loam (disturbed) soil. The phosphorus from this soil returned to levels similar to the influent after three infiltration events. This initial increase may have been the result of soil disturbance, which is being further investigated. Potassium and sulfate removal occurred early in the AB/A_1A_2 and OAB/OA_1A_2 soil horizon columns of both soils, but may only be temporary. The lower horizons and the entire profile are able to retain the leaching potassium and sulfate from the organic horizon. All soil horizons of both soil types have lowered the pH of influent stormwater and increased conductivity, turbidity, color, and hardness.

These laboratory studies confirm the importance of the organic content (O horizon) in removing pollutants from infiltrating stormwater runoff. However, as both the field and laboratory studies show, there appears to be an optimal organic content below which removal is not optimized, but above which, nutrient release is a concern. Future research should address these issues plus other chemical characteristics which affect pollutant removal, with the goal of improving the prediction of when stormwater infiltration may pose a threat to soil and/or groundwater health.

6.5 PREDICTING POLLUTANT REMOVAL IN THE VADOSE ZONE

Another important question that stormwater managers should address, but often is overlooked, when designing and installing infiltration devices, is their lifespan and associated maintenance needs. Stormwater that percolates to the groundwater relies on the soil profile (either an engineered or natural soil) to provide treatment. At some point in time, this treatment capacity may become exhausted. Therefore, methods are needed to predict the lifespan of infiltration devices. In addition, siting issues have to be addressed. The prediction of groundwater contamination potential can be very complex – depending on the concentration and form of the pollutant, the characteristics of the soil, and the rate at which water moves through the soil. Mobility is compound-specific and depends on the soil matrix (mostly the soil texture and organic content). Soil characteristics such as organic content, pH, and permeability play an important role in pollutant movement. Bardin et al. (2002) described a multi-parameter approach to assessing stormwater infiltration strategies as part of the design process. The Bardin et al. (2002) approach includes parameters such as flooding frequency, pollution concerns, life-cycle analysis, ease of maintenance, security, user perceptions, plus others. These parameters are of critical importance when addressing siting issues. This section, however, will focus, not on the social, security, and economic issues associated with urban infiltration devices, but only on the chemical and engineering factors. Clark and Pitt (2007) described two levels of modeling that can be used to assess whether a site is potentially suitable for stormwater infiltration. The computer-model can predict roughly lifespan and maintenance needs.

The first is a simplified method that links in a chart the mobility of the pollutant, the fraction in the filterable (dissolved) phase, and the concentration of the pollutant. This information is combined with the information on the soil type and general soil reactivity to provide a mobility class depending on the type of infiltration device used. The second method is a vadose-zone model, developed for predicting pollutant movement beneath a landfill. This model, unlike the simplified method, predicts concentrations in the groundwater and at various depths in the vadose zone given input parameters of rainfall amount, soil chemistry, and pollutant concentration. Specialized stormwater quality models (such as RECHARGA, http://dnr.wi.gov/runoff/stormwater/technote.htm, and WinSLAMM, http://www.winslamm.com/prod01.htm) offer detailed sizing and performance information for bioretention devices, including soil retention of pollutants. Clogging and consumption of pollutant retention capacity of the devices, are being added to these models to assist stormwater managers understand the maintenance needs of these devices for their specific conditions.

6.5.1 Simplified Method for Predicting Groundwater Contamination

Table 6.4 summarizes some of the pollutants found in stormwater that may cause groundwater contamination problems. Pitt, et al. (1994) used this information to examine risks associated with discharging stormwater to the subsurface and associated potential groundwater contamination. Characteristics that affect potential groundwater contamination include:

- High mobility (low sorption potential) in the vadose zone,
- High abundance (high concentrations and high detection frequencies) in stormwater, and
- High soluble fractions (small fraction associated with particulates that could be removed at the soil surface by straining, or by common sedimentation treatment).

It is possible to assess the need for pre-treatment of stormwater to reduce the potential pollutant loadings before infiltration. Results of a simplified method that can be used to predict groundwater contamination potential are shown in Tables 4 through 7. Sediment control is critical for long-term infiltration success since solids will clog infiltration devices, resulting in frequent maintenance or failure. In addition, solids' pretreatment will reduce the loading of particulate-bound pollutants (common) to the infiltration device. Pre-treatment that can remove dissolved fractions of the contaminants (rare) also increases the duration that the vadose zone soils can capture or sorb filterable pollutants by ion exchange, or other processes.

This simplified method is based on the following assumptions. The contamination potential is the most critical rating of the influencing factors. As an example, if no pretreatment was to be used before percolation through surface soils, the mobility and abundance criteria are most important. The filterable fraction would not be important since no pretreatment for the removal of particulate pollutants is being used. If sedimentation pretreatment is to be used before surface infiltration, then some of the pollutants will likely be removed before infiltration. In this case, all three influencing factors (mobility, abundance in stormwater, and soluble fraction) are important. If subsurface injection (with minimal pretreatment) is used, then only abundance is important. If the pollutant is present in high concentrations, it will likely have an adverse effect on the groundwater; attenuation through the vadose zone would be insignificant as the water would bypass it if using direct injection.

Table 6.4: Groundwater contamination potential for stormwater pollutants

Compound Class	Compounds	Mobility (Worst Case: sandy/low organic soils*)	Abundance in stormwater	Fraction filterable (dissolved)
Nutrients	Nitrate	I	Low/moderate	High
Pesticides	2,4-D	I	Low	Likely low
	γ-BHC (lindane)	II	Moderate	Likely low
	Atrazine	I	Low	Likely low
	Chlordane	II	Moderate	Very low
	Diazinon	I	Low	Likely low
Other Organics	VOCs	I	Low	Very high
	1,3-dichlorobenzene	III	High	High
	Benzo(a)anthracene	II	Moderate	Very low
	Bis (2-ethylhexyl) phthalate	II	Moderate	Likely low
	Fluoranthene	II	High	High
	Naphthalene	II/III	Low	Moderate
	Phenanthrene	II	Moderate	Likely low
	Pyrene	II	High	High
Pathogens	Enteroviruses	I	Likely present	Moderate
	Shigella	II/III	Likely present	Moderate
	P. aeruginosa	II/III	Very high	Moderate
	Protozoa	II/III	Likely present	Moderate
Heavy Metals	Cadmium	III	Low	Moderate
	Chromium	III/IV	Moderate	Very low
	Lead	IV	Moderate	Very low
	Zinc	III/IV	High	High
Salts	Chloride	I	Seasonally high	High

NOTE: The mobility of pollutants such as organics and metals should decrease if the soil is clayey and/or has a higher organic content because these soils are more reactive with the passing pollutants compared to sandy soils. The mobility classes are described in Tables 5 and 6 (Modified from Pitt, et al., 1994)

Table 6.5: Organic contaminant mobility classes. (Modified from Armstrong and Llena 1992)

Class	K_d	M_I
I – Mobile	< 0.1 to 1.0	0.1 to 1.0
II – Intermediate mobility	1.0 to 10.0	0.01 to 0.1
III – Low mobility	10.0 to 100.0	0.001 to 0.01
IV – Very low mobility	> 100.0	< 0.001

NOTE: K_d = the soil partitioning (soil/water) coefficient, mL/g

M_I = mobility index (ratio of pollutant's migration velocity to water migration velocity under saturated flow).

Table 6.6: Metal mobility classes. (Modified from Armstrong and Llena, 1992)

Inorganic Pollutant	Mobility Class* for:	
	Sandy Loam	Silt Loam
Arsenic	III and IV	III and IV
Cadmium	III	III and IV
Chromium	III and IV	II and III
Copper	IV	IV
Lead	IV	IV
Nickel	III	III
Zinc	III	III and IV

*Mobility Class Definitions:
I: mobile
II: Intermediate mobility
III: low mobility
IV: very low mobility

Table 6.7 is only appropriate for initial estimates of contamination potential because of the simplifying assumptions made, such as the likely worst case mobility conditions using sandy soils having low organic content. If the soil was clayey and/or had a high organic content, then most of the organic compounds would be less mobile than shown on this table. The abundance and filterable fraction information is generally applicable for warm weather stormwater runoff at residential and commercial area outfalls. The concentrations and detection frequencies would likely be greater for critical source areas (especially vehicle service areas) and critical land uses (especially manufacturing industrial areas), with greater groundwater contamination potential.

6.5.2 Vadose Zone Computer Model for Predicting Groundwater Contamination Potential and Basin Lifespan

Many vadose zone water quality models were developed initially to predict plume migration beneath leaking landfills. They typically focused on the behavior of either organic or inorganic pollutants in the vadose zone. Clark et al. (2006) used one model, SESOIL (Seasonal Soil compartment model), for predictions because it is capable of modeling both organic and inorganic pollutants.

SESOIL calculates mass balances and assumes equilibrium partitioning between phases. It contains three submodels to simulate contaminant fate and transport: the hydrologic cycle, sediment washload cycle, and pollutant fate cycle. The hydrologic cycle models the impact of rainfall, groundwater flow, surface runoff, capillary rise, soil moisture retention, infiltration, and evapotranspiration on the pollutant behavior. Next, the sediment washload cycle estimates the amount of erosion and sediment yield to predict the amount of contaminant removed from the system. Finally, the pollutant fate cycle combines natural attention of the compound with the values from the hydrologic and sediment washload cycles in a mass balance approach to predict the final fate of the contaminant. The natural degradation processes considered by SESOIL include diffusion, volatilization, hydrolysis, adsorption, and biodegradation to its pollutant fate cycle (RISKPRO, 2003).

Table 6.7: Groundwater contamination potential for stormwater pollutants post-treatment.

Compound Class	Compounds	Surface Infiltration and No Pretreatment*	Surface Infiltration with Sedimentation*	Subsurface Injection with Minimal Pretreatment
Nutrients	Nitrates	Low/moderate	Low/moderate	Low/moderate
Pesticides	2,4-D	Low	Low	Low
	γ-BHC (lindane)	Moderate	Low	Moderate
	Atrazine	Low	Low	Low
	Chlordane	Moderate	Low	Moderate
	Diazinon	Low	Low	Low
Other organics	VOCs	Low	Low	Low
	1,3-dichlorobenzene	Low	Low	**High**
	Benzo(a) anthracene	Moderate	Low	Moderate
	Bis (2-ethyl-hexyl) phthalate	Moderate	Low?	Moderate
	Fluoranthene	Moderate	Moderate	**High**
	Naphthalene	Low	Low	Low
	Phenanthrene	Moderate	Low	Moderate
	Pyrene	Moderate	Moderate	**High**
Pathogens	Enteroviruses	**High**	**High**	**High**
	Shigella	Low/moderate	Low/moderate	**High**
	P. aeruginosa	Low/moderate	Low/moderate	**High**
	Protozoa	Low	Low	**High**
Heavy metals	Cadmium	Low	Low	Low
	Chromium	Low/moderate	Low	Moderate
	Lead	Low	Low	Moderate
	Zinc	Low	Low	**High**
Salts	Chloride	**High**	**High**	**High**

NOTE: Overall contamination potential (the combination of the subfactors of mobility, abundance, and filterable fraction) is the critical influencing factor in determining whether to use infiltration at a site. The ranking of these three subfactors in assessing contamination potential depends of the type of treatment planned, if any, prior to infiltration.

* Even for those compounds with low contamination potential from surface infiltration, the depth to the groundwater must be considered if it is shallow (1 m or less in a sandy soil). Infiltration may be appropriate in an area with a shallow groundwater table if maintenance is sufficiently frequent to replace contaminated vadose zone soils. (Modified from Pitt, et al. 1994).

Mikula (2005) performed a full factorial experiment with the model to determine which factors in the model were controlling the calculations and results. The purpose of this activity was to provide guidance as to where to focus data gathering and which parameters could be obtained from the literature. The literature identified three main soil properties that affect infiltration and pollutant removal: intrinsic permeability, organic content, and pH. Other factors beyond the soil itself also affect pollutant movement and groundwater contamination potential, including pollutant concentration, rainfall, and vadose zone thickness. Zinc and sodium chloride were chosen as representative pollutants of interest because of their prevalence in stormwater, solubility, and differing migration rates.

Results indicated that the rainfall amount was a common factor controlling zinc, sodium, and chloride migration. Since the zinc and sodium chloride were assumed to be dissolved in infiltrating stormwater, higher rainfall amounts would naturally allow the pollutants to migrate deeper in the vadose zone. In addition, concentrations were influential in Zn^{2+} migration. Higher concentrations of Zn^{2+} reduced the zinc sorption in any soil layer because the available sorption sites in the upper layers were filled before the zinc in the stormwater was completely removed. Therefore, while this was a simulation, it indicated that more effort should be spent on characterizing the rainfall amounts and runoff concentrations and less effort should be spent on determining partitioning coefficients. This research used in-situ soil information for calculations. It did not address the substantial parameter differences seen in infiltration basins using engineered soils. In addition, as with most chemical computer models readily available, equilibrium was assumed and non-neutral hydrolysis could not be evaluated because no coefficients were available for the model. Therefore, soil pH was not a recognized significant factor in zinc migration because non-neutral-pH hydrolysis could not be modeled. In reality, zinc should be more mobile in acidic and alkaline soils because of its higher solubility during extreme pH conditions, although zinc is more soluble and more mobile in alkaline conditions only if organic matter is present (Shuman, 1999). For the narrow pH range of typical soils, pH may not be a significant factor, although this requires further investigation. Agricultural research on zinc solubility shows that zinc's solubility increases at decreasing pH and can be high comparatively in the pHs of typical acidic soils (pH 4 – 5). The effect of pH on solubility versus retention of the metal in the soil profile will depend on the solubility profiles of each metal as a function of pH and the presence of other competing and/or chelating ions. Organic matter also was not a significant controlling factor, even though prior filtration work (Clark and Pitt, 1999; Clark, 2000; Johnson et al., 2003, which has an extensive literature review) has shown that organic matter affects zinc migration. This is likely due to the lack of organic matter in the soils below the top layer. Organic matter in the B horizon of native soils often is as low as 0.5 – 3%.

6.5.3 Predicting Lifespan Based on Clogging and Soil Chemistry Changes

The following section addresses three issues of concern regarding the significance of clogging from the capture of sediment on the bioretention facility device, cation exchange capacity (CEC) consumption (a common measure of dissolved pollutant retention to protect groundwater), and sodium adsorption ratio (SAR) (to identify adverse infiltration rate effects due to destabilized clay fractions in the soil). The steps for these analyses outlined here are being integrated into the Source Loading and Management Model (WinSLAMM) (Pitt, 1997; Pitt and Voorhees, 2002), and were previously presented by Pitt, et al. (2007 and 2008).

The following tables summarize typical data for different soils, amendments, and plants that may be used in bioretention facilities. Table 6.8 shows the following characteristics, compiled from numerous stormwater treatability and urban soil projects (Clark and Pitt, 1999; Pitt, et al., 1999; Pitt and Lantrip, 2000; Pitt, et al., 2003; Johnson, et al., 2003). The characteristics of the soil that are critical in these evaluations include the following:

- Infiltration rate of the soil (the rate that water can enter the soil surface, with vegetation; soil assumed to be slightly compacted); in colder climates, this rate varies by season (Emerson and Traver 2008),
- Saturated hydraulic conductivity (water movement rate under completely saturated conditions, assuming a unit hydraulic gradient) at the temperature of interest,
- Cation Exchange Capacity (CEC) (calculated capacity of the soil to remove and retain cations, usually expressed in milliequivalents/100 grams or cmol/kg, which have the same numeric values),
- Dry density of the soil (assumed to be slightly compacted)

Table 6.8: Typical soil properties for different soil textures and organic amendments (Clark and Pitt 1999; Pitt, et al. 1999; Pitt and Lantrip 2000; Pitt, et al. 2003; Johnson, et al., 2003)

Soil Texture	Infiltration Rate cm/hr (in/hr)[1]	Saturated Hydraulic Conductivity cm/hr (in/hr)	CEC (cmolc/kg or meq/100 g)	Dry density (grams/cm^3)[1]
Coarse Sand and Gravel	101.6(40)	1270(500)	1	1.6
Sands	33.0(13)	127(50)	2.5	1.6
Loamy Sands	6.4(2.5)	5	5	1.6
Sandy Loams	1	12.7(2.5)	8	1.6
Fine Sandy Loams	1.3(0.5)	1.3(0.5)	10	1.6
Loams and Silt Loams	0.38(0.15)	0.013(0.005)	12	1.6
Clay Loams and Silty Clay Loams	0.25(0.1)	0.03(0.01)	20	1.6
Silty Clays and Clays	0.13(0.05)	0.038(0.015)	30	1.6
Peat as amendment	7.6(3)	7.6(3)	300	0.15
Compost as amendment	7.6(3)	7.6(3)	15	0.25

[1] assumed to be slightly compacted

The infiltration rates and saturated hydraulic conductivity for the mixtures should be based on the amount of amendment added (if 10% or greater), as shown in Table 6.9. If the infiltration system recharges groundwater does not have an underdrain system, the local soils below and to the side of the device also should be evaluated to determine their hydraulic conductivities. The native soil may be the limiting factor in the rates of recharge and, if so, may encourage greater water retention in the system than planned based only on the engineered soil's characteristics. Table 6.10 is an example showing initial estimated properties for soil/amendment mixtures that can be used in bioretention facilities. The sections following Table 6.10 describe further the processes that occur in the devices that reduce infiltration rate and available CEC over time.

Table 6.9: Infiltration Rates and Saturated Hydraulic Conductivities for Amendment Mixtures (Pitt, et al. 1999)

Organic amendment (% of total mixture)	Infiltration rate cm/hr (in/hr), assumed to be slightly compacted	Saturated hydraulic conductivity (in/hr) (same as infiltration rate)
10	2.54(1)	1
33	5.33(2.1)	2.1
50	6.35(2.5)	2.5
100	7.62(3.0)	3.0

Table 6.10: Example mixture calculations

Soil Texture	Infiltration Rate cm/hr (in/hr), assumed slightly compacted	Saturated Hydraulic Conductivity cm/hr (in/hr)	CEC (cmol/kg or meq/100 g)	Dry density (grams/cm^3), assumed slightly compacted
33% compost amendment to loam	5.33(2.1)	5.33(2.1)	13	1.15
10% compost amendment to loam	2.54(1)	2.54(1)	12	1.47
50% peat amendment to sand	6.35(2.5)	6.35(2.5)	150	0.88

The following paragraphs describe the calculations that can be used to estimate the approximate clogging conditions, CEC capacity of infiltration media, and potential SAR problems associated with the water and soil chemistry. They also describe the anticipated changes in these parameters over the life of the device and how those changes can be expected to affect device performance. These calculations are intended only to indicate the relative magnitude of the problem associated with these major issues in the operation of an infiltration device and should be confirmed by initial laboratory tests, followed by actual long-term field monitoring of similar facilities in the area.

Clogging Calculations

The service life and maintenance problems of infiltration facilities can be roughly estimated using the predicted annual discharge of suspended solids to the device and the media characteristics. Infiltration and bioretention devices may start to show significantly reduced infiltration capacities after about 5 to 25 kg/m^2 (1 to 5 lb/ft^2) of particulate solids have been loaded (Clark 1996 and 2000; Urbonas 1999). Deeply-rooted vegetation and a healthy soil structure can extend the actual life much longer. However, abuse (especially compaction and excessive siltation) can significantly reduce the life of the system. The Urbonas (1999) model was applied to organic soil media, including the effects of compaction, media surface area, influent concentration, and influent particle size (Clark and Pitt 2009b). The clogging calculations are illustrated in two case studies in Clark (2000) and Pitt et al. (2008).

Most agencies specify a minimum area for infiltration facilities to account for unforeseen problems. These sizing criteria need to be followed, as a minimum. If the estimated life-time of the facility is only a few years, then a serious problem may occur due to premature clogging. However, if the estimated life is estimated to be for a decade or longer, and especially if planted with appropriate vegetation that can help incorporates the imported sediment into the device soil matrix, the service life may actually be much longer. This calculation may help determine if a proposed design would be subject to rapid failure, or if it is of a more sustainable design. As noted above, actual laboratory and field measurements should be conducted for the area to help interpret these calculated service life estimates for local conditions.

Cation Exchange Capacity

Much of the groundwater protection offered by soils is associated with its' cation-exchange capacity (Pitt, et al., 1996; Johnson et al., 2003). The cation-exchange capacity (CEC) of a material is defined as the sum of the exchangeable cations that can be adsorbed at a given pH. Alternatively, the CEC can be calculated as the measure of the negative charges present at the sorbent surface. The CEC is generally measured to evaluate the ability of certain soil to sorb phosphorus (from fertilizers), heavy metals, and various other target cations of concern for groundwater contamination. The CEC is a function of available surface charge per unit area of material, the pH at which exchange occurs, and the relative affinities of the ions to be exchanged for the material surface. It is not readily transferable to other pHs and it changes over time both due to the exhaustion of sorption sites and the weathering of the soil which alters the soil chemistry. Sands have low CEC values, typically ranging from about 1 to 3 meq/100g of material. Comparatively, clays have relatively high

CEC values that may approach 30 meq/100 g. As the organic content of the soil increases, so does its CEC content. Natural soils can therefore vary widely in their CEC content, depending on their components. Organic soil amendments, such as compost, greatly increase the CEC of a soil that is naturally low in organic material or clays. However, this increase is temporary. Weathering processes, such as the microbial degradation of the soil and flushing of charged ions through the soil profile during infiltration, eventually will reduce the organic content to a steady-state value. For green roofs, this value has been reported as between 2 and 5% by weight (Dr. Robert Berghage, Penn State Center for Green Roof Research, personal communication). The time to reach steady state and the CEC value at steady state is a function both of the amount and rate of degradation of organic matter (which is based on the type of organic matter), the rate of exhaustion of sorption sites, and the replenishment rate of organic and inorganic matter (such as clay) that contribute to CEC.

The total cation content of a water sample can be directly calculated knowing the major ion content of the water and the associated equivalent weights, as shown in Equation 1 and Table 6.11. The sum of the cations must equal the sum of the anions (expressed in equivalent weight). Table 6.11 is an example calculation for a typical stormwater.

$$Cation\ Content\ for\ Na^+ = \frac{Concentration\ in\ mg/L}{Molecular\ Weight\ of\ Na} \qquad (1)$$

$$Cation\ Content = \frac{3.9\frac{mg}{L}}{23\frac{g}{mol\left(1{,}000\frac{mg}{g}\right)}}$$

Since sodium has a +1 valence charge, its equivalent weight is equal to its molar weight (rephrased, it has 1 equivalent per mole).

$$Cation\ Content\left(\frac{meq}{L}\right) = 0.0001696\ mol/L(1\frac{eq}{mol})(1{,}000\frac{meq}{eq}) = 0.17 meq/L$$

The above example only lists the major ions in the water. However, the concentrations of the dissolved heavy metals in stormwater are rarely more than about 0.10 mg/L and therefore contribute little to the total cation content of the water. The total (unfiltered) heavy metal concentrations of some metals can be much higher, but only the ionic forms affect the CEC. The total hardness of the above sample (the sum of the divalent cations Ca^{2+} and Mg^{2+}) is 0.94 meq/L, and with an equivalent weight of 50 meq/L per mg/L as $CaCO_3$ (based on a molecular weight of 100 g/mol for $CaCO_3$ and a valence charge of 2 [2 eq/mol], the resulting hardness concentration is about 47 mg/L.

Table 6.11: Example total cation and anion content calculation for stormwater (Johnson, et al., 2003)

Component	mg/L	Equivalent weight	meq/L
Ca^{2+}	13.3	20.0	0.67
Mg^{2+}	3.3	12.2	0.27
Na^{+}	3.9	23.0	0.17
K^{+}	2.3	39.1	0.06
		Total cations:	**1.17**
HCO_3^{3-}	36.7	61.0	0.60
SO_4^{2-}	22.4	48.0	0.47
Cl^{-}	3.7	35.5	0.10
		Total anions:	**1.17**

The maximum consumption of the CEC in the soil can be calculated by dividing the soil total CEC by the total cation content of the water. If the soil is ½ meter thick and the infiltration device has a surface area of 1 m^2, and the soil density is about 1.5 grams/cc, the total CEC of a soil having a CEC of 10 meq/100 grams, per m^2, is approximately 75,000 meq.

$$0.5\ m\ soil\left(1.5\frac{g}{cm^3}\right)\left(100\frac{cm}{m}\right)^3 = 750{,}000\ g$$

$$750{,}000\ g(10\frac{meq}{100g}) = 75{,}000\ meq$$

If the stormwater has a total cation content of about 1.2 meq/L, then the total maximum water treatment capacity of the soil, per square meter, is about 64,000 L, or a column of water about 64 m (210 ft) high.

$$\frac{75{,}000\ meq}{1.2\ meq/L} = 64{,}000\ L = 64\ m^3 = 64\ m\ of\ water\ per\ square\ meter\ surface\ area$$

If the soil is only receiving rain water (having this cation content), and 1 m (3.3 ft) of rain falls per year, then the CEC content of the soil would be exhausted in several decades (calculated to be about 60 to 70 years, assuming limited weathering. However, but from a design standpoint, it would be safer to assume a shorter life and use a fraction of the initial CEC as the design life CEC). The natural soil building process, and accumulating layers of organic material, would continue to "recharge" the soil CEC in an undeveloped setting, with very slow changes in the soil CEC with time. In an urban infiltration device, where substantial amounts of runoff are directed to the device, the CEC of a soil likely would be depleted much faster than recharge could occur due both to sorption site exhaustion and accelerated weathering (washing out of the soil into the passing water the polar molecules that provide sorption sites in the media).

Sodium Adsorption Ratio (SAR)

The sodium adsorption ratio can affect radically the performance of an infiltration device that contains even relatively small amounts of clay in the soil-media mixture. Clays in soils with an excess of sodium ions, compared to calcium and magnesium ions, remain in a dispersed condition, and are almost impermeable to rain or applied water. A "dispersed" clayey soil is extremely sticky when wet, tends to crust, and becomes very hard and cloddy when dry. Water infiltration is therefore severely restricted. Dispersion caused by sodium may result in poor physical soil conditions and water and air do not readily move through the soil. An SAR value of 15, or greater, indicates that an excess of sodium will be adsorbed by the soil clay particles. SAR values near 5 also can cause problems, depending on the type of clay present. Montmorillonite, vermiculite, illite and mica-derived clays are more sensitive to sodium than other clays. Obviously, larger amounts of clay in the mixture cause more dramatic problems, but normal amounts of clay in soil mixtures can cause degraded infiltration. The NRC reported recently (Suarez, et al. 2006) that SAR values above 4 caused decreased infiltration rates in a clay soil, while SAR levels starting above 2 degraded infiltration in a loam soil. Additions of gypsum (calcium sulfate) to the soil can be used to prevent the exchange of sodium onto the soil and allow it to pass freely through the soil column.

The SAR is calculated by using the soil concentrations of sodium, calcium, and magnesium (in meq) in the following formula:

$$SAR = \frac{Na^{+}}{\sqrt{\frac{(Ca^{+2} + Mg^{+2})}{2}}} \qquad (2)$$

Even when soils have SAR values well below the critical values, if the runoff water contains high levels of sodium in relationship to calcium and magnesium (such as snowmelt in areas using salt for de-icing control), an SAR problem may occur in the future, necessitating the addition of gypsum to the infiltration area. Again, these calculations indicate the direction and relative severity of the potential problem. If the runoff has a higher SAR than the soil, SAR problems may eventually develop. Seasonal changes in runoff sodium and leaching of sodium from the soil, results in variable conditions throughout the year. In some coastal areas, upwelling brackish groundwater may also affect an infiltration device's SAR status.

6.5.4 Screening Engineered Soils for Pollutant Removal

As the case studies illustrate, natural soils can provide treatment of stormwater runoff. These studies also illustrate that increasing certain soil properties, such as hydraulic conductivity and sorption/ion-exchange potential, have improved pollutant treatment. The data that is missing from these case studies is determining the point where increasing the hydraulic conductivity results in reduced contact time to the point where minimal treatment occurs. New media, and media combinations, are proposed regularly for installation in stormwater infiltration devices. However, many of these media are not screened properly prior to use, resulting in decisions regarding lifespan and maintenance intervals that are based on incomplete information. This section addresses the concerns and the steps required to evaluate properly infiltration media.

Properties that Affect Media Performance

Prior filter and soil amendment media tests have shown that the hydraulic flow rate and particulate solids loadings on the surface greatly affect the performance of the system. A medium with high filtration rates does not provide adequate contact time, thus resulting in reduced pollutant removals. However, a medium that has a very low infiltration rate creates extended surface ponding or requires large surface areas or deep freeboard space above the media. Therefore, a balance between hydraulic capacity/contact time and pollutant removal is critical for a successful system. In addition, clogging of the surface, or pores, of the media often causes premature failure of the system. Particulate solids (and their particle size distribution) are critical parameters in determining the longevity of a filter run and how the flow rate decreases with time and use. Other fouling constituents (such as oil and grease) also affect filter performance and need to be evaluated. These constituents need to be monitored as part of a media evaluation study, in order to supplement the data gathered on the behavior of the targeted pollutants. These other issues that affect media removal of stormwater pollutants include:

- Water salinity. Salts can strip accumulated heavy metals from sorption sites, for example.
- Oxidation-reduction potential (ORP) and dissolved oxygen (DO) concentrations of the potential stormwater. If a treatment system goes anaerobic between events, many pollutants, especially nutrients and some heavy metals, can be released from the media.
- pH. ORP in conjunction with pH determines the electro-chemical state of the media and the specific speciation of heavy metals, which in turn affects their removal and retention.
- Cation exchange capacity (CEC) and water concentrations of major cations and certain metals. Major cations, such as K, Ca, Mg, and Na, and metals in high concentration in the earth's crust, such as Al and Fe, can be washed into an infiltration device and can consume the cation exchange capacity of the media that was assumed to be available for targeted pollutant removal.
- Sodium adsorption ratio (SAR). Adverse SAR conditions, caused by an abundance of Na in contrast to Mg and Ca, destabilizes the surface charges of clays in the soil-media mixture, causing premature clogging of the system.

Other Constituents that Assist in Understanding Media Removal Mechanisms

In addition, additional constituents should/must be monitored when evaluating media in order to understand the removal mechanisms, and how the media may change if the stormwater characteristics change. Some of these additional issues include the following:

- Exchangeable cations and anions. Major cations and anions need to be monitored to evaluate which are being exchanged as targeted pollutants are being removed. As noted above, if major ions are being removed by the media, the space available for targeted pollutant removal is reduced. The targeted pollutants, if they are loosely bound through ion-exchange, also may be lost if an ion is preferentially attracted to the same site in the media.

- Organic compounds. The presence of organic compounds adds to the CEC of the mixture and also provides a basis for organometallic compounds to form which are very stable. Soil organic material, though, degrades over time, resulting in lower CEC and reduction in volume of bioretention soil, which may lead to preferential flow path development. In addition, organic toxicants may be present in the runoff and their behavior in the media needs to be considered. They may foul the media, reducing both water passage and pollutant removal.

- Metallic compounds. Total metal removal typically is of interest to treatment professionals and regulators. However, it is the filtered forms of the metals that are usually of most interest considering harmful effects on humans and wildlife. The filtered forms also are those exchanged by ion-exchange processes in the media.

- Bacteria. Stormwater indicator bacteria are normally critical contaminants in most areas, but the presence and retention of bacteria should be evaluated in media used for capturing stormwater pollutants as increased indicator bacteria in the media are a good indicator of beneficial bacteria that are needed to assist in the degradation of captured organic material. In addition, bacterial activities in media may aid in pollutant removal through several mechanisms, including creation of additional sorption capacity, creation of exopolymers that attract positively-charged pollutants, etc. A brief summary of bacterial removal mechanisms is given in Clark (2000) and Clark et al. (2006).

- Nutrients. Nitrogen and phosphorus compounds are also often critical stormwater constituents and their retention in media filters can also indicate replenishment of plant nutrients needed to support plant growth.

Tests Required for Complete Media Evaluation

Four tests are recommended for evaluating potential stormwater infiltration media, including (1) clogging, breakthrough, and removal tests; (2) contact time/media depth tests; (3) batch capacity and kinetics tests; and (4) pollutant retention under aerobic versus anaerobic conditions. These tests are summarized below. Full descriptions of the testing protocols for the data presented in this chapter can be found in Johnson et al. (2003), Clark and Pitt (2009a,b), Clark and Pitt (1999), and Clark (2000). These tests have not been standardized through traditional testing organizations. Therefore, the reader is directed to the references above, where the literature used to support the testing protocols are described. A review of the other references provided in this chapter will identify other testing protocols and the benefits and limitations of those methods.

Clogging, Breakthrough, and Removal Tests

The purpose of these tests is to simultaneously determine several important stormwater filter media characteristics, including pollutant removal and effluent quality, treatment flow rates, and losses in both pollutant removal and flow rate performance associated with clogging and breakthrough as the media becomes exhausted. Evaluation of the media under the conditions most representing the actual field conditions is important in determining the appropriateness of stormwater filtration/bioretention media. Prior testing has shown that stormwater, with its intermittent flows and relatively low concentrations (compared to industrial wastewater where traditional media evaluation has been performed), is not as easily treated as industrial wastewater. The low concentrations reduce the concentration gradient for sorption/ion-exchange and the intermittent flows often result in variable pore water chemistries that affect both the quality of the first flush from the filter in a new storm event and the potential availability of removal sites in the media. Therefore, application of high-concentration, continuous-flow testing results to predicting the life of stormwater filters rarely has been successful. Laboratory testing, both batch and continuous flow, has been most successful when the test solution and concentrations most resemble the actual runoff. Ideally, these tests would be conducted in the field or in pilot-scale outdoor plots using runoff from a specific site. Field data will best reflect what could be expected in future installations; however, the laboratory testing can provide general information about performance and certainly is effective when the goal is to compare the behavior of several media.

Contact Time - Media Depth Tests

A significant factor affecting stormwater pollutant removal with media filtration is treatment flow rate and the associated contact time. The treatment flow rate can be adjusted by modifying the media with other materials, such as sand. The use of different sand sizes and amounts has been used in the past and in current installations to modify the treatment flow rates to desired conditions. Contact time is also affected by the depth of the media in the column with deeper media, obviously, having greater contact times.

Media Capacity Tests

These traditional isotherm and kinetics batch tests are used to determine the amount of contaminant that can be retained by the media and the rate at which the pollutant is retained. Contact time should be based on the kinetics of the pollutant removal. While traditional batch testing often is only a moderate predictor of performance in stormwater filter columns, knowledge of the parameters allows comparisons of these media to traditional filter media. Also, as modeling improves, many modelers are relating traditional parameters to actual field performance. The results are applied in situations where the desired effluent concentration and given influent concentrations, as well as the flow rates, are known, and the mass of pollutant requiring removal can be calculated as the difference. The amount of media required for pollutant removal then is predicted based on the desired effluent concentration and the amount of pollutant requiring removal.

Aerobic and Anaerobic Effects on Contaminant Retention in Media

These tests are used to determine whether pollutant retention is permanent in filter media when the chemical conditions inside the filter change. While most stormwater, and therefore its filtration, occurs under aerobic (> 1 mg/L DO) conditions, stormwater filters only treat flows periodically. Clark (2000) showed that filters do not dry out completely between storm events, with water retention occurring for several months after active flow events. Drying only occurs within the first 1 – 2 inches of the media exposed to the atmosphere (if free draining, drying may occur at both the surface and at the interface with the drainage layer). Therefore, these wet/damp areas have the potential to develop anaerobic conditions in parts of the media. Because anaerobic conditions often are associated with reducing environments and the prevalence of anaerobic/anoxic bacteria, pollutant retention may be changed. Clark and Pitt (2009a) showed in a series of preliminary tests that permanent retention of nutrients and metals such as iron that are not strongly bound to the media was a concern. Organic media were a particularly concern for pollutant retention.

6.6 CONCLUSIONS

The natural hydrologic cycle incorporates a component on groundwater recharge through stormwater runoff infiltration. Urbanization has disturbed that cycle by reducing the amount of space available for recharge and, in some cases, increasing the pollutant loadings to areas where infiltration does occur. The case studies described above, and those summarized in publications such as Pitt et al. (1996) and Clark et al. (2006), indicate that stormwater infiltration can affect the chemical quality of groundwater with a few instances of contamination above the water-quality standards. Given the number of infiltration sites (both planned and natural), the percentage of groundwater contaminated to an unhealthy level due to stormwater is low. Pollutants of concern for transport and potential groundwater contamination primarily include nutrients and salts, and, in particular, nitrogen compounds and chlorides. Metals, bacteria, and organic compounds typically are readily-removed in infiltration areas where the seasonal high water table is not shallow (0.9 meter or 3 feet of unsaturated zone between the device bottom and the seasonal high water table). Soil accumulation is a concern, but if the stormwater is not heavily polluted, soil accumulation may not be sufficient to affect plant growth in biofiltration facilities.

Infiltration is becoming increasingly popular as a stormwater management technique because it promotes groundwater recharge, while reducing the damage caused by excessive surface flows and while removing pollutants from the runoff. Therefore, it is imperative that improvements to siting guidance and to predicting the lifespan of these devices occur. The field case studies summarized here indicate that organic content, clay fraction and a low phosphorus index resulted in improved treatment of nutrients and metals. The laboratory studies indicated that there was an optimal amount of organic matter that should be included in an engineered infiltration media. Below that amount, pollutant removal is not optimized and above which nutrient leaching occurs. Also, the laboratory studies indicated that infiltration in natural soils (soils which have not been amended with organic matter and have not been disturbed) can be effective. The lower soil horizons, which are typically higher in mineral content, do provide removal and were able to capture the phosphorus that was leached from the organic layer.

Two levels of modeling have been presented. The simplified model, which uses tables relating pollutant abundance, pollutant mobility, and the ability to pretreat the pollutant to predict whether contamination is a concern, provides a preliminary evaluation by general soil type of a site for stormwater infiltration. The computer model can provide a more-detailed evaluation of whether a site is suitable and if suitable, the life of the site for stormwater infiltration. However, the computer models are limited by the lack of data on appropriate modeling coefficients. Therefore, without substantial data collection, the computer model may not be an improvement over the simplified model. One limitation of the model that has not been addressed in either the model setup or in the laboratory is the effect of changing soil properties over time. This limitation is being addressed to a limited extent by Treese et al. (2008) on the loamy sand and silt loam soils. None of the research in the stormwater field addresses the impact of temporal variabilities in CEC, soil pH, and sodium adsorption ratio (SAR) on pollutant removals.

This chapter summarizes the current level of knowledge regarding stormwater infiltration and potential groundwater contamination. Predictive models were presented. However, these models are limited by the lack of supporting data and the lack of fundamental research into the processes controlling stormwater treatment in the vadose zone. Until those deficiencies are addressed, the models should be seen as providing screening guidance and not as providing absolute answers regarding site suitability and infiltration device lifespan. Predictions/calculations regarding three areas of concern for the lifespan and maintenance of stormwater infiltration devices, clogging, cation exchange capacity, and sodium adsorption ratio, also were presented. Finally, a method for complete evaluation of potential engineered soils for infiltration was presented. As this chapter shows, much is being learned about infiltration devices; however, further research is needed to address several substantial data gaps.

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CHAPTER 7

Mitigation of Urban Stormwater and Polluted River Water Impacts on Water Quality with Riverbank Filtration

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ABSTRACT: Riverbank filtration (RBF) is a relatively low-cost, natural water treatment technology in which surface water contaminants are removed or degraded as water infiltrates from a surface source (e.g., river or lake) to abstraction wells. The proportion of water from the production well that originates from river infiltration depends on the nature of the groundwater-surface water interactions in the immediate vicinity of the well. Climate, lithology, vegetation, and land use all impact the physical, chemical, and microbiological quality of surface waters that serve as source water for RBF processes. Various biotic and abiotic attenuation processes such as adsorption, precipitation/dissolution, oxidation/reduction, hydrolysis, biodegradation, physico-chemical filtration, straining, and mixing with groundwater change the physical, chemical, and microbiological quality of the surface water during RBF. These attenuation processes occur within two distinct zones: the biologically active colmation layer, where intensive degradation and adsorption processes occur within a short residence time, and along the main flowpath between the river and abstraction well where degradation rates and sorption capacities are lower, mixing processes greater, and physico-chemical filtration and straining of small particles and microorganisms continue. Although biotic attenuation processes are often considered more significant (for mass reduction) than abiotic processes in most groundwater environments (particularly for organic pollutants), the relative importance of the two attenuation zones and the associated performance of the various attenuation processes

within these zones must be weighed according to the purpose for utilizing RBF. As pollutant loads to drinking water resources increase, RBF will continue to receive increased attention and application for drinking water treatment purposes; although RBF can mitigate many of the detrimental effects of urban stormwater and river pollution, its capacity for water supply diminishes where pollution levels result in decreased permeability and clogged river beds and banks.

7.1 INTRODUCTION

As the global population grows and the demand for high quality drinking water increases, and the need to deal with the existing deficit in providing improved water supply to more than 1 billion people becomes even more urgent (Marsalek et al., 2008), water managers are also challenged by increased competition over existing water supplies, primarily due to rapidly increasing demand from agricultural use and industrial and municipal development. Global climate change projections and their likely influence on the hydrologic cycle, suggest that significant changes in the amount and quality of future water supplies can also be expected (Bates et al., 2008). Because global warming is expected to cause the types of weather extremes (i.e. increased number and intensity of droughts followed by heavy rain) that can substantially increase pollutant loads to drinking water resources (Kaushal et al., 2008), the management of water resources requires vigilant attention by utilities and continues to make the search for low-cost technologies for effective drinking water treatment increasingly challenging. Riverbank filtration (RBF) is a relatively low-cost, natural water treatment technology in which surface water contaminants are removed or degraded as the infiltrating water flows from a surface water source such as a river or lake to abstraction wells. The use of RBF for the purposes of water treatment has long been recognized as effective in Europe and utilized for at least 200 years (Kühn and Mueller, 2000; Ray et al., 2002b). RBF has demonstrated the ability to remove or degrade a wide range of dissolved and suspended waterborne contaminants, including natural organic matter and disinfection by-product precursors, organic contaminants (including some micropollutants such as pharmaceuticals, chelating agents, aromatic sulfonates, aliphatic amines etc.), inorganic contaminants such as metals, and pathogens (Kühn and Mueller, 2000; Ray et al., 2002a,b; Schubert et al., 2002a,b; Tufenkji et al., 2002; Wang et al., 2002). RBF can also mitigate chemical shock loads that may result from emergency spills or spring flooding in agricultural watersheds (Ray et al., 2002c; Ray, 2004). The physical, chemical, and microbiological quality of RBF filtrate depends upon numerous factors, including source water quality (which is influenced by land use and climate) and quantity, river flow velocity and bed load characteristics, seasonality of river flow, stability and management (e.g. dredging) of the river channel, local geologic settings, distance of the wells from the surface water source, well pumping rates, the characteristics of sediments at the river aquifer interface, and ultimately the physical, chemical, and microbiological processes acting in the aquifer (Schijven et al., 2002; Hunt et al., 2002). Accordingly, as pollutant loads to drinking water resources increase in response to land use change and increasing demand from agricultural use and industrial and municipal development, RBF will continue to receive increased attention and application for drinking water treatment purposes. This will necessitate improved understanding of the transport and fate of pathogenic microorganisms and key chemical contaminants in the subsurface environment so that reasonable design strategies and regulations can be developed and predictable and sustainable water treatment performance can be achieved. The overall goal of this chapter is to provide a review of the RBF process of drinking water treatment and its efficacy at mitigating surface water pollution arising from urban stormwater runoff and other sources of river pollution.

7.2 TREATMENT MECHANISMS AND OPERATIONAL LIMITATIONS

7.2.1 Hydrologic and Hydraulic Considerations

Unlike conventional granular media filtration or alternative pressure-driven membrane technologies that typically require source water with relatively low solids content or chemically-assisted solids reduction prior to use, RBF utilizes groundwater derived from infiltrating surface water by situating production wells near rivers and exploiting river water infiltration through river beds and/or banks. Hiscock and Grischek (2002) presented typical flow conditions associated with different types of RBF schemes (Figure 1) and emphasized that RBF can occur under natural conditions or be induced by lowering the groundwater table below the surface water level by abstraction from adjacent pumping wells. As illustrated in Figure 1, a riverbank filtration well can capture surface water infiltrating from an adjacent river (bank filtrate) under a variety of different conditions. The proportion of extracted water in the production well that originates from river infiltration depends on the nature of the groundwater-surface water interactions in the immediate vicinity of the well. If the river is receiving groundwater discharge under natural conditions, the downward hydraulic gradients created by pumping may induce infiltration over a portion of the riverbed as shown in Types 1, 3, 5 and 6 in Figure 7.1. Rivers that are already exfiltrating prior to pumping may contribute their entire exfiltration flux to the riverbank filtration well as shown in Type 2. Under either of these different scenarios, groundwater from the opposite side of the river may be drawn below the river and be captured by the well as an additional component of the total production volume of the riverbank filtration well (Types 3, 4 and 6). The river may be constantly or intermittently isolated from the underlying groundwater flow system as a result of perching due to reduced sediment permeability in the streambed. Under this set of conditions, the infiltrating water from the river that migrates through the unsaturated zone may be ultimately captured by the riverbank filtration well (Type 4). Finally, considerable effort has been dedicated to enhancing the capture of the bank filtrate under a variety of different conditions through the installation of lateral galleries and collection drains that are connected to the main production well. The geometric configuration of the collection drains depends on the nature of the subsurface geologic material and the groundwater flow system. An example is provided as Type 6 in Figure 7.1.

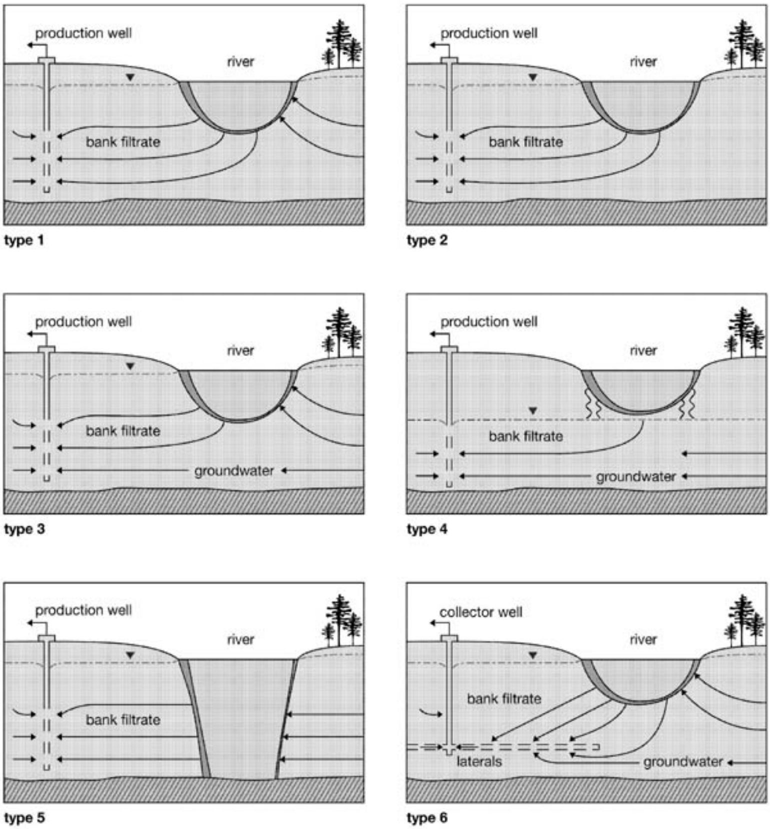

Figure 7.1: Schematic representation of types of flow conditions at bank filtration sites. In the cases illustrated above, the production well can induce conditions of downward infiltration from the river across part (Types 1, 3 and 6) or all (Type 2) of the riverbed in the vicinity of the well. Under some conditions, groundwater from the opposite side of the river may flow naturally or be induced to flow beneath the river subsequently being captured by the production well along with the bank filtrate component (Type 3, 4, and 6). The river channel may also cut through the entire aquifer unit cutting off any direct lateral underflow (Type 5). If the river is perched above the local water table, the bank filtrate will consist of the infiltration flux from the river through the underlying unsaturated zone (Type 4). Finally, lateral collector drains can be installed in a

variety of orientations to enhance the capture of the bank filtrate (Type 6) (Modified from Hiscock and Grischek, 2002).

Details regarding conceptual design of RBF wells and suitability of well designs are provided by Hunt et al. (2002). In general, RBF systems are typically constructed in alluvial aquifers consisting of a variety of deposits in low permeability zones (e.g. clay and silt) and, more commonly sand, to gravel, cobbles, and boulders. In most alluvial valley aquifers, sand and gravel predominate, but floodplain deposits also leave layers of silts and clays in the stratigraphy (Rosenshein, 1988). The size, uniformity, and distribution of aquifer sediments are dependent on the type and source of the original rock and on the degree of glacial or fluvial processes (Gollnitz et al., 1997). Both permeability and filtration efficiency are impacted by grain size and distribution of sediments. The complex and physically and chemically heterogeneous environments of alluvial aquifers that are hydraulically connected to surface water commonly comprise RBF sites because of the relative ease of shallow groundwater abstraction, their generally high production capacity, and their proximity to demand areas (Doussan et al., 1997). Although proximity to a river can enable higher recharge and pumping rates, water quality problems may be encountered during RBF (Bertin and Bourg, 1994). For effective management of RBF systems, the catchment zones, infiltration zones, mixing proportions of surface and groundwater, flow paths, and flow velocities need to be understood.

7.2.2 Chemical and Microbiological Considerations

The hyporheic zone (the interface between surface water and groundwater) of alluvial aquifers has been identified as a distinct biogeochemical environment characterized by gradients in light, temperature, pH, redox potential, oxygen, and organic carbon concentration (Gilbert et al., 1997; Tufenkji et al., 2002). Electron transfer, weathering, ion exchange, and gas exchange reactions occur as water infiltrates through river beds and/or banks into the subsurface (Jacobs et al., 1998). Chemical water composition (von Gunten et al., 1987; Bourg and Bertin, 1994), ^{222}Rn use as a tracer (Hoehn and von Gunten, 1989; Bertin and Bourg, 1994), and water temperature (Constantz et al., 2002; Anderson, 2005) have been used to describe surface water-groundwater mixing (dilution) in the hyporheic zone. Biologically mediated processes occurring within the hyporheic zone such as sediment bioturbation (reworking) by macroinvertebrates can result in locally increased permeabilities (Boulton, 2000); however, biofilm formation can have the opposite effect (Findlay and Sobczak, 2000). Permeabilities may decrease or increase as a result of chemical processes such as mineral precipitation and dissolution, respectively (Fuller and Harvey, 2000). Soil permeability may decrease as a result of ion exchange, when sodium ions (Na^+) originating from road salting replace Ca^{2+} and Mg^{2+} cations, recognizing that clays with sites primarily occupied by sodium become cohesive and impermeable (Amrhein et al., 1992; Krauskopf, 1995). Physical processes can also reduce riverbed permeabilities. For example, deposition of fine sediment particles in hyporheic interstices may lead to particle straining and may result in particularly low permeability (clogging) in the upper layers of a riverbed (Gilbert, 1997; Brunke, 1999); this behavior may vary temporally with river sediment load and be influenced by changing source areas related to land use changes.

Colmation is the process of deposition of fine-grained sediments (sometimes referred to as surficial-fined grained laminae [SFGL] (Droppo and Stone, 1994)) in the porous sediments of streambeds that typically results from the down-welling stream water and sedimentation (Brunke and Gonser 1997). Investigations of the erosion threshold of the colmation layer have demonstrated the role of small particle clusters (microflocs) in the formation of larger aggregates (flocs) in the water column as a function of shear stress. Obtained using an annular flume, the experimental data conformed to a theoretical model proposed for floc formation (Stone and Krishnappan, 2003). The critical shear stress for erosion and deposition of fine sediment in agricultural streams (that would likely be rich in pathogen load) was quantified and used to model in-stream transport of cohesive sediment in the watersheds (Stone and Krishnappan, 1997; 2002). In related studies, morphological characterization techniques were used to quantify the formational dynamics of flocs in rivers and identify sediment source areas (DeBoer et al., 1999; DeBoer and Stone, 1999). More recently, annular flume experiments demonstrated that suspended particle morphology changed with shear stress and bed age. As bed age increased, less sediment was eroded and particles appeared less porous and more angular in shape for a given shear stress; it was suggested that these changes were related to bed age-associated biostabilization of bed deposits (Stone et al., 2008). A low permeability layer known as colmatage can form under low flow conditions; in the most severe cases, it may hydraulically isolate the river completely from the underlying groundwater (Petts 1988, Schalchli, 1992). Fine particles that penetrate the colmatage and do not contribute to clogging may be transported deeper into the interstitial layers of the alluvial aquifer by depth filtration (Brunke, 1999). Although clogged riverbank sediments may increase the efficiency of natural filtration processes, the loss in permeability can significantly reduce the productivity of the well field. With the exception of streams affected by severe mine-water pollution (in which significant precipitation of minerals and/or metals can occur), colmation appears to have the most significant impact on riverbed permeability (Brunke and Gonser 1997). Thus, the hydraulic conductivity of the river bed is a critical factor for determining the volume of bank filtrate because the permeability of clogged areas varies with the dynamic hydrology and cannot be regarded as constant (Schubert, 2002a,b). Colmation is most common in heavily regulated rivers subject to excessive soil erosion in the surrounding catchment. Because these catchments are frequently also affected by anthropogenic activities that lead to pollution, the colmatage can have beneficial effects as a barrier to pollution across the groundwater–surface water interface (Younger et al. 1993, Brunke and Gonser, 1997). Application of cohesive sediment transport models and annular flume methods to model colmation layer formation/erosion in response to varying environmental conditions may contribute to improved dynamic modeling of pathogen transport into the subsurface. The causes of colmation are summarized in Figure 7.2 (Brunke and Gonser, 1997).

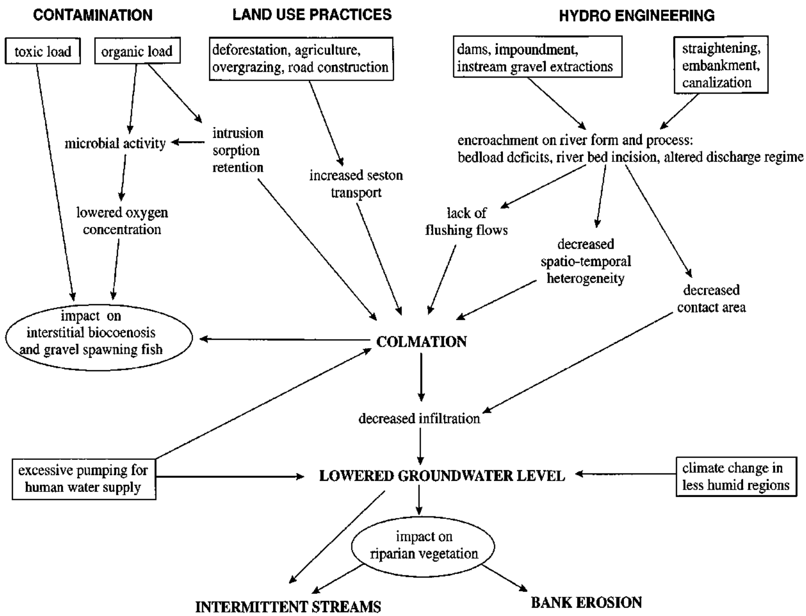

Figure 7.2: Anthropogenically induced impacts that promote colmation in stream bed sediments, and their ecological consequences (Brunke and Gosner, 1997)

The quality of RBF process filtrate ultimately depends on the physical, chemical, and microbiological processes in the aquifer (Schijven et al., 2002; Hunt et al., 2002). As surface water passes through the river bed and/or bank and into the subsurface, various biotic and abiotic attenuation processes such as adsorption, precipitation/dissolution, ion exchange, oxidation / reduction, hydrolysis, biodegradation, physico-chemical filtration, straining, and mixing with groundwater change the physical, chemical, and microbiological quality of the water (Amrhein et al., 1992; Bourg et al., 2002; Tufenkji et al., 2002; Irmscher and Teermann 2002; Hiscock and Grischek 2002) (Figure 7.3). These attenuation processes occur within two distinct zones: the biologically active colmation layer, where intensive degradation and adsorption processes occur within a short residence time, and along the main flowpath between the river and abstraction well where degradation rates and sorption capacities are lower and mixing processes greater (Hiscock and Grischek, 2002); physico-chemical filtration and straining of small particles and microorganisms also continue here. Although biotic attenuation processes are often considered more significant (in terms of mass reduction) than abiotic processes in most groundwater environments (particularly for organic pollutants), the relative importance of the two attenuation zones and the associated performance of the various attenuation processes within these zones must be weighed according to the purpose for utilizing RBF. For example, North American utilities focused primarily on achieving pathogen reductions by RBF may have different operational concerns than European utilities focused on removing organics.

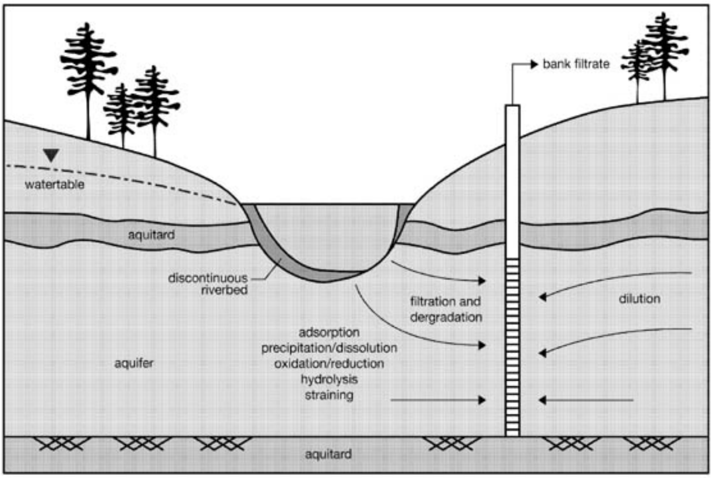

Figure 7.3: Generalized schematic of a riverbank filtration (RBF) system (Côté et al., 2009)

Several investigations have also suggested that microbial activity in the hyporheic zone results in some of the most significant changes in chemical water quality as river water infiltrates through river beds and/or banks (Bourg et al., 1989; Bourg and Bertin, 1993). Biodegradation processes are typically most relevant for organic pollutants, though a range of different reactions may be responsible for their degradation, while sorption reactions are often more important for the retardation of metals and other non-degradable substances. Significant oxygen depletion can occur in riverbed sediments because of intense microbiological activity, resulting in a reduced environment (depleted in O_2 , DOC, NO_3, Na, and K and enriched in Mn, Ca, Mg, HCO_3^-, and silica) in which the stability of mineral surface coatings (Bourg and Bertin, 1993; von Gunten and Zobrist, 1993) such as ferric and manganese oxyhydroxides that impact the physico-chemical filtration of microbial pathogens (Loveland et al., 1996; Ryan et al., 1999) can be affected and the anoxic conditions enable denitrifying and sulfate-reducing bacteria to further decrease the redox potential of the system. Manganese and iron can be mobilized in the reduced zone and may significantly deteriorate water quality; though these metals are not highly toxic, they may contribute to taste and odor problems, staining, and corrosion, and interfere with UV disinfection where it is applied in wastewater treatment (Nessim and Gehr, 2006). Thus, the development of the reduced zone, which may exhibit spatial and temporal variability due to seasonal fluctuations in microbial activity and water pumping patterns (Bourg et al., 1989; Bourg and Bertin, 1994), can have deleterious impacts on bank filtrate quality; despite the effective removal of various contaminants by riverbed sediments acting as filtration media (Tufenkji et al., 2002). Microbial activity ultimately diminishes at some distance due to electron donor deficiency enabling reaeration of the aquifer (Bourg and Bertin, 1993), allowing manganese and iron to be removed from solution by a series of precipitation reactions. The precipitation of iron and manganese oxy-hydroxides as well as calcium carbonates, however, can reduce hydraulic conductivity and permeability (Fuller and Harvey, 2000). Figure 7.4 qualitatively illustrates the evolution of dissolved oxygen, nitrate, dissolved manganese, and dissolved organic carbon in the infiltration flow path during RBF (Tufenkji et al., 2002 adapted from Bertin and Bourg, 1993).

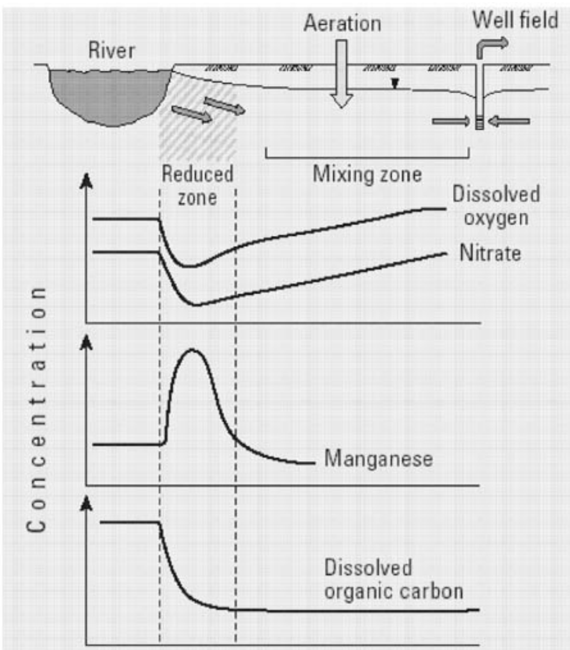

Figure 7.4. Dissolved oxygen, nitrate, dissolved manganese, and dissolved organic carbon in the infiltration flow path during riverbank filtration (Bourg and Bertin (1993) as modified by Tufenkji et al. (2002))

7.2.3 Operational Considerations

Flow pathways between rivers and abstraction wells may vary during RBF because both river and groundwater environments are dynamic and complex hydrologic systems. Rivers are operationally transient in stage, temperature, flow, and water quality. Concerns regarding potential changes in the transport and fate of contaminants arising from the transient characteristics of rivers (e.g. riverbed surface scour changing the nature of the colmation layer or potentially eroding it completely, increased river stage and/or flooding resulting in water infiltration through previously unsaturated regions which may attenuate contaminants differently than the permanent flow pathways, etc.). Schubert (2002a,b) reported that fluctuating river levels impact the permeability of clogged areas as well as water velocities and residence times in the subsurface and, in some cases, the quality of RBF process filtrate. It was also noted that age-stratification of infiltrated river water can act to balance fluctuating contaminant concentrations and temperatures in source water and can also efficiently mitigate the impacts of accidental river water pollution (Schubert, 2002a,b). In addition to changes in river stage, floods also have the potential to erode the colmation layer, destabilize the river bed, and reduce the filter efficiency of the riverbank. Furthermore, extreme floods may cause environmental pollution arising from flooded sewage treatment plants and industrial plants and storage facilities, as reported e.g. in Poland for the 1997 flood (Kulig, 2000), or in New Orleans (USA) following Hurricane Katrina (Pardue et al., 2005; Suedel, et al., 2007). Overall, it has been anticipated that floods will result in deteriorated RBF process filtrate quality arising from increased infiltration of surface water due to the removal of the colmation layer as a result of shear and the associated increase in the hydraulic gradient from the river to the wells (Hiscock and Grischek, 2002). Despite these logical hypotheses, an Austrian study focused on the first meter of the flowpath demonstrated that immediately after flooding, the portion of infiltrated river water in an abstraction well decreased significantly despite a constant hydraulic conductivity for the river bed (Wett et al., 2002). Here, bank storage in the unconfined alluvial aquifer caused a significant decrease in the seepage rate after the high-water event (flood). Specifically, groundwater recharge by precipitation and stream stage elevation during the flood increased the groundwater table; after flooding, the increased groundwater head combined with decreased stream stage reduced hydraulic slope and seepage rate by approximately 50% of the mean value (Wett et al., 2002). Numerical modeling analyses of the effects of flood-induced nitrate and atrazine loads on RBF process filtrate quality showed the possibility of a breakthrough at the pumping well for a highly conductive bank without sorption and degradation, however, an associated breakthrough of atrazine was never detected at the pumping well. When simulations involving equilibrium sorption and literature reported decay coefficients were conducted, however, they demonstrated that atrazine content at the pumping well should be below the detection level (Ray et al., 2002). Subsequent simulations conducted with both horizontal and vertical wells revealed that hydraulic conductivity of the river bed or bank material had a pronounced impact on RBF process filtrate quality (Ray, 2004). These outcomes underscore the site specific nature of contaminant transport and fate during RBF.

7. 3 RBF AROUND THE WORLD

Despite the challenges associated with the quantitative and qualitative management aspects of designing and maintaining RBF systems, several European countries have utilized RBF (some for over 200 years) to augment the removal of natural organic matter (NOM), organic contaminants, inorganic pollutants, and microbial pathogens from as much as 80% of their drinking water (Netherlands, 7%; Germany, 16%; Hungary, 40%; Finland, 48%; France, 50%; Slovakia, 50%; Switzerland, 80%) (Figure 7.5) (Stuyfzand, 1989; Doussan et al., 1995; Miettinen et al., 1997; Kühn and Mueller, 2000). In the United States and Canada, the use of RBF systems for municipal water production began less than 50 years ago, with several installations (e.g., Cincinnati, OH; Columbus, OH; Sonoma County, CA; Kansas City, KS; Sioux Falls, SD) exceeding production capacities of 152 Million liters per day (Ml/d) (40 million gallons per day, MGD) (Ray et al., 2002a). The use of RBF is not limited to Europe and North America, however; globally, several cities are investigating the use of RBF to produce higher quality water from polluted river sources. For example, Ray (2008) reports that several cities in India have started using RBF (Faridabad, located just south of Delhi in the state of Haryana, and metro Delhi). Ahmedabad in the State of Gujarat, and Kota in Rajasthan State have used RBF; Hardwar in the state of Uttarakhand uses a number of collector wells to extract water from the Ganga River and its diversion canals (Ray, 2008). In 2004, a full-scale RBF plant was built in the Nile Valley in Upper Egypt to supply safe drinking water for Sidfa City (population 30,000) (Shamrukh and Abdel-Wahab, 2008). In the City of Kimhae, along the Nakdong River in Korea, an RBF system is being installed to supply 200,000 m^3/day of drinking water daily (Ray, 2008).

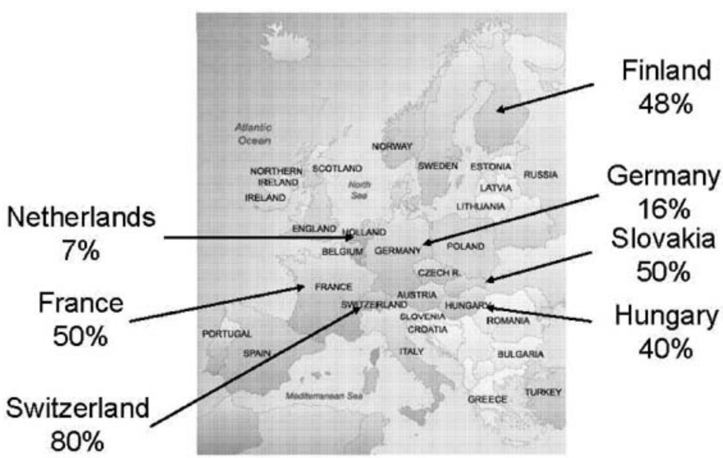

Figure 7.5: Percentage of European Drinking Water Treated by RBF (%) (After Kühn and Mueller, 2000)

In most European countries, there are no specific regulations regarding the implementation of RBF; rather, local authorities recommend guidelines to water utilities to ultimately help ensure acceptable drinking water quality. In the United States and Canada, most of the interest around RBF has focused on its potential to remove pathogens from water, thus improving raw water quality and reducing required levels and costs of subsequent treatment. The water supply industry has adopted the broadly defined regulatory concept of "groundwater under the direct influence" (GWUDI; GUDI in Canada) of surface water (variably defined and implemented in response to local conditions by each State, Tribe or other regulatory agent). Groundwater sources in this category are considered at risk of being contaminated with surface waterborne pathogens; species of the human pathogen *Cryptosporidium* are of particular concern because the oocysts of these protozoan parasites are particularly resistant to common chemical disinfectants used during drinking water treatment. As discussed by Emelko et al. (2008), North American approaches to regulatory development to protect public health from waterborne pathogens have been based on a Treatment Technique (TT) approach that relies on performance indicators and assumes specific levels ("credits") of pathogen removal through well-operated treatment processes. The U.S. Long Term-2 Enhanced Surface Water Treatment Rule (LT2ESWTR) and similar regulations in Canada (e.g., Ontario Ministry of the Environment, 2001) award treatment credit for RBF systems that meet certain design criteria, including defined separation distances and water travel times between a river and an abstraction well (USEPA, 2006; Ontario Ministry of the Environment, 2001). For example, the design criteria specified in LT2ESWTR are based on conservative estimates drawn from colloid filtration theory (described below) and an analysis of microbial monitoring data from existing bank filtration sites. Only unconsolidated, granular aquifers (i.e., those comprising sand, clay, silt, rock fragments, and pebbles) are eligible for bank filtration credit (USEPA, 2006). Horizontal and vertical wells drilled into unconsolidated, granular aquifers are eligible for 0.5 log (68%) removal credit or 1.0 log (90%) removal credit when located at least 25 or 50 feet (7.6 or 15.2 m) from the source, respectively.

In the United States and Canada, federal and state/provincial agencies are reluctant to grant treatment credits for RBF without studies demonstrating the pathogen removal efficacy of RBF processes. Given that such studies often rely on non-pathogen indicators of performance and cannot always anticipate operational changes (e.g., changes in land and river use and management, water demand, etc.), utilities are often hesitant about investing in intensive and costly site-specific testing needed to demonstrate RBF performance. As pollutant loads to drinking water resources increase in response to land use change and increasing demand from agricultural use and industrial and municipal development, the worldwide use of RBF is likely to continue increasing, because it is a relatively inexpensive and sustainable means to improve the quality of surface waters and produces water that is relatively consistent in quality and easier to treat to higher levels of finished quality. Accordingly, an increased understanding of the transport and fate of pathogenic microorganisms and key chemical contaminants in the subsurface environment is necessary so that reasonable design strategies and regulations can be developed and predictable and sustainable water treatment performance can be achieved.

7.4 SURFACE WATER QUALITY CHANGES DUE TO URBAN STORMWATER RUNOFF AND POLLUTION

River water quality varies considerably in nature due to differing rates of geochemical weathering processes that are governed primarily by environmental conditions such as basin lithology, vegetation and climate (Meybeck and Helmer, 1989). However, streams and groundwater in river basins that have undergone significant land use change due to deforestation, agriculture and urban development contain a complex mixture of inorganic, organic pathogenic components that profoundly alter water quality (Meybeck, 1998; Jackson et al., 2001; Jiang and Chu, 2004). The types and concentrations of these components are closely linked to land use (Table 7.1) and related management practices (Klein, 1979; Johnson, 2001; Marsalek and Chocat, 2002). Runoff from urban areas is ultimately discharged to receiving waters as urban stormwater by storm sewers or as combined sewer overflows (CSOs). Both stormwater and CSO discharges seriously impair beneficial water uses in many locations (House et al. 1993) and produce a range of physical, chemical, biological and combined effects (acute or cumulative) on receiving waters (Harremoes 1988). In particular, urban wet-weather flows (i.e., stormwater and CSOs) are recognized as sources of fecal contamination impacting on recreational waters and threatening drinking water sources (Marsalek and Rochfort, 2004). The concentrations of many pollutants observed in impacted streams from both stormwater and CSOs vary (per rainfall event, seasonally and annually) as well as among basins that are more vulnerable to contamination (Horowitz et al., 1990; Marsalek et al., 1993). In one half of the Areas of Concern in the Canadian Great Lakes region, Weatherbe and Sherbin (1994) reported serious impacts of stormwater and CSO discharges. For the most impacted water bodies, a term "effluent dominated water bodies" has been coined (Novotny, 2007) and refers to the water bodies, which contain predominantly wastewater effluents during all or a part of a year. This is caused by importing large quantities of water into urban areas and changing it into wastewater through urban use.

Natural features of the landscape (geology, soil type, vegetation) and land use and drainage management (tile drains, irrigation and urban drainage practices) can affect the surface and subsurface transfer of chemicals and thereby exert important local and regional controls on water quality (Horowitz et al., 2001). Improved knowledge of the local, regional and national importance of land and chemical use on water quality is central to increasing the effectiveness of policies designed to protect water resources in a wide range of geographical settings (Mitchell and Hollick, 1993; Mitchell, 2001). The impacts of urbanization on the environment and water quality have been well documented (Chambers et al., 1997; Marsalek et al., 2008). Urbanization increases pollutant loads in runoff because 1) the volume and rate of runoff typically increase as an area is developed thereby increasing the capacity for transporting pollutants and 2) pollutants may be more prevalent given the type of development or more available for loss in runoff as the intensity of the land use increases. Several regional studies have shown that relative to reference (undisturbed) rivers, urban rivers often have elevated levels of phosphorus (Paul and Meyer, 2001; Jarvie et al., 2006), nitrogen (Jarvie et al., 1998), suspended solids (Bilotta and Brazier, 2008), biochemical oxygen demand (Lawler et al., 2006), pesticides (Paul and Meyer, 2001), trace metals (Marsalek and Schroeter, 1989; Stone and Marsalek, 1996), trace organic pollutants (USEPA, 1983; Heaney and Huber, 1984; Marsalek and Schroeter, 1989; Stone and Haight, 2000) and pathogens (Lopes et al. 1995; Ceballos et al., 2003; Jiang and Chu, 2004).

Table 7.1: The effect of land use on nutrient and pesticide levels in streams and groundwater in 20 river basins and aquifer systems in the United States (USGS Circular 1225)

RELATIVE LEVEL OF CONTAMINATION

	Streams			Shallow Ground Water	
	Urban areas	Agricultural areas	Undeveloped areas	Urban areas	Agricultural areas
Nitrogen	Medium	Medium–High	Low	Medium	High
Phosphorus	Medium–High	Medium–High	Low	Low	Low
Herbicides	Medium	Low–High	No data	Medium	Medium–High
Currently used insecticides	Medium-High	Low–Medium	No data	Low–Medium	Low–Medium
Historically used insecticides	Medium-High	Low–High	Low	Low-High	Low-High

In urban watersheds, water quality is strongly influenced by the extent and type of urbanization (Paul and Meyer, 2001) as well as differences in land use and related management practices (Jones et al., 2001). Urban lands can have impervious surface cover (ISC) ranging from 10 to 90 percent (Brun and Band, 2000) and the relationship between the surface cover type, extent, function and connectivity to surface waters strongly influence the water quality of receiving streams. Several studies have demonstrated that when ISC is less than 10%, stream ecosystems remain healthy and water quality is acceptable. However, many studies suggest ISC has a threshold effect causing water quality degradation when ISC > 10 % (Arnold and Gibbons, 1996). Examples of stream degradation from increasing ISC include excessive bed and bank stream erosion (Leopold, 1968), channel stability (Bledsoe and Watson, 2001), changes to water temperature (LeBlanc et al., 1997), reduced groundwater recharge (Brunke and Gossner, 1997), increased size and frequency of 1-25 year floods (Leopold, 1968), decreased movement of groundwater to surface water (Brunke and Gossner, 1997), loss of riparian habitat (Paul and Meyer, 2001), increased contaminants in water (Marsalek and Schroeter 1989; Stainton and Stone, 2003), increased fine sediment deposition in the stream bed (Stone and Droppo, 1994; Brunke and Gossner, 1997; Droppo et al., 2002) and overall degradation of aquatic habitat (Paul and Meyer, 2001). With respect to biodiversity, the impacts on urbanization are measured by the biological community performance, which is affected by habitat structure, flow regime, water quality, food (energy) sources, and biotic interactions (Yoder, 1989). All these factors are adversely impacted by urbanization, and consequently, biodiversity in urbanizing areas is reduced.

Cohesive materials (<63 μm) represent a significant proportion of the annual suspended sediment flux in many rivers (Stone and Saunderson, 1992) and in urban stormwater runoff (Krishnappan et al., 1999; Droppo et al., 2002). The settling of fine-grained suspended solids in receiving streams promotes the formation of cohesive sediment beds referred to as surficial fine-grained lamina (SFGL) (Droppo and Stone, 1994) that represents a significant in-channel source of contaminants (Stone and Droppo, 1994). The physical characteristics (i.e. particle size, settling velocity, porosity and effective density) of SFGL are affected by turbulent shear, suspended sediment concentrations (Dyer and Manning, 1999) and biological activity (i.e. presence of micro-organisms, biofilm formation, etc.) (Droppo, 2001). Biofilm formation can change the deposited sediment characteristics (i.e. particle structure, morphology, size, porosity, shape, degree of consolidation) and influence erosion rates (Lau 1997; Droppo et al., 1997; Amos et al., 2003).

The formation of SFGL and the development of microphytobenthos populations (biofilms) on the stream bed clogs the top layer of channel sediment, which has a significant effect on important physical and biogeochemical processes that occur within the stream bed. In impacted rivers receiving increased quantities of cohesive sediment from urban and agricultural sources, the rates and magnitude of colmation will increase and therefore hinder the exchange processes between surface water and groundwater. Accordingly, in streams where physical and biological processes governing colmation are enhanced, the hydraulic conductivity of the stream bed will be reduced (Brunke and Gossner, 1997). Ultimately in degraded rivers, the colmation process will likely have a profound negative effect on both the quality and quantity of river water thus reducing opportunities for riverbank filtration.

7.5 ORGANICS REMOVAL

The fate and transport of organic compounds (e.g., NOM, pesticides, herbicides, pharmaceuticals) in the subsurface (and their removal by RBF), depends on hydrologic conditions as well as the physical, chemical, and biological processes occurring in the soil and the vadose and saturated zones. Advective transport and hydrodynamic dispersion in conjunction with processes such as precipitation, adsorption/desorption, physico-chemical filtration and straining, volatilization, metabolism, and biotic and abiotic degradation lead to the retardation or complete removal of anthropogenic compounds. Because many organic pollutants are hydrophobic, they tend to be readily adsorbed by sediments or organic materials. Polynuclear (or polycyclic) aromatic hydrocarbons (PAHs) are another example of hydrophobic organics that are strongly attenuated by soil and aquifer materials. Most pesticides, except herbicides, are hydrophobic and can adsorb to aquifer materials; volatilization after application is also important for pesticides. When organics adsorb onto colloidal particles, some of the fate and transport processes they are impacted by (e.g., physico-chemical filtration and straining) are the same as those discussed below for pathogens. Transformation of organics by both biotic and abiotic degradation. Many organics such as carbamates or triazines are transformed by hydrolysis

(Verstraeten et al., 2002). Microbiologically-mediated redox processes, or biodegradation processes, are critical for the removal of many organic compounds; groundwater is particularly vulnerable to contamination by organic compounds if biological transformations are slow because of lower temperatures and decreased enzymatic activity (Grass et al., 2000). For example, attenuation of common pollutants such as chlorinated ethenes, often generally referred to as chlorinated solvents, is typically dominated by microbiologically-mediated reductive dehalogenation reactions that only occur in reducing environments (Wiedemeier et al., 1999). Petroleum hydrocarbons, including the BTEX compounds, are aerobically and anaerobically biodegraded, though aerobic degradation processes are more rapid (Wiedemeier et al.1999). Biodegradation only results in complete mineralization of organics to compounds such as carbon dioxide and water in a few cases; for example, acetyl salicylic acid (Verstraeten et al., 2002). In most cases metabolites are formed that may increase, decrease, or negligibly affect net toxicity (Barbash and Resek, 1996). Several studies have reported the fate of organic pollutants such as herbicides, pesticides, pharmaceuticals, and odor compounds during riverbank filtration (Piet and Zoeteman, 1980; Sontheimer, 1980; Schwarzenbach et al., 1983; Dünnbier et al., 1997; Heberer et al., 1997; Jüttner, 1999,1995; Grass et al., 2000).

A complex mixture of dissolved and particulate humic and non-humic organic substances, NOM contributes undesirable taste and odor compounds to drinking water, facilitates the transport of toxic contaminants in groundwater, and acts as the main precursor for disinfection and oxidation by-products. NOM and disinfection by-product (DBP) precursor material can be effectively removed by RBF (Miettinen et al.1994, 1998; Ćosović et al. 1996; Kühn and Mueller 2000; Ray et al. 2002a,b; Wang, 2002; Weiss et al. 2002, 2003a,b, 2004). Different components of NOM can have a substantial effect on its treatability, and its mobility in the subsurface, its reactivity with chlorine, and the speciation of the DBPs formed as a result of this reaction (Aiken and Cotsaris 1995; Croué et al. 1999). The mobility of NOM in the subsurface increases with decreasing molecular weight and hydrophobicity, which suggests that smaller and more hydrophilic fraction of NOM may dominate NOM-facilitated transport of contaminants in groundwater (McCarthy et al., 1993; Ludwig et al., 1997; Gerlach and Gimbel, 1999). While the overall concentrations of organic DBP precursors are effectively reduced during bank filtration, the reductions appear to be largely the result of the reduction in NOM concentration rather than a consistent change in NOM character (Weiss et al., 2004). It has been generally found that NOM concentrations decrease with retention time during RBF; however, the influence of dilution with groundwater must also be considered (Sontheimer, 1980; Schwarzenbach et al., 1983; Kühn and Mueller, 2000). Figure 7.6 illustrates reductions of dissolved organic carbon (DOC) achieved by an RBF along the Rhine River in Germany.

7.6 INORGANICS REMOVAL

Inorganic pollutants such as heavy metals, nitrates, and sulfates that are harmful to humans and animals may also be toxic to microorganisms that are critical to controlling the quality of RBF process filtrate (von Gunten and Kull, 1986).

Anthropogenic sources of these pollutants include urban stormwater runoff, atmospheric deposition, road salting, discharges from mining operations, sewage, and pesticide and fertilizer residues. The transport and fate of inorganic pollutants such as metals and other non-degradable cations during RBF is impacted by several potentially concurrent and competing processes and has therefore been studied extensively (Doussan et al., 1985; von Gunten and Kull, 1986; Jacobs et al., 1988; Bourg et al., 1989; von Gunten et al., 1991; Bourg and Bertin, 1993; von Gunten and Zobrist, 1993; Bourg and Darmendrail, 1995; Doussan et al., 1998). These attenuation and mobilization processes include sorption onto clay minerals, oxides, and/or hydroxides; precipitation; redox reactions; complexation with organic matter; mobilization due to microbial degradation of NOM; and dilution (von Gunten and Kull, 1986; Jacobs et al., 1988; Darmendrail, 1988; Bourg et al., 1989; von Gunten et al., 1991; Bourg and Bertin, 1993;).

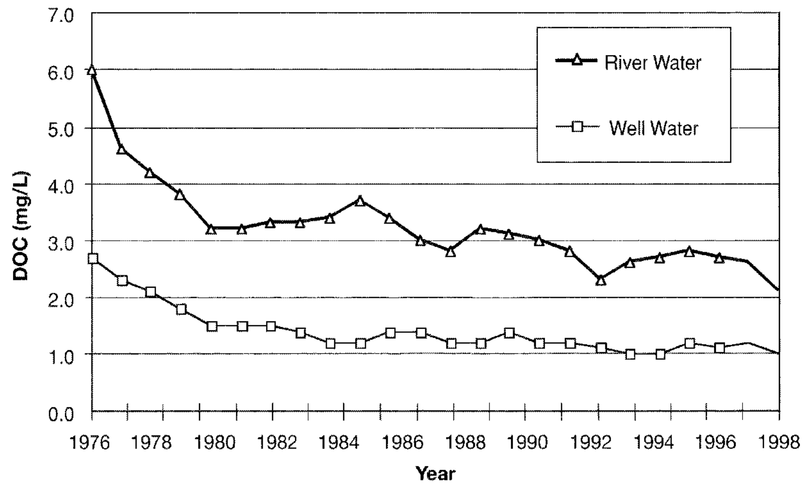

Figure 7.6: DOC concentration in the Rhine River and RBF process filtrate (Schubert et al., 2002a)

Co-precipitation and/or adsorption colloidal particles and aquifer materials is an critical removal mechanism for metals such as zinc and cadmium (Jacobs et al., 1988; von Gunten et al., 1991; Bourg et al., 1992; Bourg and Bertin, 1993). In addition, complexation with other inorganic and organic ligands, the heterogeneous mixture of refractory organic substances arising from microbial degradation and synthesis processes that comprises NOM complexes metal ions such as aluminum, cadmium, copper, iron, lead, and nickel and facilitates their transport in aqueous systems (Christensen et al., 1996; Schmitt et al., 2003; Borrok et al., 2007). In RBF systems, near the river bed or bank, a reduced environment can arise because of oxygen depletion due to microbiological activity in riverbed sediments. The decreased redox potential in this zone can readily mobilize oxide-forming metals such as iron and manganese by reductive dissolution (Bourg and Bertin, 1993). Fluctuations in microbial activity and water pumping patterns impact the development of this reduced zone (Bourg et al., 1989). Conditions become more oxidizing when conditions do not favor microbial degradation of organic matter; for example, at colder water conditions (Bourg and Bertin, 1994) or at some distance from the infiltration point due to electron donor deficiency (Bourg and Bertin, 1993). Aquifer reaeration removes metals such as iron and manganese from the solution by a series of precipitation reactions. Sorption onto or co-precipitation by manganese oxides can significantly inhibit the transport of trace metals (von Gunten and Kull, 1986). The precipitation of iron and manganese oxy-hydroxides as well as calcium carbonates, however, can reduce hydraulic conductivity and permeability (Fuller and Harvey, 2000).

7.7 PATHOGEN REMOVAL

A wide range of microbial pathogens are known to contaminate surface water bodies, including more than 100 viral and several bacterial pathogens and protozoan parasites. Such pathogens can be introduced into surface water bodies from various sources including discharges of municipal wastewater effluents, urban stormwater, CSOs, land disposal of agricultural waste, and runoff from wildlife fecal deposits. During infiltration of contaminated surface waters through riverbanks, various physical processes influence the fate of microbial pathogens in the aqueous phase, including advective transport, dispersive transport, physicochemical filtration (or retention) of organisms on sediment grain surfaces, physical straining (or wedging) of organisms in pore spaces that are too small to allow passage, and dilution with subsurface waters. Biological processes that may also affect the concentration of microbial pathogens during RBF include inactivation or die-off of microbes, and grazing by higher trophic levels such as bacterivorous flagellates. Of these physical and biological processes, filtration and inactivation are believed to play an important role in the reduction of pathogens in the aqueous phase (Tufenkji, 2007).

In granular porous matrices such as those found at RBF sites, microbe filtration rates are controlled by properties of the microorganisms and the sediment grains. Microbe size is of particular importance as it governs the likelihood of microbe contact with grain surfaces during surface water infiltration (Yao et al., 1971; Tufenkji and Elimelech, 2004). Filtration of smaller-sized pathogens such as viruses (typically < 0.2 μm in size) is governed by the mechanism of Brownian diffusion, whereby the microbe is brought into contact with the grain surface as a result of its Brownian motion. Larger microorganisms such as the protozoan parasites *Cryptosporidium* (4 – 6 μm) or *Giardia* (9 – 12 μm) contact sediment grain surfaces as a result of gravitational sedimentation and interception (Yao et al., 1971; Tufenkji and Elimelech, 2004). Thus, the contact efficiency of microbial pathogens with sediment grains increases with microbe size for larger organisms, but decreases with microbe size for smaller (e.g., virus-sized) pathogens. Accordingly, microbial pathogens on the order of 1 μm in diameter (i.e., bacteria) are least effectively removed during granular filtration as they experience the fewest contacts with grain surfaces. Figure 7.7 shows the theoretical contact efficiency as a function of microbe size for conditions typically encountered during riverbank filtration. In effect, the likelihood of microbe contact with the sediment grain surface is shown to be at a minimum for organisms about 1-2 μm in size.

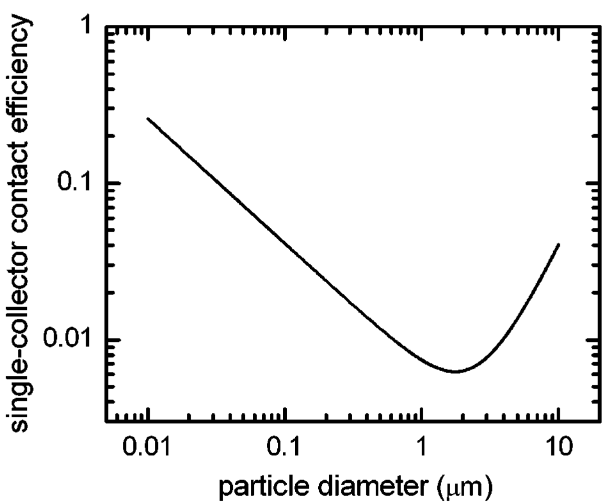

Figure 7.7: Predicted single-collector contact efficiency (η_0) based on a correlation equation developed by Tufenkji and Elimelech, 2004 calculated for representative conditions encountered in riverbank filtration. Adapted with permission from Tufenkji and Elimelech (2004). Copyright (2004) American Chemical Society

Few field-scale studies have examined microbe removal efficiency during riverbank filtration (Havelaar et al., 1995; Medema et al., 2000; Schijven et al., 2002, Weiss et al., 2005). Several of these studies have been completed in The Netherlands, where 7% of the drinking water supply is treated by RBF. In general, RBF effectively removes enteric viruses or their surrogates (e.g., bacteriophage) from the pore fluid. Interestingly, concentrations of total coliforms, fecal streptococci, and spores of sulfite-reducing clostridia were also found to be significantly reduced in these studies (Havelaar et al., 1995; Medema et al., 2000; Schijven et al., 2002; Wang, 2002), as demonstrated in Figures 7.8 and 7.9. Because of the inherent variability in physical and geochemical conditions at different RBF sites and the limited data on microbe transport and fate in these settings, it is difficult to make generalizations and predictions regarding the expected extent of microbe removal. Indeed, more studies are needed to examine microbe transport and fate in different RBF environments, particularly in alluvial valley aquifers consisting of gravel deposits. Moreover, the role of the colmation layer in pathogen removal during RBF is not well understood. In nutrient-rich granular environments such as those typical of riverbanks, biofilms may develop on the surface of sediment grains. Laboratory studies have shown that the presence of biofilm in granular porous matrices can affect particle and microbe transport behavior as well as matrix properties including permeability and porosity. Yet, the importance of biofilms in retaining (and releasing) microbial pathogens during RBF has not been established. Clearly, a better understanding of the influence of site-specific factors such as surface water quality and properties of the riverbed sediments is needed before a generalized framework for assessing RBF treatment credit can be proposed.

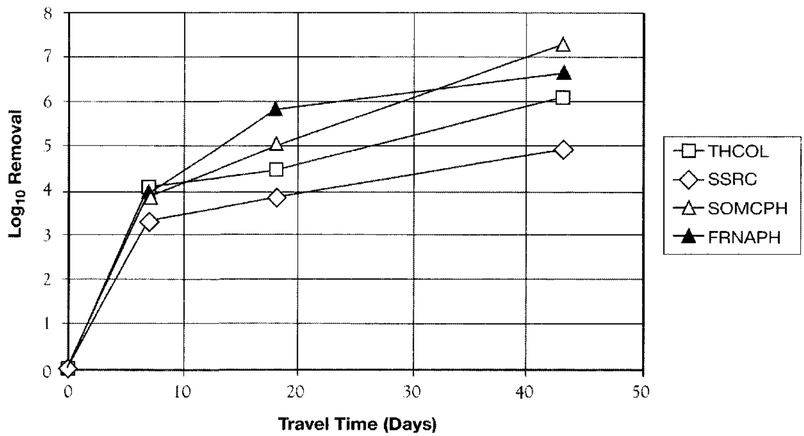

Figure 7.8: Removal of thermotolerant coliforms (THCOL), spores of suphite-reducing clostridia (SSRC), somatic coliphages (SOMCPH), and F-specific RNA bacteriophages (FRNAPH) by RBF (Medema et al., 2000)

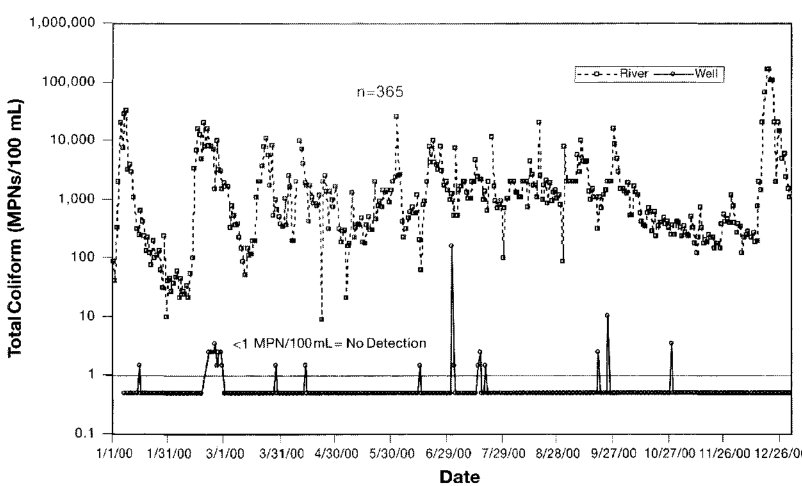

Figure 7.9. Removal of total coliforms by RBF (Wang, 2002)

7.8 COMPENSATION FOR PEAKS AND SHOCK LOADS

Water flow, municipal and industrial discharges, runoff, seasonal effects (e.g. changes in temperature, water quality, water flow), and accidental spills (of both minor magnitude that go unreported/unnoticed and of severe magnitude that cause shock loads and may cause water treatment plant intake shut down) all cause the concentration of river pollutants to vary significantly. It is generally recognized that RBF systems can provide compensation for concentration peaks and shock loads (specifically associated with accidental contaminant releases of severe magnitude) and that the impacts of the releases or concentration peaks of many contaminants are generally less evident in RBF process filtrate than in impacted source waters (Kühn and Mueller, 2000; Mälzer et al., 2002; Ray, 2004). RBF compensates for contaminant peaks and shock loads because sorption and degradation phenomena contribute to significantly longer travel times relative to direct intakes into water treatment plants; blending with groundwater further compensates for peaks and shock loads by dilution (Kühn and Mueller, 2000; Mälzer et al., 2002). There is also a lag time between the peaks observed between pumped water and surface water, which allows utilities the opportunity to better respond to emergencies (Mälzer et al., 2002). An additional advantage of RBF is that riverbanks are filled with more water during periods of high flow than in drought situations. Thus, during periods of high flow, there is also more dilution (Kühn and Mueller, 2000). Kühn and Mueller, (2000) presented examples of the ability of RBF systems to compensate for contaminant concentration peaks and variability by presenting a comparison of chloride concentrations in Rhine River water and RBF process filtrate (Figure 7.10). The authors noted that while chloride concentration in the river water varied between 145 and 320 mg/L, chloride concentrations in the RBF process filtrate only varied from

100 to 160 mg/L; moreover, the periodic variations in chloride concentration observed in the river water were not observed in the RBF process filtrate.

Water treatment systems along the Rhine River have provided several examples of the protection that RBF systems can provide against shock loads. Figure 7.11 presents concentrations of the organic solvent and polyvinyl chloride (PVC) precursor1,2-dichloro-ethane (also known as ethylene dichloride [EDC]) in the Rhine River during part of 1986. During the course of approximately one day, the concentration of 1,2-dichloroethane reached ~35 μg/L; approximately three weeks later, the contaminant was detected in the RBF process filtrate at a concentration of ~1 μg/L over a period of two to three weeks (Kühn and Mueller, 2000). Similarly, on November 1, 1986, a fire started in an agrochemical store of the Sandoz Chemical Plant in Basel, Switzerland. The water used to fight the fire drained into the Rhine River and carried large quantities of insecticides, herbicides, and fungicides with it, causing a massive fish kill and the shutdown of many water intakes along the river (Mälzer et al., 2002). To investigate the impacts of chemical shocks on filtrate quality, a series of four filter columns were filled with granular pumice so that the combined effects of sorption, dispersion, and biodegradation on sodium chloride and nitrobenzene concentration could be investigated (Mälzer et al., 2002); it was found that biodegradation substantially reduced nitrobenzene leakage. The ability of RBF systems to compensate against shock loads of contaminants has also been demonstrated by Ray (2004) with numerical modeling analyses of the effects of flood-induced nitrate and atrazine loads on RBF process filtrate quality as discussed above.

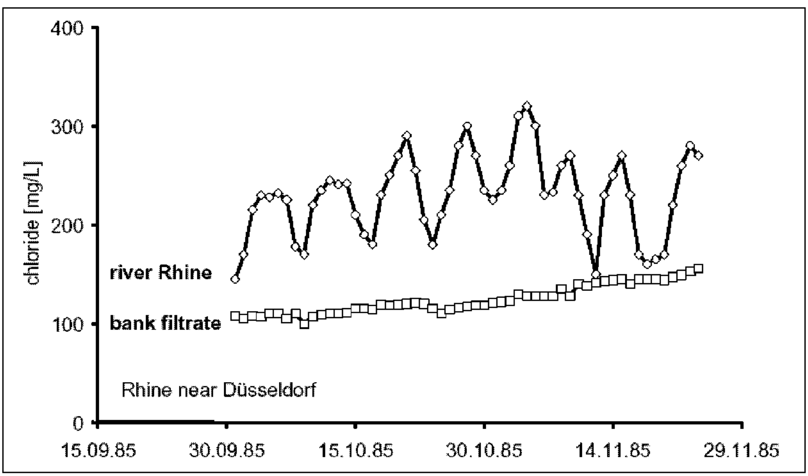

Figure 7.10: Chloride concentration in the Rhine River and RBF process filtrate in 1985 demonstrating RBF compensation for contaminant concentration peaks and variability (Kühn and Mueller, 2000)

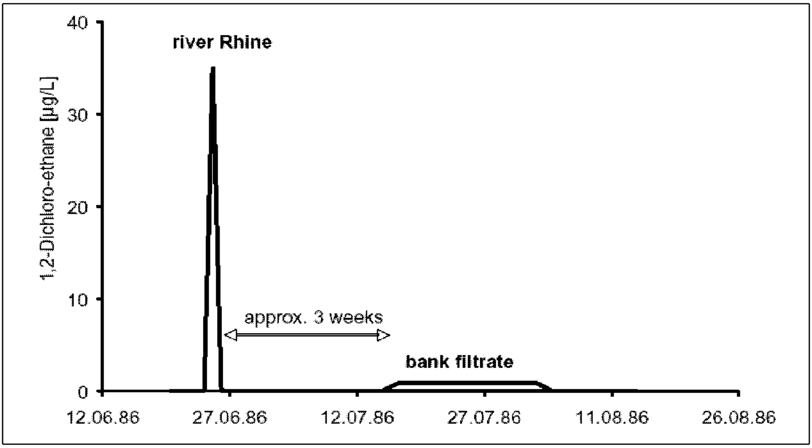

Figure 7.11: 1,2-dichloro-ethane concentrations in the Rhine River and RBF process filtrate in 1986 demonstrating RBF compensation for shock loads of some contaminants (Kühn and Mueller, 2000)

7.9 CONCLUSIONS

Urbanization increases pollutant loads in runoff because of the increased pollutant production by land use activities (particularly traffic byproducts and industrial releases) and the increased volume and rate of urban runoff providing increased capacity to transport such pollutants. Accordingly, these pollutants may be more prevalent given the type of development or more available for loss in runoff as the intensity of the land use increases. Several regional studies have shown that relative to undisturbed rivers, urban rivers often have elevated levels of phosphorus, nitrogen, suspended solids, biochemical oxygen demand, pesticides, trace metals, organic pollutants and pathogens. The most impacted rivers contain predominantly municipal effluents and are referred to effluent dominated waters. Riverbank filtration (RBF) has been a recognized and utilized water treatment technology for at least 200 years in Europe and has the potential to effectively mitigate several urbanization-associated, detrimental impacts on water quality. Unlike conventional granular media filtration or alternative pressure-driven membrane technologies that typically require source water with relatively low solids content or chemically-assisted solids reduction prior to use, RBF utilizes groundwater derived from infiltrating surface water by situating production wells near rivers and exploiting river water infiltration through river beds and/or banks. RBF has demonstrated the ability to remove or degrade a wide range of dissolved and suspended waterborne contaminants, including natural organic matter and disinfection by-product precursors, organic contaminants (including some micropollutants such as pharmaceuticals, chelating agents, aromatic sulfonates, aliphatic amines etc.), inorganic contaminants such as metals, and pathogens; it can also mitigate chemical shock loads that may result from emergency spills or spring flooding in agricultural watersheds. The physical, chemical, and microbiological quality of RBF filtrate depends upon numerous factors, including source water quality (which is influenced by land use and climate) and quantity, river flow velocity and bed load characteristics, seasonality of river flow, stability and management (e.g. dredging) of the river channel, local geologic settings, distance of the wells from the surface water source, well pumping rates, the characteristics of sediments at the river aquifer interface, and ultimately the physical, chemical, and microbiological processes acting in the aquifer. As pollutant loads to drinking water resources increase, RBF will continue to receive increased attention and application for drinking water treatment purposes. Like other infiltration strategies, RBF is clearly a valuable and relatively inexpensive tool that can be used to mitigate runoff impacts from events that see the greatest relative increases from urbanization; that is, the relatively frequent rainfall events that are small enough to produce little or no runoff from pervious surfaces, but produce runoff from impervious areas. RBF also mitigates the impacts of shock loads of a variety of pollutants such as those that are associated with larger, more intense, and relatively rarer storm events, which are capable of producing significant runoff even in undeveloped basins. Consequently, RBF represents an effective barrier in the multiple barrier approach to defending against waterborne pathogens and contaminants in drinking water.

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CHAPTER 8

Use of Functionalized Filter Medium for Nutrient Removal in Stormwater Ponds

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ABSTRACT: Many best-management practices (BMPs) currently used in urban regions have been developed to minimize flood risk, sustain downstream ecosystems, and secure the quality of groundwater sources either directly or indirectly. One such BMP is the reuse of stormwater by removing nutrients from the nutrient-laden "first flush" water collected in wet or dry ponds; this practice has led to the development of multifunctional sorption materials. In the interests of sustainable infrastructure, this chapter describes a design philosophy for proper use of functionalized green sorption or filter media placed in stormwater ponds for nutrient removal. The chapter includes a thorough literature review of past uses of green sorption media and also describes the results of a laboratory column test conducted to mimic a field environment and evaluate the feasibility of the design philosophy. This approach to stormwater treatment has "green" implications because it includes recycled material in the sorption media to promote treatment efficiency and effectiveness. Such design strategies may be extended to enhance the sustainability of low-impact developments such as rain gardens, bioswales, and green roofs where plants, sorption media, and soils naturally filter nutrients and other pollutants from stormwater.

8.1 INTRODUCTION

Nutrients such as ammonia, nitrite, nitrate, and phosphorus are common contaminants in water bodies worldwide. Directly or indirectly, these nutrients have acute and chronic harmful effects on humans and ecosystems. Ammonia can exist in aqueous solution in the form of ammonium ions or ammonia gas. Ammonium ions (NH_4^+) are much more toxic than ammonia gas (NH_3); however, the form present depends on the pH (Crites and Tchobanoglous, 1998). According to the United States Environmental Protection Agency (USEPA), unionized ammonia is extremely toxic for salmonid and nonsalmonid fish species (USEPA, 1993). The presence of 0.10–10.00 mg/L of ammonia can cause fish mortality and negatively affect fish health and reproduction

(USEPA, 1993). Nitrate is more toxic than nitrite and can cause human health problems such as liver damage and even cancer (Gabel et al., 1982; Huang et al., 1998). Nitrate can also bind with hemoglobin and create an oxygen deficiency called methemoglobinemia in warm-blooded animals, especially in the young babies (WEF, 2005). Meanwhile, nitrite has been associated with "brown blood diseases" in warm-water fish (Kentucky Water Watch, 2008), and is carcinogenic when involved in chemical and enzymatic reactions with amines (Sawyer et al., 2003). In addition, both phosphorus and nitrogen species can trigger eutrophication, which occurs when excessive nutrients in a water body encourage the excess growth of plants such as algae and weeds. These plants consume available oxygen in the water, leaving less available for fish and other aquatic species. Ultimately this condition degrades both the aesthetics and ecosystem health of rivers, springs, and lakes.

Compounds containing nitrogen and phosphorus are found in stormwater runoff. While highways are a main source of these nutrients in stormwater runoff (USEPA, 1999a), many other sources are also contributing to increased nutrient contents in stormwater and groundwater. These sources include agricultural fertilizers, untreated wastewater, insufficiently treated wastewater from septic tanks, and animal urine and droppings. Phosphorus is released from fertilizer, dead plants, animal waste, detergents, forest fires, synthetic materials, and decaying animal bones. Most phosphorus exists in the form of orthophosphate (PO_4^{3-}, OP) (Crites and Tchobanoglous, 1998). As populations increase and various regions worldwide face water scarcity, obtaining high-quality clean drinking water is increasingly important, especially when the groundwater becomes the major source of drinking water. Treating stormwater in a way that is both cost effective and meets drinking-water regulatory standards is challenging.To meet the goal of sustainable development, stormwater may also be needed to recharge and sustain groundwater supplies. If the nutrient-laden "first flush" runoff water collected in wet or dry ponds cannot be properly treated, it will eventually percolate through the soil and contaminate the groundwater. According to the USEPA maximum contaminant levels (MCLs), nitrate and nitrite levels in water bodies should not exceed 10.00 mg/L $N0_3^-$–N and 1.00 mg/L NO_2^-–N, respectively (USEPA, 1988). Nutrient removal from stormwater runoff is therefore important for sustaining human society and aquatic ecosystems.

Under existing hydrologic systems, the high nitrogen and phosphorus concentrations in stormwater runoff, contaminated groundwater, landfill leachate, and domestic and industrial wastewater effluents will continue to increase surface and groundwater contamination and reduce the potential for water reuse. In water management, best-management practices (BMPs) include maintenance procedures and management practices aimed at preventing or reducing water pollution. As both quantity and quality of stormwater runoff are taken into account in the U.S., low-impact development (LID), applying a suite of landscape architecture practices in urban ecology to minimize the hydrological effects of urban development, has been emphasized and assessed by comparing pre- and post-development hydrology as an integral part of BMPs. Relevant concepts associated with LID worldwide include the sustainable urban drainage system (SUDS) in the United Kingdom and water-sensitive urban design (WSUD) in Australia. In an effort to promote these engineering BMP practices, engineered, functionalized, and natural sorption media can be used to remove nutrients in stormwater runoff, wastewater effluent, groundwater flows, landfill leachates, and drinking-water sources via both physicochemical and microbiological processes embedded in most of the BMPs, such as bioswale, biofiltration, permeable reactive barriers, and pervious pavement (Chang et al., 2008a).

Synthesis of the literature on the relationships of urban development and the hydrological cycle is an important step in developing more advanced and adaptive materials, biomaterials, and multifunctional engineering materials with co-treatment capacities and in creating sustainable neighborhoods and responding to rapid changes in land use and stormwater runoff. Recent studies have reported that removal of ammonia, nitrite, nitrate, and phosphorus can be enhanced by the inclusion of various sorption media such as sawdust, tire crumb, sand, clay, zeolite, sulfur, and/or limestone in natural soil (Kim et al., 2000; Clark et al., 2001; Jokela et al., 2002; Hsieh and Davis, 2005). A number of devices, collectively known as structural BMPs, which can be an effective integrative concept for examining urban carrying capacity, may be used in combination with green sorption media to treat contaminated stormwater based on both physicochemical and microbiological principles (Chang et al., 2008a; Ray et al., 2006). Examples of such devices are rain barrels and bioswales filled with green sorption media, rain gardens with retention/detention ponds and green sorption media, bioswales in front of constructed wetlands filled with green sorption media, green roofs integrated with green sorption media and stormwater reuse, permeable pavement that includes a mixture of coarse sand and green sorption media, level spreaders with green sorption media, and vegetative filter strips with soil and green sorption media mixture.

In this study, we deemed an appropriate media mix to be any kind of multifunctional material or functionalized sorption media that may be used in both natural and built environments to improve the existing physicochemical and microbiological processes for nutrient removal (Chang et al., 2008a). Such a media mix can be made "green" by including recycled materials, such as tire crumb or sawdust, to increase the treatment efficiency and effectiveness (Chang et al., 2008a). The use of these filtering media in stormwater treatment could also resolve solid waste management problems. However, the inclusion of recycled materials in the mixture may raise concerns about toxicity. To answer such questions, Birkholz et al. (2003) conducted toxicological tests on tire crumb and found no DNA- or chromosome-damaging chemicals. From an engineering standpoint, by the use of such green sorption media, nutrients in water bodies can be reduced or even mostly removed by enhanced absorption/adsorption, nitrification/denitrification, and other chemical reactions such as precipitation and ion exchange (Chang et al., 2008b).

8.2 DESIGN PHILOSOPHY

Most filter media improve solid-liquid contact, prevent channeling, and enhance physicochemical processes. In general, the calcium/iron minerals and larger surface areas of clay in natural soil provide more contact areas where solids can be absorbed and more space for bacteria colonies to develop. Soil with fewer clay particles can present problems in regard to the removal of stormwater nutrients. In comparison, functionalized filter media have better ion exchange capacities to support absorption/adsorption, greater capabilities to retain adsorbed nutrients, and larger surface areas for bacterial colony development possibly. Riverbank filtration is an innovative process of this kind that has been widely used for centuries in Europe to remove pollutants, including microorganisms, from surface water (Tufenkji et al., 2002).

Proper deployment of green sorption media may be challenging in structured BMPs. Figures 8.1 and 8.2 show typical detention (wet) ponds and retention (dry) ponds, respectively. The filter media operate as a four-phase system (solid media, water, gas phase, and attached biofilm) designed to remove nutrients and other pollutants, such as heavy metals, pesticides, and bacteria, when stormwater infiltrates into the underlying vadose zone. Isotherm tests can help understand the necessary periodic maintenance and replacement of the filter media. However, laboratory-scale isotherm tests of the filter media can only reveal the physicochemical characteristics of adsorption, which are unrelated to the microbiological activities (i.e., nitrification and denitrification); additional laboratory and field studies are required. This type of system may be also used for groundwater remediation (Schipper et al., 2005).

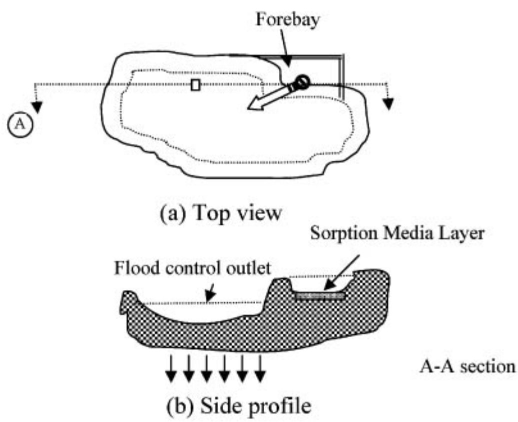

Figure 8.1: Detention (wet) pond with *in situ* treatment units and low infiltration

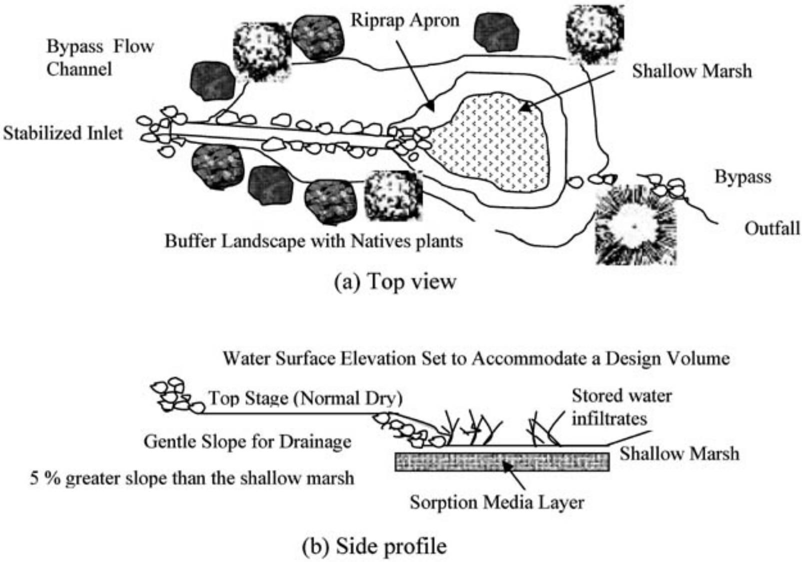

Figure 8.2: Retention (dry) pond with *in situ* treatment

8.3 LITERATURE REVIEW

This section presents examples of increased nutrient concentrations in stormwater runoff in Florida, followed by technologies currently available to remove nutrients, and the use of green sorption media to remove nutrients from stormwater runoff. There are several benefits to using sorption media for nutrient removal. They decrease the cost of treatment without compromising public health, on one hand. They eliminate the cost of chemicals used for nutrient removal, on the other hand. In addition, they solve problems related to the management and disposal of sludge. The conventional use of aluminum sulfate to form the coagulation floc for fine-particle removal produces tremendous amounts of chemical sludge; sorption media achieve the same treatment efficiency without the problem of chemical sludge. Furthermore, sorption media reduce some solid-waste management and disposal problems by including media that would otherwise have no recycle value or reuse potential. For example, scrap tires cannot be used to make more tires (Lisi et al., 2004). The use of sorption media also reduce dependence on bacteria and do not require aerobic or anaerobic conditions.

8.3.1 Nutrient Concentrations in Stormwater and Groundwater in Florida

Sources of nitrogen include stormwater runoff, as well as septic tanks and land-based applications of reclaimed wastewater and fertilizer. Such sources have contributed to elevated nitrate and nitrite concentrations in the Upper Floridian aquifer. Nitrate concentrations have increased in many springs of the Upper Floridian aquifer since the 1950s. Phelps (2004) reported that nitrate concentrations ranged from less than 0.02 to 12.00 mg/L, with a median of 1.20 mg/L, for 56 Upper Floridian aquifer wells sampled in Marion County during 2000 and 2001. Nitrate concentrations have exceeded 1.00 mg/L in recent years at a number of springs in Lake, Marion, Orange, Seminole, and Volusia counties, according to Phelps et al. (2006) and researchers from the St. Johns River Water Management District (SJRWMD, 2008). Increasing trends in nitrate concentration have also been documented at springs in Volusia County, such as DeLeon and Gemini springs (Phelps et al., 2006) as well as Blue Spring (SJRWMD, 2008).

8.3.2 Multifunctional Sorption Media for Nutrient Removal

Much work prior to 1995 focused on removing nutrients by the sand filter method. Various types of sand filter methods were developed, including the Washington D.C. sand filter method, the Delaware sand filter design, and the Austin sand filter (USEPA, 1999a). Removal efficiency of the Delaware sand filter is 70.20% for total suspended solids (TSS), 71.10% for total phosphorus (TP), 67.00% for ammonia nitrogen (NH_3-N), and 59.90% for total Kjeldahl nitrogen (TKN) (Bell et al., 1995). Later on, more advanced physicochemical technologies were considered. Nutrients in stormwater, groundwater, and wastewater can be removed using physicochemical processes, such as activated carbon absorption, ion exchange with synthetic resins, reverse osmosis, and electrodialysis (ED). Absorption is a physical or chemical process in which ions enter into the bulk phase, either a solid or liquid material. This is a different process than adsorption. Ions are taken by bulk volume in absorption, whereas ions accumulate on the surface of solids in adsorption. In the process of sorption, adsorption and absorption take place simultaneously. Ion exchange is a process of purification, demineralization, or decontamination of aqueous solutions or drinking water by polymer or mineral ion exchange (i.e., exchange of an ion in liquid phase for an ion in solid phase). Reverse osmosis uses pressure to pass contaminated solution through a membrane; the pure solution reaches the far side of the membrane while the contaminants collect on the near side of the membrane. In ED, used for desalination or deionization, ions in the solution pass through an ion exchange membrane to another solution under the influence of an electric potential difference. However, most of these technologies are not cost effective for treating stormwater, which is characterized by large volume flow rates and low concentrations over short time periods. In this study, we deemed an appropriate media mix to be any kind of multifunctional material or functionalized sorption media that could be used cost effectively in both natural and built environments to improve physicochemical and microbiological processes for nutrient removal (Chang et al., 2008).

The ability of bioinfiltration to remove nutrients from stormwater depends on a combination of vegetation and soil effects and is influenced by soil adsorption and uptake in vegetative root zones. After treatment, the treated stormwater percolates to the ground through the vadose zone. Bioinfiltration with different filter media has been gaining popularity due to its cost effectiveness. In bioinfiltration, two important processes that transform ammonia to nitrogen gas are nitrification by autotrophic bacteria and denitrification by either autotrophic or heterotrophic bacteria. Carbon is generally used as a cell building block in both autotrophic and heterotrophic bacteria. Autotrophic bacteria generally derive their cell carbon from carbon dioxide (i.e., inorganic carbon sources), whereas heterotrophic bacteria derive cell carbon from organic carbon sources.

Nitrification involves two steps: ammonia is transformed to nitrite with the help of *Nitrosomonas* bacteria, and nitrite is transformed to nitrate with the help of *Nitrobactor* bacteria in an aerobic environment. In nitrification, ammonium and nitrite are electron donors and oxygen is an electron acceptor. In denitrification, nitrate is transformed to nitrogen gas by heterotrophic or autotrophic bacteria under anoxic conditions. If heterotrophic bacteria are prevalent, organic compounds that exhibit carbonaceous biochemical oxygen demand (CBOD) are electron donors and nitrate is an electron acceptor. BOD is the amount of oxygen required for the biological decomposition (i.e., chemical or biological transformation) of organic waste. The term "biochemical" in this context means biological actions that cause chemical change. This process creates CO_2 and NH_3 through chemical reaction. To determine CBOD, a nitrification inhibitor such as thiourea or allylthiourea is added to represent the amount of oxygen required for biological conversion of only carbonaceous organic matter into cell tissue. As the denitrification system requires anoxic conditions, the oxygen acts as an inhibitor in the process. Skerman and MacRae (1972), Terai and Mori (1975), Nelson and Knowles (1978), and Dawson and Murphy (1972) investigated this issue (Metcalf & Eddy, Inc., et al., 2002). All of these researchers noted that a dissolved oxygen (DO) level greater than 0.2 mg/L can halt the denitrification process. Thus denitrification is very sensitive to DO.

Nutrient removal treatment using sorption media in stormwater retention and detention ponds normally occurs in natural or semi-built environments. Ammonification and nitrification occur simultaneously in the filter media when stormwater containing ammonium and biodegradable carbon contacts aerobic soil or media. Denitrification is then achieved by cycling between oxic and anoxic conditions. Either autotrophic denitrification or heterotrophic denitrification may occur depending on the type of sorption media. Some attention was given to autotrophic denitrification systems involving elemental sulfur-based media filters (Zhang, 2002). Sulfur-based denitrification filters may include limestone or oyster shell as a solid-phase alkalinity source to buffer the alkalinity consumption during biochemical denitrification (Zhang, 2002). On the other hand, heterotrophic denitrification systems use solid-phase carbon sources including woodchips (Kim et al., 2003), sawdust (Kim et al., 2003), cardboard (Greenan et al., 2006), paper (Kim et al., 2003), and agricultural residue (Kim et al., 2003; Greenan et al., 2006; Della Rocca et al., 2005). Some proprietary media mixes containing woodchips and other materials have also been developed (Lombardo, 2005; Chang et al., 2008c).

Cellulose-based systems using palm tree or coconut shell and lignin-based systems using wood chips or sawdust are the most common heterotrophic denitrification filter technology, although elemental sulfur (autotrophic denitrification) may also be used as an electron donor. In lignocellulosic materials such as wood chips and sawdust, facultative heterotrophs may quickly degrade the organic carbon and deplete the oxygen. However, this simultaneous process with intermittent cycling between oxic and anoxic conditions may be sustainable for denitrification as it maintains lower oxygen requirements. It also recycles the alkalinity in denitrification required for nitrification. During this stage, ammonia may be retained in the filter media depending on the cation exchange capacity because ammonium cannot be nitrified under anoxic conditions. The adsorbed ammonia will be nitrified when the next storm event raises the DO level, changing conditions in the soil or media from anoxic to oxic. Ultimately the amount of denitrification may be limited by the frequency and duration of the oxic/anoxic fluctuations within the filter with respect to the reaction rates or dosing conditions in the treatment during intermittent storm events.

Simultaneous nitrification and denitrification processes in cellulose-based systems of stormwater ponds may occur intermittently. The two steps in the oxidation of ammonia can be summarized in Eqs. (1)–(3) (Metcalf and Eddy, 2003):

$$2NH_4^+ + 3O_2 \rightarrow 2NO_2^- + 4H^+ + 2H_2O \tag{1}$$
$$2NO_2^- + O_2 \rightarrow 2NO_3^- \tag{2}$$

The overall nitrification reaction is

$$NH^+_4 + 2O_2 \rightarrow NO_3^- + 2H^+ + H_2O \tag{3}$$

and the denitrification of wastewater is shown in Eq. (4) (Metcalf and Eddy, 2003):

$$C_{10}H_{19}O_3N + 10NO_3^- \rightarrow 5N_2 + 10CO_2 + 3H_2O + NH_3 + 10OH^- \tag{4}$$

Both pH and temperature also have important impacts on these two biological processes. A pH range of 7.00–8.00 is considered good for both nitrification and denitrification (Metcalf and Eddy, 2003). Nitrifiers can grow best in a temperature range of 35–42°C, while denitrifiers work well in the range of 10–25°C (Rittmann, 2000; USEPA, 1993).

Technologies to remove phosphorus include chemical precipitation, biological treatment, crystallization, ion exchange, magnetic separation, adsorption/absorption, tertiary filtration, and sludge treatment (Lazaridis, 2003). Most phosphorus can be removed from stormwater by both precipitation and absorption processes. Precipitation is the formation of a solid in a solution as the result of a chemical reaction; the solid settles to the bottom for pH > 7.00. The precipitation reactions of phosphorus with calcium, iron, and aluminum are given below in Eqs. (5)–(7). The end product in a calcium reaction is hydroxylapatite, and the end product in an iron reaction is insoluble ferric phosphate (Metcalf and Eddy, Inc., 2002).

$$10Ca^{2+}+6PO_4^{3-}+2OH^- \leftrightarrow Ca_{10}(PO_4)_6(OH)_2 \quad (5)$$
$$Fe^{3+}+H_nPO_4^{3-n} \leftrightarrow FePO_4+nH^+ \quad (6)$$
$$Al^{3+}+H_nPO_4^{3-n} \leftrightarrow AlPO_4+ nH^+ \quad (7)$$

The geochemical process of crystallization converts the thickened concentration into crystals, which is followed by dewatering in a centrifuge. Tertiary filtration is a physicochemical process that involves removing solids from the secondary treated effluent. Sludge treatment refers to the management and disposal of the sludge produced during the wastewater treatment process. Some functionalized sorption media used for phosphorus removal are sand rich in Fe, Ca, or Mg, as well as gravel, limestone (sedimentary rock largely composed of calcium carbonate, $CaCO_3$), shale (fine-grained sedimentary rock composed mostly of clay minerals), light-weight aggregates, zeolite (natural mineral or artificially produced hydrated aluminosilicates with a microporous structure), pelleted clay (alone or in combination with soil), opaka (siliceous sedimentary rock), pumice (natural porous volcanic rock with an average porosity of 90% that generally floats on water), wollastonite (white mineral containing calcium and ferrous metasilicate produced from impure limestone under high temperature and pressure), fly ash (residue generated from coal combustion), blast furnace slag (porous non-metallic byproduct of the iron and steel industry), alum (hydrated aluminum potassium sulfate, $KAl(SO_4)_2 \cdot 12H_2O$), goethite (hydrous ferric oxide found in soil), hematite (black or reddish-brown mineral form of iron(III) oxide, Fe_2O_3), dolomite (sedimentary carbonate rock or mineral composed of calcium magnesium carbonate, $CaMg(CO_3)_2$), and calcite (carbonate mineral) (Korkusuz et al., 2007).

8.3.3 Nutrient Removal in Stormwater Runoff by Sorption Media

Various types of sorption media may be used to treat stormwater, wastewater, groundwater, landfill leachate, and drinking water sources. Key studies by Richman (1997), DeBusk et al. (1997), Kim et al. (2000), Clark et al. (2001), Tshabalala (2002), Boving and Zhang (2004), Hsieh and Davis (2005), Birch et al. (2005), Analytical and Environmental Consultants (AEC, 2005), Ray et al. (2006), and Seelsaen et al. (2006) demonstrated successful uses of sorption media to remove nutrients. These tests found that nutrients can be removed by adsorption and biological nitrification/denitrification processes. The sorption materials involved included compost, peat, sand, wollastonite, limerock, alfalfa, sawdust, newspaper, wheat straw, wood chips, lignocellolosic materials, aspen wood fibers, mulch, hardwood mulch, fine and coarse glass, and clinoptilolite. In addition to nutrients, sorption media can also remove significant amounts of solids, BOD material, and heavy metals from stormwater runoff. Table 1 summarizes the removal efficiency of nutrients from stormwater using sorption media.

In Table 8.1, TP (i.e., 70%–90% OP) is the sum of all phosphorus types, both dissolved and particulate (suspended). The dissolved and particulate portion can be separated using a filter with a nominal pore size of 2.0 μm (APHA, 1995). The dissolved portion passes through the filter, and the particulate portion is retained on the filter. OP can exist in the form of PO_4^{3-}, hydrogenophosphate (HPO_4^{2-}), dihydrogen phosphate ($H_2PO_4^-$), or phosphoric acid (H_3PO_4) (Metcalf and Eddy, 2003). OP is also known as soluble reactive phosphorus. TKN is the sum of the organic nitrogen, ammonia in gaseous form, and ammonium ions. Total nitrogen (TN) is the sum of all organic and inorganic nitrogen species. Organic nitrogen includes proteins, urea, amino acids, and the nitrogen found in decayed plant and animal tissues. Inorganic nitrogen is NH_3, NH_4^+, NO_3-, and NO_2^-.

Table 8.1: Nutrient removal efficiency from stormwater by various sorption media

Year	Researcher	Sorption media	Mechanism	Removal efficiency
1997	Richman	compost	adsorption	90.00% solids, 85.00% oils and greases (O/G), and 82.00–98.00% heavy metals
1997	Debusk et al.	wollastonite	adsorption	87.80% TP, 81.40% Cu, 97.70% Cd, and 80.30% Ni
		limerock		41.50% TP, 32.20% Cu, 81.34% Cd, and 31.30% Ni
		peat		44.00% TP, 41.20% Cu, 97.80% Cd, and 92.30% Ni
		sand		41.50% TP, 76.95% Cu, 94.40% Cd, and 77.67% Ni
2000	Kim et al.	sulfur	nitrification and denitrification	
		alfalfa		100.00% nitrate
		leaf mulch compost		60.00% nitrate
		newspaper		100.00% nitrate
		sawdust		>95.00% nitrate
		wheat straw		>95.00% nitrate
		wood chips		>95.00% nitrate
2002	Tshabalala	lignocellulosic materials	adsorption	82.00% dichlobenil (DBN)
				92.00% chlorothalonil (CTL)
				96.00% chlorpyrifos (CPS)
2004	Boving and Zhang	aspen wood fibers	adsorption	60.00% anthracene
				89.00% pyrene
				36.00% fluorene
2005	Hsieh and Davis	100% sand	adsorption	96.00% TSS, 96.00% O/G, 98.00% lead, 85.00% TP, 11.00% nitrate, and 8.00% ammonia

		100% sand		96.00% TSS, 96.00% O/G, 96.00% lead, 10.00% TP, 1.00% nitrate, and 15.00% ammonia
		2% mulch, 93% soil, 5% sand		29.00% TSS, 96.00% O/G, 98.00% lead, 47.00% TP, 1.00% nitrate, and 6.00% ammonia
		2% mulch, 93% soil, 5% sand		88.00% TSS, 96.00% O/G, 98.00% lead, 41.00% TP, 14.00% nitrate, and 24.00% ammonia
		2% mulch, 93% soil, 5% sand		91.00% TSS, 96.00% O/G, 98.00% lead, 48.00% TP, 8.00% nitrate, and 16.00% ammonia
		91% mulch, 9% sand		86.00% TSS, 96.00% O/G, 75.00% lead, 4.00% TP, 43.00% nitrate, and 16.00% ammonia
		100% sand		96.00% O/G, 66.00% lead, 84.00% TP, 13.00% nitrate, and 5.00% ammonia
		3% mulch, 97% sand		96.00% TSS, 96.00% O/G, 98.00% lead, 61.00% TP, 9.00% nitrate, and 9.00% ammonia
		2% mulch, 21% soil, 77% sand		66.00% TSS, 96.00% O/G, 98.00% lead, 47.00% TP, 3.00% nitrate, and 2.00% ammonia
		8% mulch, 26% soil, 66% sand		94.00% TSS, 96.00% O/G, 98.00% lead, 50.00% TP, 4.00% nitrate, and 7.00% ammonia
		6% mulch, 32% soil, 62% sand		93.00% TSS, 96.00% O/G, 98.00% lead, 39.00% TP, 4.00% nitrate, and 7.00% ammonia
		24% soil, 76% sand		93.00% TSS, 96.00% O/G, 98.00% lead, 39.00% TP, 2.00% nitrate, and 5.00% ammonia

		3% mulch, 43% soil, 54% sand		96.00% TSS, 96.00% O/G, 98.00% lead, 83.00% TP, 13.00% nitrate, and 26.00% ammonia
		3% mulch, 24% soil, 73% sand		96.00% TSS, 96.00% O/G, 98.00% lead, 57.00% TP, 24.00% nitrate, and 17.00% ammonia
		11% mulch, 19% soil, 70% sand		96.00% TSS, 96.00% O/G, 98.00% lead, 54.00% TP, 27.00% nitrate, and 20.00% ammonia
		2% mulch, 17% soil, 81% sand		96.00% TSS, 96.00% O/G, 97.00% lead, 24.00% TP, 6.00% nitrate, and 11.00% ammonia
		2% mulch, 72% soil, 26% sand		92.00% TSS, 96.00% O/G, 98.00% lead, 72.00% TP, 9.00% nitrate, and 19.00% ammonia
		2% mulch, 49% soil, 49% sand		92.00% TSS, 96.00% O/G, 98.00% lead, 71.00% TP, 9.00% nitrate, and 19.00% ammonia
2005	Birch et al.	1:6 mixture of zeolite (clinoptilolite) and coarse, pure quartzitic sand	adsorption	47.00–74.00% TKN, 33.00–40.00% TN, 37.00–67.00% TP, 49.00–81.00% Cu, 88.00–98.00% Pb, −1.00–77.00% Zn, 10.00% Cr, −213.00–38.00% Fe, and 20.00–88.00% TSS
2006	Ray et al.	hardwood mulch	adsorption (2 hour HRT)	85.00% Cu, 75.00% Cd, 21.00% Cr, 90.00% Pb, 60.00% Zn, 63.00% dichlorobenzene, 63.00% naphthalene, 89.00% fluoranthene, 90.00% butybenzylphthalate, and 80.00% benzopyrene

			adsorption (4 hour HRT)	85.00% Cu, 83.00% Cd, 26.00% Cr, 85.00% Pb, 72.00% Zn, 71.00% dichlorobenzene, 65.00% naphthalene, 95.00% fluoranthene, 95.00% butybenzylphthalate, and 84.00% benzopyrene
			adsorption (72 hour HRT)	85.00% Cu, 86.00% Cd, 68.00% Cr, 92.00% Pb, 72.00% Zn, 100.00% dichlorobenzene, 88.00% naphthalene, 93.00% fluoranthene, 77.00% butybenzylphthalate, and 92.00% benzopyrene
		fine glass		68.00% Zn and 40.00% Cu
		sand		15.00% Zn and 30.00% Cu
		coarse glass		15.00% Zn and 28.00% Cu
		ash		50.00% Zn and 97.00% Cu
		zeolite		97.00% Zn and 50.00% Cu
		compost		97.00% Zn and 90.00% Cu
2006	Seelsaen et al.	packing wood	adsorption	88.00% Zn and 84.00% Cu
2006	Huang et al.	clinoptilolite	ion exchange	100.00% Fe

8.4 MATERIALS AND METHODS

8.4.1 Physical Properties of Sorption Media

The following six criteria were often considered to screen possible filter media to be used in BMPs: 1) the relevance of nitrification and/or denitrification processes; 2) the hydraulic conductivity or permeability; 3) the cost; 4) the removal efficiency reflected in the literature with regard to adsorption, precipitation, and filtration capacity; 5) the availability in Florida; and 6) additional environmental benefits. Sand, tire crumb, and sawdust were selected for this study based on these criteria. The final composition of the filter media mixture for this demonstration was 50% sand (masonry sand in natural soil, 25% retained on a number 140 sieve and 25% retained on a number 200 sieve), 30% tire crumb, and 20% sawdust by weight. The natural soil was collected from a stormwater dry pond (Hunter's Trace) in Ocala, Marion County, Florida, located at coordinates 29°11'49.42"N, 82°3'52.83"W, as shown in Figure 8.3.

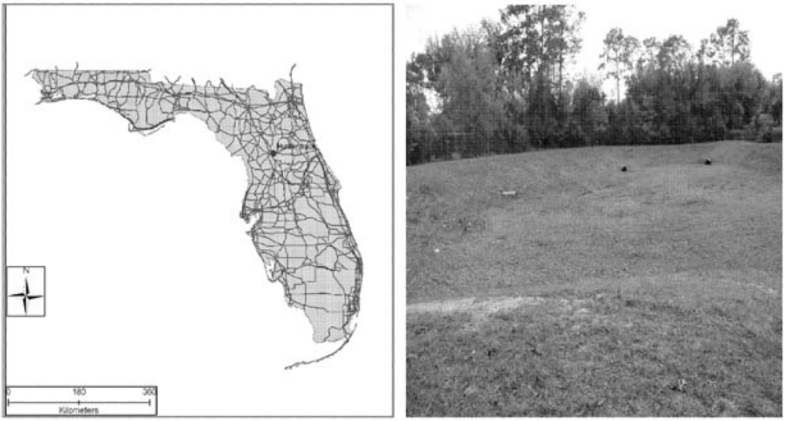

Figure 8.3: Location and photograph of the Hunter's Trace pond

It is important to understand the physical properties of the filter media, including the particle-size distribution, density, void ratio, porosity, specific gravity, surface area, and hydraulic conductivity. American Society for Testing and Materials (ASTM) procedures were used to determine the particle-size distribution, specific gravity, and hydraulic conductivity (ASTM, D421-85, D854-92, D2434-68). The surface area of the sorption media mixture was determined using the multipoint Brunauer, Emmett, and Teller (BET) method with nitrogen adsorption at 77 K, determined using the vacuum volumetric method of Quanta Chrome Instruments, Boynton Beach, Florida. The void ratio is the volume of the voids divided by the volume of the solid. The porosity is the volume of the voids divided by the total volume. These properties are used to determine the hydraulic residence or retention time (HRT) and the adsorption area available for the nutrients. The particle-size distribution curve can suggest the grain size and type of distribution of particles in a certain soil or sorption media sample. The effective size (D_{10}) is calculated from the particle-size distribution as the diameter in millimeters at which 10.00% of particles are finer. The porosity gives some idea about the volume of voids in a sorption media sample, and the amount of water that actually comes in contact with the media can be determined from the void volume. The porosity is calculated from the specific gravity and void ratio. These properties can influence dispersion phenomena in the soil or the filter media column test. The surface area is also important for nutrient removal as greater surface area will remove more nutrients from the stormwater. The void ratio and porosity can be calculated by Eqs. (8) and (9):

$$\text{Void ratio of filter media mixture (E)} = \frac{G_S * \rho_W}{\rho_d} - 1 \qquad (8)$$

$$\text{Porosity of filter media mixture (N)} = \frac{E}{1+E} \qquad (9)$$

where G_s is the specific gravity of the filter media mixture, ρ_w is the density of water, ρ_d is the density of the filter media mixture, E is the void ratio, and N is the porosity.

8.4.2 Experimental Setup of the Column Study

The column test was designed to determine the nutrient removal performance of the selected green sorption media in saturated and unsaturated conditions that mimic the field condition of stormwater dry ponds. Figure 8.4 shows the two pairs of plexiglass columns that were prepared in the University of Central Florida laboratory to represent the two-stage flow pathway by which pond water infiltrates into the vadose zone (stage 1, unsaturated) and then percolates into the sorption media layer placed between the groundwater table and the capillary zone (stage 2, saturated) to mimic th egepphysical environment beneath the pond as shown in Figure 2(b). The columns were 182.88 cm (6 ft) long, with an inner diameter of 14.73 cm (5.8 in) and a wall thickness of 0.51 cm (0.2 in). All four columns were attached to wooden frames with fabric straps. The wooden frames were attached to a steel rack in the laboratory. Each column had three sampling ports. The first port was 39.37 cm (15.50 in) from the bottom of the column, the second port was 44.45 cm (17.5 in) above the first, and the third port was 38.10 cm (15.00 in) above the second. Pipe thread sealant was used to make all the joints in the columns leak proof. The top and bottom of each column were closed but had removable screw caps through which to add and remove media.

A plastic mesh filter with gravel was placed at the bottom of each column to prevent the outward flow of finer particles from the columns during sample collection. All the columns were filled with sand or sorption media to a height of 121.92 cm (48 in). The fill line was slightly below the first port and 38.10 cm (15 in) above the second port. The first pair of columns was filled with natural soil collected at the Hunter's Trace pond as a control case. In the second pair, the first column was filled with natural soil, and the second column was filled with the sorption media mixture. The natural soil was sun dried, and impurities were removed with a number 10 sieve. The control case natural soil was compacted to a density of 516.00 kg/m^3 (106.00 lb/ft^3), and the media mixture was compacted to a density of 204.45 kg/m^3 (42.00 lb/ft^3). In each pair, the first column was considered to be the unsaturated (vadose) zone, and the second column was considered to be the saturated zone.

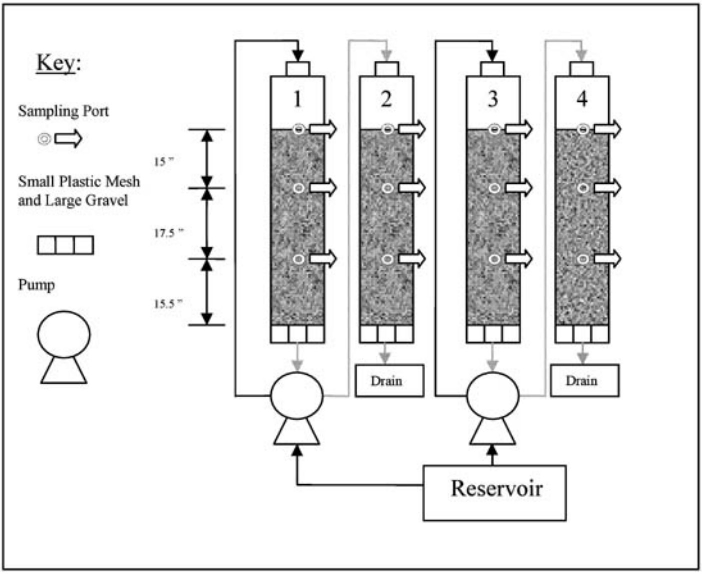

Figure 8.4: Schematic diagram of the column setup for the laboratory experiment

Water was pumped from the unsaturated column using a peristaltic pump. The flow rate in all the columns was 10 mL/min, equivalent to 1.38 in/hr. Typically, detention pond infiltration ranges from 2.54–5.08 cm/hr (1–2 in/hr). Stormwater was supplied from a 25-gallon reservoir. The initial nitrate concentration in the stormwater was set to 0.40, 1.25, and 2.50 mg/L in three successive experiments, and the initial orthophosphate concentration was set to 0.13, 0.36, and 0.79 mg/L. These nitrate and phosphorus concentrations were higher than the actual concentrations given in the above literature review section. The stock solutions of nitrate and phosphorus were prepared from potassium nitrate and potassium phosphate according to standard methods (APHA, 1995). The removal efficiencies of TN, nitrate, nitrite, ammonia, and OP by the sorption media were measured by the Hach methods listed in Table 8.2 below. The detention time was calculated by

$$T_d = V/Q \qquad (10)$$

where T_d is the HRT, V is $(3.14\ n)(d_{inside}^2)/4$, d_{inside} is the inside diameter of column, n is the porosity; and Q is the flow rate.

Table 8.2: Column study water quality parameters and methods

Parameter	Method	Range*
nitrates and nitrites	Hach method 8192	0.01–0.50 mg/L NO_3–N
nitrites	Hach method 8507	0.002–0.30 mg/L NO_2–N
ammonia	Hach method 8155	0.01–0.50 mg/L NH_3–N
total nitrogen	Hach Method 10071	0.50–25.00 mg/L N
reactive/orthophosphate	Hach method 8048	0.02–2.50 mg/L PO_4^{3-}
total phosphorus	Hach method 8190	0.06–3.50 mg/L PO_4^{3-}

*Note: In some cases, samples were diluted with deionized water to fall within this range.

8.5 RESULTS AND DISUCSSION

8.5.1 Physical Properties of Sorption Media

Table 8.3 shows the physical properties of the natural soil and sorption media mixture. The hydraulic conductivity of the moist sample of Hunter's Trace soil and the sorption media were measured as 4.470 cm/hr (1.759 in/hr) and 4.380 cm/hr (1.724 in/hr), respectively. The Hunter's Trace soil contained small clay particles, which created a surface area larger than that in the sorption media with larger particles such as sawdust and tire crumb. A larger surface area means greater removal efficiency because there is more solid-phase area to adsorb nutrients. As noted above, finer particles may increase the HRT and decrease the amount of solids in the effluent. The porosity of the sorption media was greater than that of the natural soil.

Figures 8.5 and 8.6 give particle-size distribution curves for the natural soil and the sorption media. The control sample (natural soil) was well graded, but the sorption media mixture was not. The effective sizes (D_{10}) of the natural soil and sorption media were 0.17 mm and 0.08 mm, respectively, as calculated from the particle-size distribution curve. These D_{10} values can be used to determine the hydraulic conductivity using an empirical equation (i.e., $k = 1.0 \times D_{10}^2$; Das, 2002).

Table 8.3: Physical properties of natural sand and sorption media.

	Hunter's Trace (dry sample)	Hunter's Trace (moist sample)	Sorption media
dry density (g/cm^3)	1.560	1.730	1.41
void ratio	0.670	0.510	0.56
Porosity	0.400	0.340	0.36
specific gravity	2.620	2.620	2.19
surface area (m^2/g)	3.111	3.111	0.129
intrinsic conductivity (cm/hr)	62.48	4.470	4.38

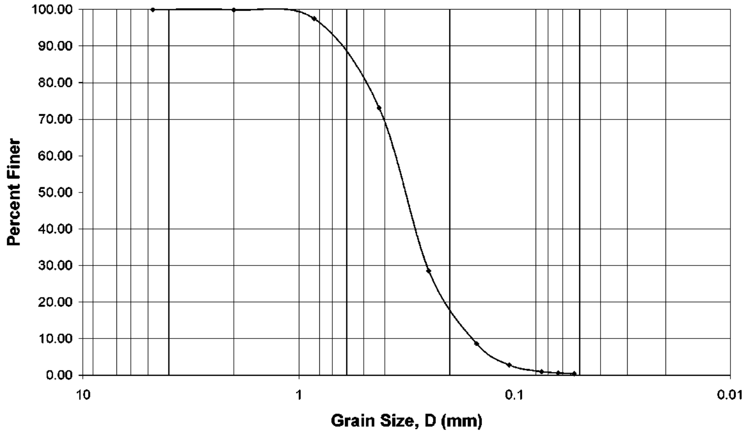

Figure 8.5: Particle-size distribution of natural soil collected from Hunter's Trace pond

8.5.2 Experimental Results of the Column Study

At the beginning of the experiment, the sorption media mixture contributed some nutrient (ammonia) and color in the effluent stormwater from the column due to the inclusion of sawdust. But with time, both the color and contributed ammonia diminished and the sorption media started to remove all the nutrients. The pH of the effluent was in the range of 7.00–8.00, and the temperature was maintained at 22.0–23.0°C throughout the experiment.

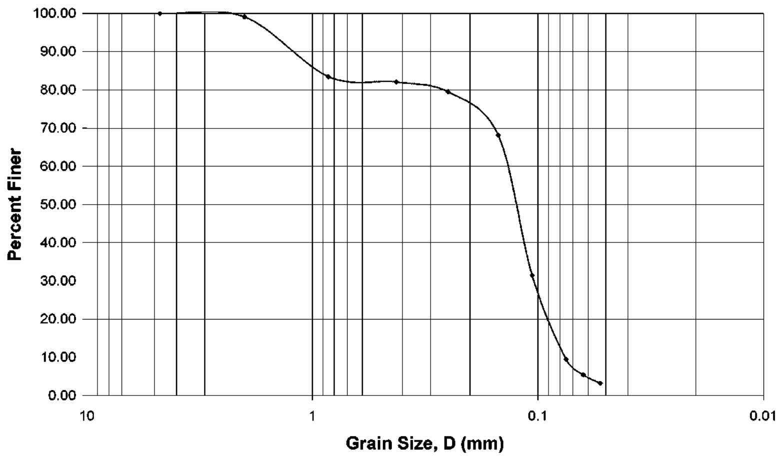

Figure 8.6: Particle-size distribution of filter media mixture

Nitrate removal

Table 8.4 presents the nitrate removal efficiencies with initial nitrate concentrations of 0.4, 1.25, and 2.53 mg/L in both the saturated and unsaturated columns. In particular, the media mixture with an initial concentration of 2.53 mg/L removed approximately 99.20% of the NO_3-N compared to the natural soil that removed only 39.50% of the NO_3^--N with an HRT of 4.00 hours. This result was very similar to a previous study of batch-fed augmented stormwater with an initial nitrate concentration of 2.5 mg/L in a 30.48-cm (12-in) column. In that experiment, the nitrate removal efficiency was 90.28% and 90.83% with HRTs of 3 and 5 hours, respectively (Chang et al., 2008b).

A possible explanation for the lower removal of nitrate in the control case is as follows. Both soil and nitrate are negatively charged (Kim et al., 2000) and may repel each other when soil and stormwater come in contact. During the adsorption process, the surface charge may change with the pH; thus, in some cases, the surface charges of the soil particles change and adsorb some of the nitrate ions. As noted, the removal efficiency also depends on the porosity of the sorption media. Both sawdust and tire crumb have larger porosities, and these two media can accelerate the removal efficiency via absorption. In summary, the effluent nitrate concentration (0.021 mg/L) from the sorption media column met the USEPA MCL requirement, although that from the natural soil column (Hunter's Trace soil) did not.

Table 8.4(a): Nitrate removal efficiency in the two-stage system with approximate nitrate influent concentration of 0.40 mg/L NO_3^--N

Control Run	Initial Concentration (mg/L NO_3-N)	Final Concentration (mg/L NO_3-N)	Removal Efficiency (%)
1	0.382	0.294	23.0
2	0.382	0.266	30.2
3	0.382	0.139	63.6
Average	0.382	0.233	38.9
Media Run	Initial Concentration (mg/L NO_3-N)	Final Concentration (mg/L NO_3-N)	Removal Efficiency (%)
1	0.382	0.021	94.5
2	0.382	0.022	94.1
3	0.382	0.023	94.0
Average	0.382	0.022	94.2

Table 8.4(b): Nitrate removal efficiency in the two-stage system with approximate nitrate influent concentration of 1.25 mg/L NO_3^--N

Control Run	Initial Concentration (mg/L NO_3-N)	Final Concentration (mg/L NO_3-N)	Removal Efficiency (%)
1	1.269	0.312	75.4
2	1.269	0.391	69.2
3	1.269	0.438	65.4
Average	1.269	0.380	70.0
Media Run	Initial Concentration (mg/L NO_3-N)	Final Concentration (mg/L NO_3-N)	Removal Efficiency (%)
1	1.269	0.023	98.2
2	1.269	0.022	98.2
3	1.269	0.023	98.2
Average	1.269	0.023	98.2

Table 8.4(c): Nitrate removal efficiency in the two-stage system with approximate nitrate influent concentration of 2.50 mg/L NO_3^--N

Control Run	Initial Concentration (mg/L NO_3-N)	Final Concentration (mg/L NO_3-N)	Removal Efficiency (%)
1	2.529	1.615	36.1
2	2.529	1.508	40.4
3	2.529	1.463	42.1
Average	2.529	1.529	39.5
Media Run	Initial Concentration (mg/L NO_3-N)	Final Concentration (mg/L NO_3-N)	Removal Efficiency (%)
1	2.529	0.021	99.2
2	2.529	0.021	99.2
3	2.529	0.021	99.2
Average	2.529	0.021	99.2

Orthophosphate (OP) removal

Table 8.5 shows the total OP removal efficiency for initial concentrations of 0.36 and 0.79 mg/L PO_4-P. In the latter case, the OP removal efficiency was about 55.20% by the soil and 91.40% by the sorption media. In the former case, the performance of natural soil was relatively lower. In general, natural soil performs better for OP removal than for nitrate removal. Clay particles have some affinity for phosphorus ions (Gisvold et al., 2000), and the particle-size distribution curve in Fig. 3 reveals that the Hunter's Trace soil had some clay particles. These clay particles undoubtedly contributed to the phosphorus removal from stormwater. The final effluent concentrations of phosphorus met the USEPA requirements of less than 0.1 mg/L after passing through the two-stage treatment, but not after passing through the natural soil column. This finding confirms the necessity of an underground sorption media layer. According to Table 2, a larger surface area (as in natural soil) resulted in a smaller amount of nutrients being removed. This negative relationship also strongly supports the use of sorption media for nutrient removal.

Table 8.5(a): Orthophosphate removal efficiency in the two-stage system with approximate PO_4-P influent concentration of 0.36 mg/L PO_4-P

Control Run	Initial Concentration (mg/L PO_4-P)	Final Concentration (mg/L PO_4-P)	Removal Efficiency (%)
1	0.361	0.293	18.8
2	0.361	0.285	21.0
3	0.361	0.302	16.3
Average	0.361	0.294	18.7
Media Run	Initial Concentration (mg/L PO_4-P)	Final Concentration (mg/L PO_4-P)	Removal Efficiency (%)
1	0.361	0.043	88.2
2	0.361	0.077	78.8
3	0.361	0.031	91.4
Average	0.361	0.050	86.1

Table 8.5(b): Orthophosphate removal efficiency in the two-stage system with approximate PO_4-P influent concentration of 0.79 mg/L PO_4-P

Control Run	Initial Concentration (mg/L PO_4-P)	Final Concentration (mg/L PO_4-P)	Removal Efficiency (%)
1	0.785	0.339	56.8
2	0.785	0.358	54.3
3	0.785	0.357	54.5
Average	0.785	0.351	55.2
Media Run	Initial Concentration (mg/L PO_4-P)	Final Concentration (mg/L PO_4-P)	Removal Efficiency (%)
1	0.785	0.099	87.4
2	0.785	0.048	93.9
3	0.785	0.057	92.7
Average	0.785	0.068	91.4

8.6 CONCLUSIONS

Stormwater retention and detention ponds are designed to store water during a wet season and to maintain an artificial hydrologic balance to meet groundwater recharge during a dry season. As clearly demonstrated above, the use of sorption media is a promising way to remove nutrients from stormwater runoff through various design strategies. The life expectancy of the sorption media and practical installation can be a challenging task for engineers attempting to integrate sorption media with other system components. Even so, use of sorption media to clean stormwater runoff offers cost-effective option and will play a vital role in urban water resources management and sustainability.

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CHAPTER 9

Comparative Assessment of Two Standard Septic Tank Drain Fields Using Different Sands with Recirculation for Nutrient Removal

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ABSTRACT: On-site wastewater treatment systems (OWTSs) using a septic tank followed by a recirculating sand filter (RSF) with effluent discharged to an unlined standard drain field are now typical performance-based treatment facilities for nutrient removal. The types of effluent distribution in standard drain field systems include gravity systems, low-pressure dosed systems, and drip irrigation systems. Within the gravity systems, the standard drain field may be constructed of a series of parallel, underground, perforated pipes with slopes, allowing the septic tank effluent to percolate into the surrounding soil in the vadose (unsaturated) zone, where most of the residual nutrients can be assimilated. The inclusion of an RSF may improve the nitrification to some extent, promoting denitrification in the drain field finally. With such designs, most residual nutrients in wastewater are expected to be consumed as the wastewater passes through the soil. The aim of this chapter is to present the effectiveness of an RSF and a comparative study with two different drain field sands for nutrient removal in the vadose zone. They were astatula sand (i.e., citrus grove sand) and washed building sand, popular in central Florida, for drain field use. The real-time polymerase chain reaction (PCR) was applied to prove that the nitrification and denitification processes can be sustained naturally as expected. The use of such a gene identification method is novel in supporting the biological removal. Research findings show that the performance of washed building sand and astatula sand in terms of ultimate nutrient removal is about the same. Because astatula sand is less costly, it may be an appropriate replacement for the more expensive washed building sand.

9.1 INTRODUCTION

Both decentralized, on-site wastewater treatment systems (OWTSs) and centralized, public owned wastewater treatment systems (POWTSs) handle the needs for wastewater disposal. When urban regions gradually expand due to regional development, centralized sewage collection, treatment, and disposal are often unavailable for both geographic and economic reasons. Thus, OWTSs with the use of septic tanks may be necessary to protect public health in many rural areas. The septic tank system was developed in 1860 in France by John Mouras (Crites and Tchobanoglous, 1998). Since then, many aspects of its design have changed to reach the present form. Septic tanks made their way to the USA and to some parts of Africa because of the British navy around 1883 (Crites and Tchobanoglous, 1998). Later, septic tank systems became very common in both developed and developing countries. More than 25 million homes, or 25% of the US population, use an OWTS to meet their wastewater treatment and disposal needs (US Environmental Protection Agency; USEPA, 2002). According to USEPA, the number of systems increases significantly each year (USEPA, 2003). The construction and maintenance costs of OWTS may be lower than those for POWTS. Furthermore, the use of OWTSs can avoid potentially large transfers of wastewater from one watershed to another via centralized collection pipelines and treatment facilities, allowing maximum flexibility in planning for future growth. Indeed, the EPA has stated that, "adequately managed, decentralized wastewater treatment systems are a cost-effective and long-term option for meeting public health and water quality goals (USEPA, 2003)." However, the USEPA estimates that failures of OWTSs nationally result in the annual discharge of 920 billion liters (250 billion gallons) of improperly treated wastewater to ground and surface waters (National Environnemental Services Center; NESC, 2005).

On average, medium-strength untreated domestic wastewater (460 L/capita × day) has 40 mg/L of total nitrogen (TN), 25 mg/L of ammonia (NH_3-N), 720 mg/L of total solids (TS), 190 mg/L of biochemical oxygen demand (BOD_5), 7 mg/L of total phosphorus (TP), and 90 mg/L of oil and grease (Metcalf and Eddy, Inc., 2002). TN is the sum of organic nitrogen, ammonia-nitrogen, nitrate-nitrogen, and nitrite-nitrogen. TS is the volume of solids remaining after water has been evaporated by drying at 103–105°C. TP is the amount of all forms of phosphorus (dissolved and particulate) available in a sample. Nationwide, wastewater effluent from OWTS can represent a large fraction of nutrient loads in groundwater aquifers.

Ammonia, nitrite, nitrate, and phosphorus all have harmful effects on humans, marine life, and the aquatic ecosystem generally. Nitrates in drinking water, for which groundwater aquifers serve as the main source, have been associated with methemoglobinemia (MHB). Commonly known as "blue-baby" syndrome, MHB affects infants under 6 months of age and is a public health concern. The most characteristic symptom is the ashen, bluish (cyanotic) hue to the skin and nails. High ammonia concentrations in bodies of water increase fish mortality and affect reproduction. Phosphorus can trigger eutrophication in surface water bodies. On the other hand, nutrient fluxes may follow an existing pathway, getting into the surface water system. Additionally, some aquifers may discharge into springs or other surface waters, adversely affecting them.

Decentralized wastewater treatment has been accepted fully as a permanent part of the wastewater infrastructure in the US relatively recently (USEPA, 1997). According to the 1990 United States Census, approximately 26% of Florida's population was served by OWTS. Since that time, approximately 40,000 new systems have been installed each year. More than 1.8 million systems were estimated to be in use statewide in 1999 (Department of Community Affairs; DCA et al., 1999).

Regulation of septic tanks began in Florida in the 1920s because of frequent occurrences of water-borne diseases (DCA et al., 1999). It has evolved extensively over the years, but the primary focus is protecting human health, not water quality. Over the past 25 years, numerous changes have been made to Florida's septic tank regulations (Chapter 10D-6, F.A.C.). These regulations have required increased setback distances from surface waters, have increased minimum separation between the bottom of the drain field and the seasonally high ground-water table, and have limited the amount of wastewater discharged per acre (DCA et al., 1999). However, many older systems have been exempted from those changes. It was estimated in 1998 that OWTSs discharge 1,710 million liters (450 million gallons) per day of partially treated, non-disinfected wastewater (DCA et al., 1999).

Traditional septic tank systems are not efficient enough to remove nutrients from wastewater to meet the Clean Water Act (CWA) goals (USEPA, 2003). As a result, on-site wastewater effluent disposal has contributed to significant adverse impacts on the dynamics of the natural environment. Two types of OWTS, active and passive treatment technologies, have been developed to reduce nutrient impact. Active OWTS uses external energy and moving mechanical parts to lift and distribute wastewater and to introduce oxygen in the wastewater by an aeration pump to improve the nutrient removal process. According to Florida Department of Health (FDOH), passive OWTS does not use aeration pumps, includes only an effluent dosing pump as the mechanical and moving part, and uses the medium as a nutrient removal system.

The aim of this chapter is two-fold: 1) to present a cost-effective passive OWTS technology that is designed to promote both nitrification and denitrification, and 2) to demonstrate how the use of different types of sand available in Florida in standard drain fields alter performance in removing total nitrogen and total phosphorus. To supplement the routine monitoring and measurement of the nutrient parameters, a quantitative real-time polymerase chain reaction (real time PCR) was used to evaluate nitrogen removal effects of media by quantifying the ammonia-oxidizing bacteria (AOB), nitrite-oxidizing bacteria (NOB), and denitrifying bacteria in the sand filtration tank and drain fields.

9.2 MATERIALS AND METHODS

9.2.1 Technical Background

A conventional OWTS usually includes a septic tank and 1-m wide subsurface trenches dug 0.6–1.0 m deep in 3-m centers. Gravity distribution is used in the drain field, and the size is from 30 to 200 linear meters of trench for a single family home, depending on soil properties. The traditional septic tank is a watertight container made of concrete, fiberglass, or other durable material (Figure 9.1(a)). It provides primary treatment for wastewater as solids settle to the bottom of the tank where they are partially decomposed by bacteria. A layer of soaps, greases, and scum normally floats on the top of the liquid wastewater. The accumulated floating scum and submerged solids must be removed periodically. The liquid wastewater contained in the septic tank, called effluent, enters the next major component of the septic system, the drain field or leach field, which consists of a series of parallel, underground, perforated pipes. This drain field allows wastewater to percolate into the surrounding soil (the vadose zone). Ideally, various physical, chemical, and biological processes reduce nutrients, bacteria, and viruses in the wastewater as the wastewater effluent travels down through the soil layers. Where systems are improperly designed or managed, they may create a higher, undesirable level of nutrient loading (Hoover, 2002).

Figure 9.1(a): A traditional septic tank

The USEPA has suggested that a multiple-chamber tank can perform well (USEPA, 1980). In a dual-chambered septic tank (Figure 9.1(b)), a pipe is fixed in the middle of the wall separating the two chambers. Thus, only the wastewater in the clean zone is allowed to go to the next chamber for further decomposition. The outlet should be designed in such a way that it does not allow sludge or scum to flow out. According to the EPA, the vertical pipe in the outlet should be submerged at about 40% of the liquid depth (USEPA, 1980). It is recommended that a lower diameter pipe be used in the outlet to reduce the velocity near the outlet (USEPA, 1980). A tee or baffle is used in the outlet to ensure that the effluent flowing out is not mixed with scum and sediment. Overall, the septic tank provides the primary treatment, and the drain field provides secondary treatment of the wastewater. The organic materials at the bottom of the tank undergo some decomposition and create stable compounds and gases, including methane (CH_4) and hydrogen sulfide (H_2S) (Crites and Tchobanoglous, 1998).

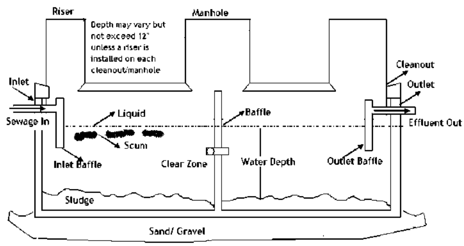


Figure 9.1(b): Schematic representation of the septic tank (USEPA, 1980)

Several types of drain field are available, such as conventional drain fields, pressure-dosed drain fields, gravity-dosed drain fields, and at-grade drain fields. A conventional drain field is the simplest type, in which effluent is passed to a drain field by gravity through perforated pipes buried 0.6–0.9 m (2–3 feet) below ground level. The pipes are placed on a gravel bed (Fig. 1(a)). The effluent percolates through the soil and undergoes aerobic and anaerobic 'treatment' in the soil, resulting in nutrient removal. Various physical, chemical, and biological processes are involved when wastewater passes through the soil layers in the vadose zone, and the size of the drain field depends on soil conditions, flow rates, groundwater levels, and the slope of the land. Sometimes, clay in soil creates problem in the percolation process when fluids pass through soil. In that case, a pressure-dosed drain field has to be adopted, using a dosing pump to periodically dose the effluent in the drain field. A dosing tank is used to collect effluent from the septic tank for intermittent dosing to the drain field. A gravity-dosed drain field has a similar process, using a siphon rather than a pump, as long as there is a sufficient elevation difference between the dosing tank and drain field. In an area where the groundwater level is too high, such as in coastal communities, at-grade or mound drain fields can be used. Such systems are generally built at ground level, and effluent needs to be dosed by pressure.

Advanced OWTS, aimed at improving the overall performance of nutrient removal, include the use of suspended growth reactors, such as a sequencing batch reactor (SBR), or attached growth reactors, such as rotating biological contactors (RBCs), sand filters, and bio-filters. Additionally, alternatives such as constructed wetlands, sand mounds with low-pressure pipe (LPP) distribution systems, drip irrigation systems, and modified shallow placed gravity distributors may be considered as an integral part of the OWTS. A sand filter is a packed-bed filter of sand or other granular material that provides advanced secondary treatment of septic tank effluent, whereas a bio-filter is a packed-bed filter using other, more porous materials (e.g., peat, textiles, foam) to provide advanced secondary treatment of septic tank effluent. These two types of filter are often used for nutrient removal as an additional unit operation, placed between the septic tank and the drain field. In an ecological engineering paradigm, an OWTS may incorporate an aquatic treatment system at the end, instead of the drain field. This type of system, a so-called 'constructed wetland,' consists of one or more lined basins that may be filled with a medium and plant species where wastewater undergoes a combination of physical, chemical, and/or biological treatment and evapotranspiration. Sand mounds are an above-ground treatment system incorporating at least 30.5 cm (12 inches) of clean sand above the original soil surface that disperses the treated wastewater into the original soil. A drip irrigation system is a subsurface soil dispersal system that distributes treated wastewater through drip irrigation lines.

9.2.2 Past Experience

In the last few decades, scientists, engineers, and manufacturers in the wastewater treatment industry had developed a wide range of alternative technologies designed to address increasing hydraulic loads and water contamination by nutrients and pathogens in septic tank systems. In particular, the Florida Keys On-site Wastewater Nutrient Reduction Systems (OWNRS) Demonstration Project was initiated in 1995 to demonstrate the use of OWTS to reduce the concentrations of nutrients discharged to the coastal region of the Keys (Anderson et al., 1998). Five treatment trains were adopted for testing in that study: 1) a septic tank, followed by a recirculating sand filter (RSF) and an anoxic bio-filter (ABF) with effluent discharged to an unlined drip irrigation field; 2) a septic tank with effluent discharged to a lined drip irrigation field; 3) a fixed-film activated sludge (FAS) treatment, known as the Bio-Microbics FAST aerobic treatment unit (ATU), and an anoxic bio-filter (ABF) with effluent discharged to an unlined drip irrigation field; 4) a suspended-growth biological treatment system, operated by a continuous feed cyclic reactor (CFCR), known as the AES BESTEP-IDEA system and similar to a SBR, with effluent discharged to an unlined drip irrigation field; and 5) an RBC and an anoxic bio-filter (ABF), with effluent discharged to an unlined drip irrigation field. Additional unit operations, such as chemical precipitation, supplemental carbon addition for denitrification, and additional phosphorus adsorption media, are also available. Twenty-four hour flow composite samples were collected from the influent mix tank and from each of the three treatment process effluents, and samples were analyzed by standard methods (American Public Health Association; APHA, 1992) for biochemical oxygen demand (BOD_5), carbonaceous biochemical oxygen demand ($CBOD_5$), total suspended solids (TSS), total Kjeldahl nitrogen (TKN), nitrate nitrogen (NO_3-N), nitrite-nitrogen (NO_2-N), and total phosphorus (TP). Total nitrogen (TN) was obtained by summation. However, without chemical precipitation, the quality of the effluents did not meet the Florida advanced wastewater treatment (AWT) standards of 5 mg/L for CBOD and TSS, 3 mg/L for TN, and 1 mg/L for TP.

Septic tanks, followed by lined and/or unlined sub-surface wetlands in sequence, have also been used for on-site wastewater treatment over the last decade (Mankin and Powell, 1998; Thom et al, 1998; Sun et al., 1998). The University of West Florida installed a constructed wetland in 1994 to treat 1,900 lpd (500 gpd) of residential wastewater. The system uses a hybrid approach, combining subsurface and free-water surface flow designs. It consists of one 3,780 lpd (1,000 gallons) primary treatment septic tank and a 76.2 × 76.2 cm (30 × 30') cell of wetland, sub-divided into three compartments. The removal efficiencies for TSS, TPO_4, NH_3, BOD_5, TKN, and fecal-coliform were 98%, 88%, 60%, 94%, 77%, and 97%, respectively. These tests proved the potential of using wetlands as a means of polishing septic tank effluent, without the need to use complex aerobic/anaerobic wastewater treatment technologies such as AES BESTEP-IDEA or Bio-Microbics FAST. However, Florida's current septic tank regulations require subsurface flow of wastewater effluents in OWTS. Many other commercial units have been developed, such as the Waterloo Biofilter combined with a leaching trench, and NITREX combined with a drain field (Lombardo, 2005). The former is a trickling filter for aeration with foam media, and the latter separately performs the nitrification and denitrification in two units in sequence. An attached anoxic growth reactor is used to foster the denitrification.

In short, an aerobic treatment unit can be added to some septic tank systems, followed by a clarifier chamber. In most cases, the capacity of such a unit is about 2271–5678 liters (600–1,500 gallons) (USEPA, 1980). The main objective in adding an aerobic treatment unit is to gain additional BOD_5 and ammonia removal. However, careful supervision is required to ensure that this occurs. Presently, only two commercial systems are available; one uses suspended growth and the other uses fixed growth. In the suspended-growth system, the medium is kept in suspension in the system using a rotating fan or high air pressure at the bottom of the tank. In the fixed-growth system, the medium is kept fixed in the tank. The main purpose of the medium is to support the growth of microorganisms. A suspended-growth system can be affected by several factors, including long hydraulic and solid-retention time

(SRT), dissolved oxygen (DO) concentration, and wastewater characteristics (USEPA, 1980). SRT should not be high enough that suspended solid or medium can be washed out of the system. This is one of the major operating problems in this system. DO concentrations should be maintained above 2.0 mg/L to obtain high removal efficiency. As in centralized domestic wastewater treatment plants, separated biomass in the clarifier must be returned to the system, using a return flow. The performance of the clarifier is also very important. If the clarifier is not working properly, biomass, the mass of living organisms in the system, cannot be separated from the effluent liquid.

In fixed-growth systems, an intermittent sand filter or recirculation sand filter tank (ISF or RSF, respectively) is used between the septic tank and the drain field. If the ISF is designed properly, it can increase effluent water quality. An ISF can be constructed partially buried in the ground or completely above ground. If the groundwater table is too high or shallow bedrock is present, it is better to construct the ISF completely above ground. An ISF is generally constructed at a depth of 60–90 cm (24–36 in) (USEPA, 1980). It has been found that most treatment occurs in the top 23–30 cm of the bed. Additional depth is typically not helpful in achieving extra treatment. The sand filter tank size is about 1.5× the peak daily flow. These two systems include a recycle flow. The recycle ratio generally varies from 3:1 to 5:1 (USEPA, 1993a), meaning that three (to five) parts enter the recycle flow and one part flows forward. Medium size is very important for filter media; filter medium size is generally expressed in terms of effective size and uniformity coefficient. The effective size (D_{10}) is defined as the size of the sieve (in mm) through which 10% of the sample of sand, by weight, will pass. The uniformity coefficient (D_{60}/D_{10}) is defined as the ratio of the sieve size in mm through which 60% of a sample of sand will pass to the effective size. For effective filter media, the effective size ranges from 0.25 to 1.50 mm and uniformity coefficient should be less than 4 (USEPA, 1980). The medium should be durable and insoluble in water. It is generally recommended that a finer medium be placed between courser medium materials. If only finer material is placed at the bottom or top, it may accelerate clogging or water seal, and thereby reduce the amount of oxygen required to decompose organic materials. The hydraulic loading rate is generally in the range 0.30–0.60 m^3/m^2/day, and the optimal organic loading rate has not yet been determined. However, it has been found that ISF performance is affected by the accumulation of organic matter in the filter bed (USEPA, 1980). The hydraulic loading rate is defined as the volume of liquid applied on the surface area of the bed for the intended time period. The organic loading rate is defined as the volume of soluble and insoluble organic materials applied to the filter bed within the intended time period. The distribution and under-drain pipes generally have a minimum diameter of 10.2 cm (4 inches) (USEPA, 1980). A split-bed recirculation sand filter is also recommended to increase the nitrogen removal (DOC, 1999). This filter has two chambers: the recirculation side and the effluent side.

The filter bed may become clogged over time, and this happens more quickly with increasing hydraulic loading rate (Venhuizen, 1998; Hurst, 2006). Three types of clogging are generally observed in an ISF: physical, chemical, and biological clogging. Physical clogging may occur due to the accumulation of solids in the pore space of the sand particles. This depends on the porosity of the medium and the particle size. Chemical clogging results from precipitation reactions with minerals in the sand. Biological clogging is due to the formation of biofilms on the surface of sand particles or the slow decomposition of organic materials; this depends on the microorganism population and the balance of microorganisms in the sand filter. The population of microorganisms also influences the quality of effluent water (Hurst, 2006). Both these factors are dependent on the presence of toxic compounds in the wastewater, the dissolved oxygen concentration, and the temperature. The amount of oxygen may help the microorganisms to decompose the organic materials and may affect their survival. Temperature can influence chemical reactions and microorganism growth. Start nitrification in a sand filter can take from about two weeks to six months. However, the process may be hampered in winter, as bacterial growth may be slowed due to the low temperature. The denitrification process is very slow in sand filters (USEPA, 1980).

According to the USEPA, biological and chemical treatment takes place within 15.2 cm (6 inches) of sand (USEPA, 1993a). If properly operated, an ISF can remove 86–91% of ammonia-nitrogen, 90% of fecal coliforms, and 20–60% of TN (RIDEM, 2000). Another type of sand filter is a single-pass sand filter (SPSF), which has no recycle flow. A low-rate SPSF can be loaded with a maximum of 50.9 L/sq. m/day (1.3 gallons/sq. ft/day) and a high-rate SPSF can be loaded with a maximum of 81.5 L/sq. m/day (2.0 gallons/sq. ft./day). A low-rate SPSF is preferred when microorganism removal is a major concern, and a high-rate SPSF is preferred when space for the septic tank system is limited (RIDEM, 2000). It is generally recommended that sand filter and drain field performance be increased by adding some red mud or iron oxide with the sand (USEPA, 1993b). Red mud is a by-product of bauxite mining and is high in iron oxide (Fe_2O_3) and aluminum oxide (Al_2O_3). Red mud should be about 30% of the total volume of the filter bed (USEPA, 1993b).

The drain field (distribution system) in an engineered OWTS may be designed in several different ways, including as a standard drain field, a low-pressure drip dispersal drain field, a subsurface drip distribution system, a spray distribution system, a mound system, an evapotranspiration bed, and a soil substitution drain field. In combination with different arrangements of septic tanks and treatment filters, there are at least 22 different types of system configurations (Texas Cooperative Extension, 2005). These technologies can achieve significant pollutant removal rates. It is also known that the use of sulfur and limestone may create a similar anoxic environment for denitrification in the drain field directly (Shan and Zhang, 1998). However, improperly functioning systems may still pose a contamination risk to groundwater and surface water supplies. As different sorption media may exhibit different macropore structures and forms or foster differing biofilms and sorption capacities, their nitrification and denitrification potentials are of interest. To date, little is known about how these alternative on-site wastewater treatment technologies, especially passive treatment systems with different sorption media, may impact nutrient removal efficiency. The effects of medium variation on intermittent sand filters and lined drain fields are of particular concern (Weaver et al., 1998).

Recently, a lined drain field with soil substitution using different sorption media was developed as a new, passive on-site wastewater treatment technology. Thus, OWTS treatment trains that include a filtering device, either in a filtering tank or in a drain field, designed with different sorption media as a soil substitute in some engineered OWTSs provide an opportunity to improve nutrient removal efficiency. In the past, sand, recycled crushed glass, sulfur, crushed limestone, polyethylene pellets, peat, and open-cell foam have been tested (Weaver et al., 1998). Extended research focusing on the use of different treatment media such as sawdust, zeolites, tire crumbs, oyster shells, and spodosols for improving nutrient removal from alternative on-site wastewater treatment technologies has become a focus. Additional passive on-site wastewater treatment technologies, such as constructed wetlands and drain fields with soil substitutes, could be more cost-effective than the other currently promoted performance-based OWTSs. Verification of the cost-effectiveness of such an OWTS would also be of value.

However, simply using different sand in the drain fields can affect performance, which may enable cost-effective compliance with requirements. Drain field soil condition is very important for nutrient removal. If the soil is permeable, the water can percolate through it, and biological and/or chemical process can occur. Otherwise, it may cause flooding due to low permeability. For course sand or loamy course sand, the rate is 47.3 s/cm (<2 min/inch); for loamy sand, sandy loam, course sandy loam, or fine sand, the rate is 47.3–94.5 s/cm (2–4 min/inch); for fine sandy loam, very fine sand, silt loam, very fine sandy loam, loamy fine sand, loamy very fine sand, the rate is 2.0–3.9 min/cm (5–10 min/inch); for clay loam, silty clay loam, sandy clay, silty clay, and clay, the rate is >5.9 min/cm (15 min/inch) and <11.8 min/cm (30 min/inch); for clay, organic soil and bedrock, the rate is >11.8 min/cm. This last soil type is generally considered unsatisfactory for subsurface drain fields (FDOH, 2007). Amador et al. (2008) conducted laboratory experiments to understand the effect of sand depth on nutrient removal (i.e., nitrogen and phosphorus species, fecal coliforms) in septic tank effluent. They found that sand depth had almost no effect on nitrogen species removal from wastewater. Experiments were conducted at depths of 7.5, 15, and 30 cm. All the chemical species had the same behavior at these depths. Ammonia removal was about 81% regardless of depth. Total nitrogen removal was about 28, 22, and 23% at depths of 7.5, 15, and 30 cm, respectively. Total phosphorus removal was about 18, 13, and 16% at depths of 7.5, 15, and 30 cm, respectively. The removal of fecal coliforms varied from 82.4 to 92.3% (Amador et al., 2008). Havard et al. (2008) examined the performance of sand filters with different slopes and materials. They used six sand filters: three filled with mortar (i.e., fine particles), concrete (i.e., medium particles), and silica (i.e., course particles) sand, with a 3% slope and three filled with the same filter materials with a 30% slope. In the case of nitrogen removal, slope and filter medium size did not influence the removal process. Ammonia removal varied from 98.6 to 96.5%, nitrate removal was increased, and TN removal varied from 60.4 to 66.4% in all six columns. However, phosphorus removal was affected by slope and medium particles. Phosphorus removal was higher with fine particles in both slopes (Havard et al., 2008).

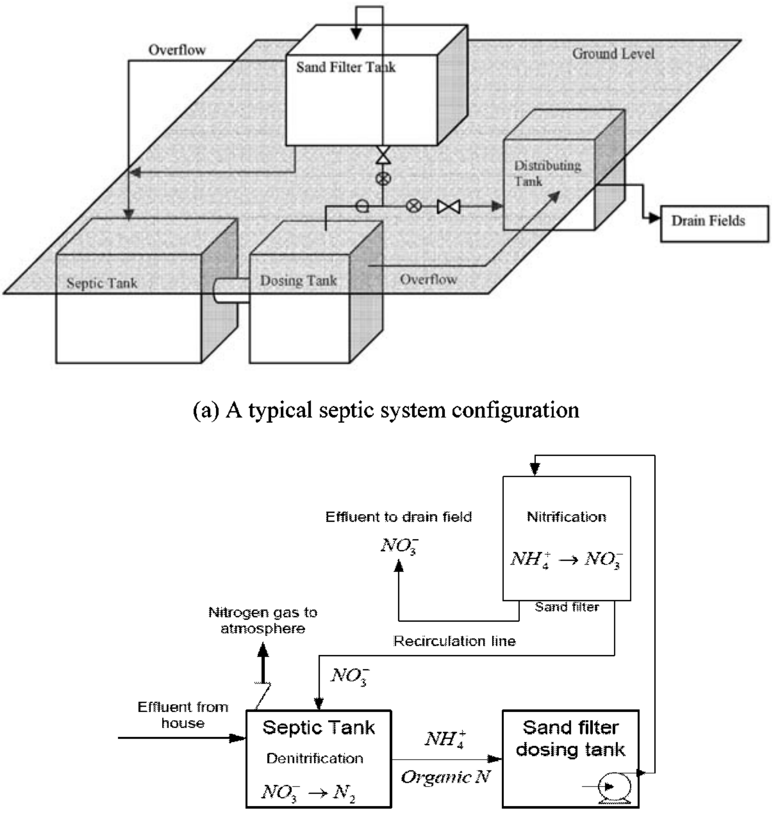
In particular, both astatula sand and washed building sand that are commonly available in Florida may be useful for comparison because the latter is much more expensive than the former. Astatula sand is used without pretreatment, whereas washed building sand is washed to remove impurities. Astatula sand has a permeability of 25 cm/min, and washed building sand has a permeability of 40 cm/min. Washed building sand is frequently used by contractors in drain fields. Astatula sand was selected for our laboratory experiments, and we demonstrated that this type of sand has the capacity to remove nutrients. A regional need exists for a comparative study that allows the evaluation of these two types of sands in drain field use in performance-based, passive on-site wastewater treatment systems for technological assessment and verification of cost-effectiveness.

9.2.3 System Configuration

The OWTS test center at the University of Central Florida (UCF) is sponsored by Florida Department of Environmental Protection (FDEP), with guidance from the state Department of Health (DOH), and was designed to systematically evaluate nutrient removal; it includes four treatment trains. The system presented in this paper consists of a septic tank followed by a recirculating sand filter (RSF). Effluent is discharged equally to two unlined conventional gravity drain fields in parallel, hereafter called standard drain fields; one is filled with washed building sand and the other with citrus grove sand (i.e., astatula sand). The wastewater source for the test center is the 15-person BPW Scholarship House (a female dormitory on the UCF campus), which contains a kitchen and living quarters. The wastewater is pumped to a 5.10 m^3 (1,350 gallon) septic tank from which the effluent is divided and sent to different final disposal facilities.

Figure 9.2(a) shows a schematic of such a treatment process, depicting a typical septic tank system with enhanced nitrification and denitrification using a RSF (USEPA, 1999). Due to the full exposure of the effluent to the air, the inflow of wastewater contains some dissolved oxygen, so it can trigger nitrification in the septic tank at first and in the RSF later. On the other hand, the area between the sludge and scum can become completely anaerobic with time, so it may support denitrification processes in the septic tank (Bounds, 1997). Figure 9.2(b) illustrates the rationale for promoting nitrification by using the RSF.

A septic tank with a capacity of 5,110 liters (1,350 gallons) receives sewage from the 15–person BPW scholarship house. A hydraulic retention time (HRT) of 24 h is assumed in the septic tank. The septic tank then discharges the sewage to a dosing tank. The dosing tank sends the flow in two directions: one part goes to the distribution tank as feed-forward flow and the other three parts go to the recirculation sand filter (RSF) as recycled flow. Thus, the recycling ratio is about 3:1. RSF has a depth of about 1.2 m (4 feet) and it is filled with filter medium up to 1.1 m (3.5 feet). The RSF has three layers: the top layer is a gravel layer, the middle layer is a sand layer with a depth of 0.6 m (2 feet), and the bottom layer is gravel with depth of 0.31 m (1 foot). The capacity of the sand filter tank is about 3,975 liters (1,050 gallons). The RSF receives hourly dosing from the dosing tank. It has been reported that multiple dosing is very effective in increasing removal efficiency (USEPA, 1980). The recycled flow from the RSF goes to the septic tank and mixes with the fresh sewage flow, promoting further denitrification. From a distribution tank, the wastewater goes to the two drain fields in equal volumes so that each drain field receives about 50% of the total flow from the dosing tank. The size of each drain field is about 6.09 × 4.57 × 1.22 m (20 × 15 × 4 feet). One drain field is filled with astatula sand and the other with washed building sand. Each drain field is filled with native soil on the top, with a depth of 0.6 m, and the remaining 0.6 m is filled with astatula sand or washed building sand.



(a) A typical septic system configuration

(b) Nitrification and denitrification in a septic tank system

Figure 9.2: Layout of an effluent filter and a gravel-filled drain field (Chang et al., 2007; Venhuizen, 1998)

To sample the infiltrate and proceed with water quality monitoring in the vadose zone, three lysimeters (Soil Moisture Equipment Corporation) were installed in each of the two standard drain fields. The lysimeter equipment collects water from unsaturated or vadose zone by a porcelain cup. The water from a lysimeter is collected using a vacuum pump. The lysimeters are placed at three different depths: 20.3 cm (8 inches), 40.6 cm (16 inches), and 61 cm (24 inches).

A schematic diagram of the system layout at the UCF Test Center is shown in Figure 9.3(a). Samples are taken at 10 sampling points (Fig. 3(a), denoted S1–S10). Samples from points S5–S10 are collected by the lysimeters directly. Sampling points S5, S6, and S7 are located in the drain field filled with astatula sand, whereas sampling points S8, S9, and S10 are in drain fields filled with washed building sand. For this study, a composite sampling method was applied for sample collection; the composite sample is a combination of samples collected at different time periods and combined into one representative sample for chemical analysis, saving a significant amount of analysis cost, as only one sample is analyzed. Samples were collected biweekly in the morning (from 6:00 to 8:00 am), at mid-day (from 11:00 am to 1:00 pm), and in the evening (from 5:00 to 7:00 pm). Major nutrients of concern included ammonia, NO_X-N (the sum of nitrate and nitrite), NO_2^--N, TN, soluble reactive phosphorus (SRP), TP, fecal coliforms, and *E. coli.* All samples were analyzed by a certified laboratory, Environmental Research & Design Inc. (ERD) in Orlando, Florida. Groundwater nutrient concentrations were also determined in this project. For this purpose, 17 monitoring wells were constructed around the study site (Figure 9.3(b)). The monitoring wells were divided into two groups: 1) monitoring wells used to observe the effect of the drain field; and 2) monitoring wells used to observe the effect of the whole system, including wetlands and the B&G drain field system. Monitoring wells for the first group included wells M1–M8 and wells for the second group included wells MW1–MW8 . All wells were about 3 m (10 feet) deep. The groundwater depth of each well was monitored once a week, and nutrient concentrations were analyzed bi-weekly.

9.2.4 Sampling and Chemical Analysis

The list of methods used to determine the concentrations of the chemicals is shown in Table 9.1. Quality assurance and quality control were conducted for each sample set. To assess precision of measurement, duplicate samples were analyzed and the relative percentage difference (RPD) between the two measurements was determined. The accuracy of the measurements was determined by spiking a sample with a known concentration of a chemical and calculating the percentage recovery.

Table 9.1: Water quality parameters and methods

Parameter	Method	Range*
Nitrates + Nitrites	Hach method 8192	0.01–0.50 mg/L NO_3-N
Nitrites	Hach method 8507	0.002–0.30 mg/L NO_2-N
Ammonia	Hach method 8155	0.01–0.50 mg/L NH_3-N
Total Nitrogen	Hach Method 10071	0.50–25.00 mg/L N
Reactive/Orthophosphate	Hach method 8048	0.02–2.50 mg/L PO_4^{3-}
Total phosphorus	Hach method 8190	0.06–3.50 mg/L PO_4^{3-}

*Note: In some cases, samples were diluted with DI water to bring them within this range.

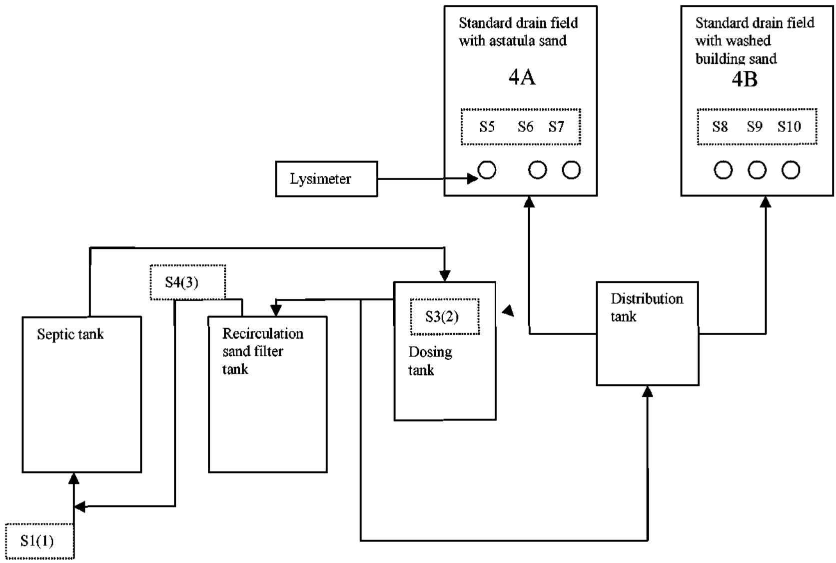


Figure 9.3(a): System layout of the OWTS consisting of a septic tank, a dosing tank, a sand-filtered circulation tank, a distributing tank, and two parallel drain fields.

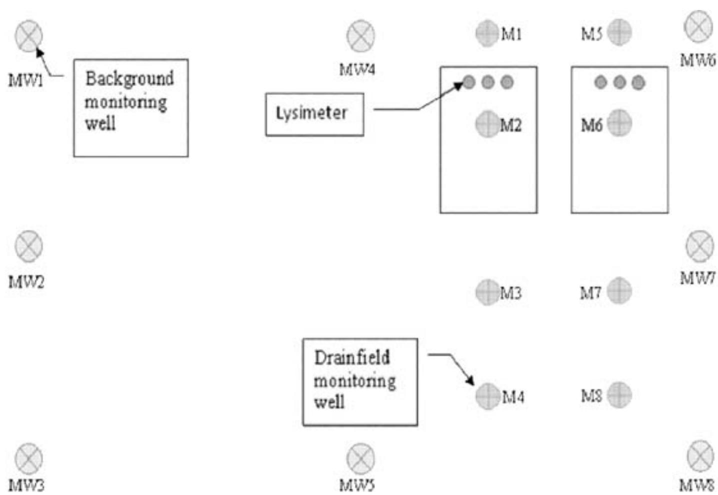


Figure 9.3(b): Figure shows the approximate location of groundwater monitoring wells

9.2.5 Microbiological Assessment

As mentioned above, the middle layer of the RSF has a depth of 0.6 m (2 feet). Sand samples were collected from the top (0 ft, sample R1), midpoint (1 ft, sample R2), and bottom (2 ft, sample R3) of the middle layer of RSF for quantifying the ammonia-oxidizing bacteria (AOB) and nitrite-oxidizing bacteria (NOB) to assess the profile of microbial activity responsible for nitrification effects. As part of total nitrogen removal in this whole wastewater treatment system, the denitrification effect of the soil profile along the flow path was also measured by quantifying denitrifiers in both drain fields. Three samples were collected from each drain field at different depths. The first was collected at the depth of the bottom of shallowest lysimeter installed (20.3 cm (8 inches) deep in the sand layer), sample W1 or A1. The deepest, sample W3 or A3, was close to the groundwater table. The other sample was collected about 1.8 m (6 feet) deep from the ground surface, sample W2 or A2. All nine samples were collected in 1.5-mL micro-centrifuge tubes, transported at 4°C, and kept at -20°C until use.

DNA from each sample was extracted, in duplicate, using the SoilMaster DNA Extraction Kit (EPICENTRE), following the manufacturer's protocol; 50 mg of sample, instead of 100 mg, was weighed out into 1.5 mL micro-centrifuge tubes to decrease the effects of enzymatic inhibitors. The extracted DNA was then resuspended in 300 μL TE Buffer. Real-time PCR quantification was performed with a Stepone (Applied BioSystems) to amplify the *amoA* gene from the AOB, the *NSR* gene from NOB, and the *nirK* gene from the denitrifiers. The target genes were in direct proportion to the number of bacteria. The PCR mixture was prepared in a total volume of 25 μL using 12.5 μL of the SYBR Green PCR Master Mix kit (a combination of SYBR Green 1 Dye, AmpliTaq Gold DNA Polymerase, dNTPs with dUTP, Passive Reference 1, and optimized buffer components; Applied BioSystems), 10 μM of each primer, standard DNA or extracted DNA from samples, and DEPC-treated water, to complete the 25 μL volume (Table 9.2).

Table 9.3 shows the sequence of the primers used. To generate a standard curve for real-time PCR, plasmid carrying the target DNA was extracted with the PureLink Quick Plasmid Miniprep Kit (Invitrogen), and the plasmid DNA concentration (ng/μL) was measured by spectrophotometry (NanoDrop 1000). Because the sequences of the vector and the PCR insert are known, we calculated the copy numbers of *amoA, NSR*, and *nirK* directly from the concentration of extracted plasmid DNA. A standard DNA was prepared from plasmid DNA for ten-fold serial dilutions, and each was quantified in triplicate. Standard curves were obtained by plotting Ct (threshold cycle) as a function of the log of the copy number of target DNA. The copy number can be obtained by the known mass concentration of standard DNA and molecular mass of the target gene.

Table 9.2: The composition of real time PCR mixture for different genes to be amplified (μL)

	amoA	NSR	nirK
SYBR Green	12.5	12.5	12.5
Primer	1.5	1.5	1
Standard DNA or template DNA	2	2	1
DEPC water	9	9	10.5
Total	25	25	25

Table 3: Sequences of the oligonucleotide primers

Ammonium monooxygenase (*amoA*)	amoA-1F GGGGTTTCTACTGGTGGT	Rotthauwe et al. (1997)
	amoA-2R CCCCTCKGSAAAGCCTTCTTC	
16S rRNA Nitrospira sp. (*NSR*)	NSR 1113F CCTGCTTTCAGTTGCTACCG	Dionisi et al. (2004)
	NSR 1264R GTTTGCAGCGCTTTGTACCG	
Nitrite reductase (*nirK*)	nirK 876 ATYGGCGGVAYGGCGA	Braker et al. (1998)
	nirK 1040 GCCTCGATCAGRTTRTGGTT	

The real time PCR protocol for *amoA* quantification was as follows: 2 min at 50°C, 10 min at 95°C, and then 40 cycles of 30 s at 95°C, 45 s at 55°C, and 30 s at 72°C (Okano et al. 2004). PCR amplification for *NSR* consisted of 2 min at 50°C, 10 min at 95°C, 35 cycles at 95°C for 15 s, and 63°C for 30 s (Harms et al. 2003). The conditions for *nirK* real-time PCR were 120 s at 50°C, 900 s at 95°C, and six 'touchdown' cycles of 15 s at 95°C for denaturation, 30 s at 63°C for annealing, 30 s at 72°C for extension, and 30 s at 80°C for a final data acquisition step. The annealing temperature was progressively decreased by 1°C, down to 58°C. The final cycle with an annealing temperature of 58°C was repeated 40 times (Henry, et al. 2004).

9.3 RESULTS AND DISCUSSION

9.3.1 Standard Drain field Performance for Nutrient Removal

Nutrient concentrations are very high in influent wastewater. By microbiological processes, ammonia and nitrite can be converted to nitrate through nitrification, and nitrate can be converted to nitrogen gas through a denitrification process. The concentrations at different points are shown in Table 9.4. It was found that both ammonia and NO_x-N concentrations decreased in sand filters and drain fields, showing that both nitrification and denitrification occurred in the drain fields. The RSF can achieve about 99.5% ammonia removal. However, at the same time, the RSF filter was removing about 84.3% of NO_x-N. Thus, it can be concluded that RSF can support both nitrification and denitrification processes. In fact, the top layer of RSF may support the nitrification process and the bottom layer, the denitrification process. To determine this, we used real time PCR analysis.

Table 9.4: Nutrient concentrations at different sampling points

Sample description	NH_3-N (mg/L)	NO_X-N (mg/L)	TN (mg/L)	SRP (mg/L)	TP (mg/L)
S1	42.140	0.102	46.110	4.918	9.891
S3	6.398	0.005	8.318	3.122	3.802
S4	0.199	0.016	7.329	2.725	2.918
S5	0.056	0.005	4.737	2.815	2.989
S6	0.048	0.005	6.674	2.591	2.771
S7	0.110	0.044	9.931	2.289	2.897
S8	0.006	0.011	4.128	2.273	2.430
S9	0.032	0.005	11.579	2.054	2.214
S10	0.044	0.005	5.667	2.072	3.907

Ammonia and NO_X-N removal can be further enhanced in drain fields with washed building sand. The average concentration of ammonia in the astatula sand drain fields and washed building sand drain fields were about 0.07 mg/L (average of S5, S6, and S7) and 0.03 mg/L (average of S8, S9, and S10), respectively. Thus, the removal efficiency was about 99.8% and 99.9%, respectively. NO_x-N removal in the astatula sand and washed building sand drain fields was about 82.4% and 93.1%, respectively. The effluent TN concentration was lower than the influent TN concentration. TN removal in the astatula sand drain field was about 84.6%, and in the washed building sand drain field, it was 84.5%. Denitrifiers may use oxygen, rather than nitrate, if an anaerobic environment cannot be created. Denitrification may occur at a slow pace or not at all if anaerobic conditions are not created in the sand filter. It is possible that anaerobic conditions become prevailing in the ground at greater depths. In our project, the depth of the drain field was only 1.22 m. Sand cannot remove nitrate or nitrite by adsorption. Nitrate, nitrite, and sand all are negatively charged, and when they come in close contact, they repel each other (Kim et al., 2000).

For phosphorus removal, the washed building sand drain field performed relatively well. The average removal efficiency of SRP in the astatula sand drain field and washed building sand drain field was about 47.8% and 56.6%, respectively. TP removal was about 70.8% in the astatula sand drain field and 71.2% in the washed building sand drain field. RSF plays an important role in removing phosphorus species. About 44.6% of SRP and 70.5% of TP was removed by RSF. After operation of the RSF, the phosphorus concentration increased or remained unchanged. This may have been due to the feed-forward flow from the dosing tank to the drain field, as the dosing tank has higher phosphorus concentrations. Thus, RSF performance for phosphorus removal was not of value in this feed-forward flow pathway. Phosphorus may be removed by chemical reactions with minerals present in the sand drain fields or adsorption by clay particles present in the sand. Figure 9.4 summarizes the variations in the pathway in terms of nutrient removal. Table 9.5 summarizes the nutrient removal efficiencies for the constituents of interest.

Table 9.5: Summary of nutrient removal efficiency from influent to drain field

Description	NH_3-N (%)	NO_X-N (%)	TN (%)	SRP (%)	TP (%)
astatula sand	99.83	82.35	84.57	47.84	70.82
washed building sand	99.93	93.13	84.54	56.62	71.18

The microorganism concentration in the influent was very high. The fecal coliform concentration was about 412 × 10^3 colony forming units (CFU)/100 mL, and the *E. coli* concentration was about 214 × 10^3 CFU/100 mL. The concentration was much lower in the drain fields, and the washed building sand drain field performed better for microorganism removal. However, both drain fields achieved about 99.99% removal efficiency. The average *E. coli* concentration in both the astatula sand and the washed building sand drain field was <1 CFU/100 mL. The average fecal coliform concentration in both fields was <1 CFU/100 mL.

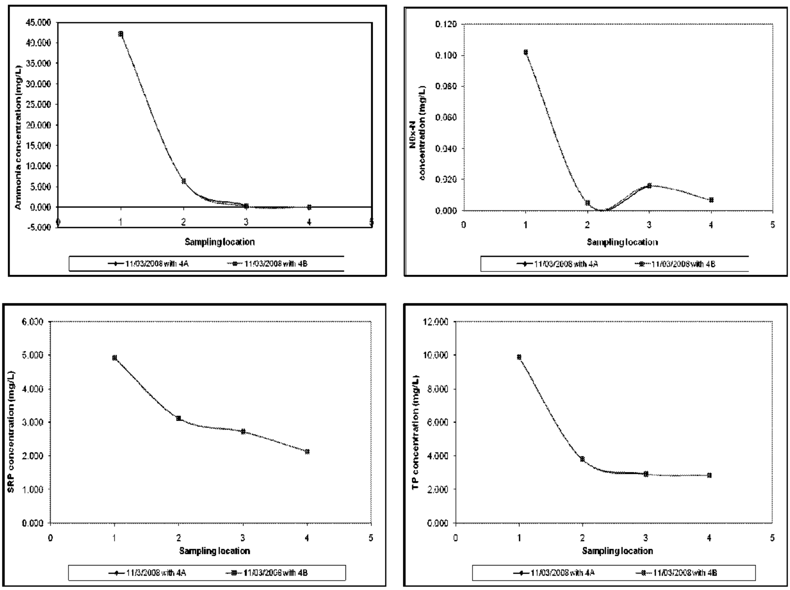


Figure 9.4: Variations in the pathway in terms of nutrient removal

9.3.2 Microbiological Assessment

The standard plasmid DNA was prepared in ten-fold serial dilutions (from 10^1 to 10^5 gene copies/μL), and each was quantified in triplicate. The Stepone software (V1.0, Applied BioSystems) was used to process and analyze the data.

Standard curves were obtained by plotting Ct (threshold cycle) as a function of the log of the copy number of the target DNA (Table 9.6).

Ammonium oxidation is the first stage of nitrification. The function of autotrophic AOB is to convert ammonium to nitrite under aerobic conditions. As Figure 9.5(a) shows below, the top sample, with the highest oxygen exposure, had the highest quantity of AOB. The amoA gene copy number per gram of sample might be expected to decrease with depth. However, the bottom sample has more AOB than the midpoint sample, possibly because there is still some oxygen remaining between the coarse particles of the gravel layer. Thus, the midpoint sample, rather than bottom sample, has the least amount of AOB.

Nitrite oxidation is the other stage of the nitrification process. Autotrophic nitrite-oxidizing bacteria convert nitrite to nitrate under aerobic conditions. Figure 9.5(b) shows that the top sample had the highest quantity of NOB. From Table 9.4, about 40% of NH_3-N had not been removed and flowed out of the outlet of the RSF. The limited concentration of nitrite produced by AOB made the level of NOB almost the same at the middle point and the bottom of the RSF.

Table 9.6: Quantity of different target genes in samples and standard curves used

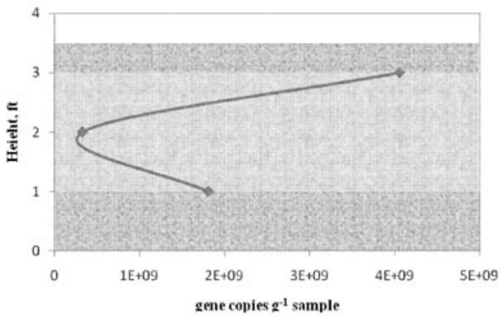
Target gene	Sample NO.	gene copy number g^{-1} sample	Standard Curves			
			Slope	Y-Intercept	R^2	Dilution range, gene copies/µL
amoA	R1	4.1×10^9	-3.45	49.15	0.99	10^5-10^9
	R2	3.3×10^8				
	R3	1.8×10^9				
NSR	R1	9.8×10^4	-3.25	32.65	0.99	10^1-10^5
	R2	1.6×10^4				
	R3	1.5×10^4				
nirK*	W1	1.4×10^4	-3.49	36.37	0.98	10^1-10^5
	A1	UD				
	W2	9.5×10^4				
	A2	1.6×10^5				
	W3	UD				
	A3	UD				

UD: under the detection limit, which was about 6000 gene copy number g^{-1} of sample.

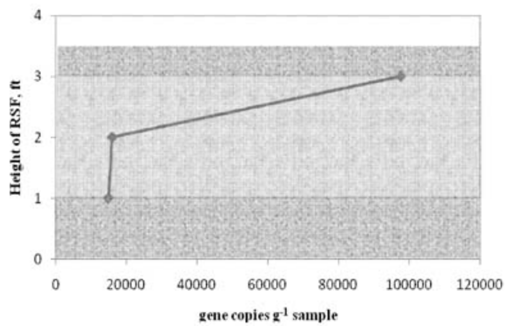
W: washed sand; A: Astatula sand

*The deepest (Sample No., W3, A3) was close to the groundwater table whereas surface samples (Sample No., W1, A1) was close to the surface (20 cm beneath the surface). Sample No., W2, A2 collected in between at about 1.8m in depth.

As shown in Figure 9.6, the *nirK* gene copy numbers per gram of sample showed a trend toward higher values at increasing depths. The *nirK* gene copy number per gram of the sample close to the water table was reduced to about zero. The oxygen content in the soil decreased with depth, which is the main reason why *nirK* gene copy number per gram of sample increased with depth. Dissolved oxygen in groundwater and reduced organic matter may cause the reduction of denitrifying bacteria number in the sample close to groundwater table. Additionally, the number of denitrifiers in washed building sand was higher than in astatula sand. However, NO_X-N removal in the astatula sand drain field was as good as that in the washed building sand drain field. As discussed above, anaerobic conditions were probably not ideally created during the initial test run of the treatment system. Nitrification may have started in the drain fields, but denitrification had apparently not yet started.



(a) *amoA* gene copy number versus depth: real-time PCR



(b) *NSR* gene copy number versus depth: real-time PCR

Figure 9.5: Changing AOB and NOB versus depth in the RSF

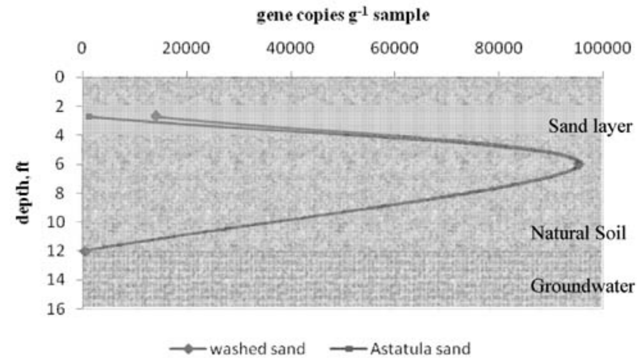
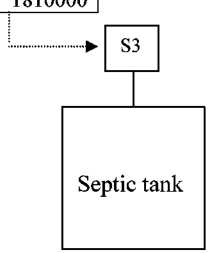


Figure 9.6. ***nirK* gene copy number versus depth of drain field: real-time PCR**

9.3.3 Mass Balance in Treatment Processes

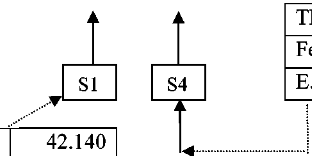
A mass balance analysis, also known as a material balance, measures the conservation of mass in a physical or chemical system; it is. According to the law of material conservation, the mass in a closed system will remain constant and is not dependent on the process going on inside the system. In the case of a chemical reaction, mass can be changed from one chemical species to another. The flow is measured by a flow meter established in the inlet of the tank. Figure 9.7 shows a mass balance diagram to help clarify changes in inflow and outflow concentrations of nutrients and *E. coli* in each unit, on average. Figure 9.7(e) illustrates the average performance of the two drain fields.

NH_3-N in mg/L	6.398
NO_X-N in mg/L	0.005
N in mg/L	8.318
SRP in mg/L	3.122
TP in mg/L	3.802
Fecal coliform in cfu	2820000
E. Coli in cfu	1810000



NH_3-N in mg/L	0.199
NO_X-N in mg/L	0.016
TN in mg/L	7.329
SRP in mg/L	2.725

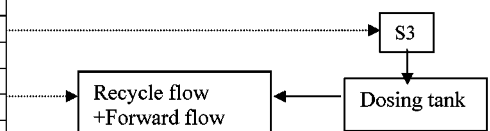
TP in mg/L	2.918
Fecal coliform in cfu	23333
E. Coli in cfu	2300



NH_3-N in mg/L	42.140
NO_X-N in mg/L	0.102
TN in mg/L	46.110
SRP in mg/L	4.918
TP in mg/L	9.891
Fecal coliform in cfu	4120000
E. Coli in cfu	2140000

(a) Mass balance diagram for septic tank

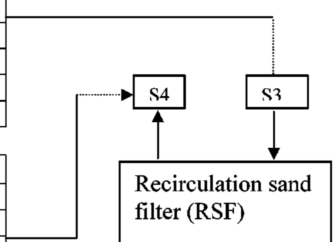
NH_3-N in mg/L	6.398
NO_X-N in mg/L	0.005
TN in mg/L	8.318
SRP in mg/L	3.122
TP in mg/L	3.802
Fecal coliform in cfu	2820000
E. Coli in cfu	1810000



(b) Mass balance diagram for dosing tank

NH_3-N in mg/L	6.398
NO_X-N in mg/L	0.005
TN in mg/L	8.318
SRP in mg/L	3.122
TP in mg/L	3.802
Fecal coliform in cfu	2820000
E. Coli in cfu	1810000

NH_3-N in mg/L	0.199
NO_X-N in mg/L	0.016
TN in mg/L	7.329
SRP in mg/L	2.725
TP in mg/L	2.918
Fecal coliform in cfu	23333
E. Coli in cfu	2300



(c) Mass balance diagram for RSF

Flow to astatula sand ← Distribution tank → Flow to washed building

NH_3-N in mg/L	6.398
NO_X-N in mg/L	0.005
TN in mg/L	8.318
SRP in mg/L	3.122
TP in mg/L	3.802
Fecal coliform in cfu	2820000
E. Coli in cfu	1810000

S3 (Forward flow)

(d) Mass balance diagram for distribution tank

Drain field

S5 S6 S7

NH_3-N in mg/L	6.398
NO_X-N in mg/L	0.005
TN in mg/L	8.318
SRP in mg/L	3.122
TP in mg/L	3.802
Fecal coliform in cfu	2820000
E. Coli in cfu	1810000

Flow from distribution tank

(e) Mass balance diagram for drain field

Figure 9.7: The mass balance diagram

9.3.4 Groundwater Monitoring

Groundwater elevation data are shown in Figure 9.8. The groundwater was flowing toward the southwest, as indicated by the arrow. Nutrient concentrations in groundwater at different well locations were analyzed. Ammonia concentration varied from a maximum of 7.870 mg/L at M7 to a minimum of 0.153 mg/L at MW2. The distance between these two wells is about 45.9 m. The NO_X-N concentration varied from a maximum of 2.23 mg/L at MW2 to a minimum of 0.005 mg/L at M4, separated by a distance of 37.3 m. Both NO_X-N and NO_2-N concentrations in groundwater were in compliance with the USEPA requirements. The USEPA-recommended maximum concentration for NO_3^--N is 10.00 mg/L and for NO_2^--N is 1.0 mg/L (maximum contaminant levels, MCLs) (USEPA, 1988). The findings indicate that denitrification may increase with increasing depth. This is reasonable because less oxygen was present at deeper depths and more nitrate was available for denitrifiers to grow. The maximum TN concentration was 9.610 mg/L at M7, and the minimum was 1.30 mg/L at MW8, separated by a distance of 37.8 m. The maximum SRP concentration in groundwater was about 0.90 mg/L at MW6, and the maximum

TP concentration was about 1.76 mg/L at MW7. This phosphorus concentration may not be as harmful in groundwater as it is in surface water, in terms of eutrophication. However, drinking water should contain some phosphorus for teeth and bone formation in the human body. The US National Academy of Science (NAS) recommended a dietary intake of about 700 mg of phosphorus per day. The number of *E. coli* was <1 CFU/100 mL over all the wells. The maximum fecal coliform concentration was 57 CFU/100 mL at well M8. These microorganism concentrations decreased with increasing depth. It is obvious that soil in this drain field area was generally very effective in removing nutrients and harmful microorganisms, such as fecal coliform.

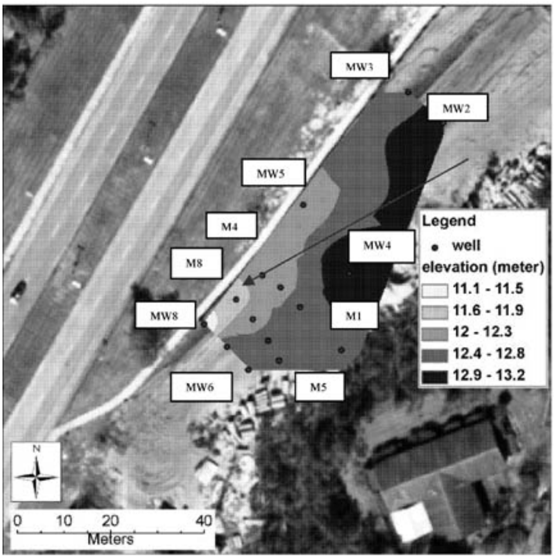


Figure 9.8: The groundwater table at different groundwater monitoring wells at the test site. The arrow indicates the direction of groundwater flow.

9.4 CONCLUSIONS

Removal of nutrients by RSF and drain fields occurs by a combination of physicochemical and microbiological processes. NO_X-N removal efficiency in the washed building sand drain field was slightly better than that in the astatula sand drain field. In all cases, the ammonia and NOx-N concentrations met the USEPA regulations. Although the system removed significant amounts of ammonia, it did not remove much nitrate. The groundwater sampling analysis demonstrated that NO_X-N and NO_2-N concentrations were within safe limits, according to the USEPA requirement. Phosphorus removal was not high enough in the system. However, it is not likely to have had a serious negative effect on groundwater quality. Research findings suggest that phosphorus removal can possibly be enhanced by adsorption, chemical reactions, or ion exchange if the sand has more clay and mineral particles. Sorption using a tire crumb mix, currently under investigation at the University OWTS test center, also proved effective for phosphorus removal. Due to cost-effectiveness concerns, astatula sand may be a useful and appropriate substitute for costly washed building sand in the future.

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CHAPTER 10

Considerations Regarding Geochemical Transformations Downstream of Subsurface Wastewater Effluent Disposal Facilities

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ABSTRACT: Many suburban and rural communities use subsurface disposal in conjunction with on-site and other small decentralized systems. Since these facilities introduce a variety of pollutants (including organics, nutrients, metals, and a variety of other constituents) into the subsurface, they can have important impacts on groundwater quality. This chapter provides an overview of some of the key geochemical transformations that may occur in groundwater down-gradient of wastewater effluent disposal facilities. A simple approach based on geochemical inverse modeling is described for identifying and confirming key processes. A case study is also presented to illustrate the processes that occur downstream of a wastewater effluent discharge facility. Application of inverse modeling for this facility demonstrates the significance of oxidation of carbon and ammonium in the groundwater downstream of the discharge. Although non-unique solutions are noted, the geochemical inverse modeling approach is found to be a helpful tool. The results illustrate the importance that biogeochemical processes and geochemical interactions with the sediment matrix can have on groundwater quality.

10.1 INTRODUCTION

Many suburban and rural communities use subsurface disposal in conjunction with on-site and other small decentralized systems. To reduce the impacts of wastewater treatment plant discharges on surface water resources, many communities are making use of subsurface disposal for the discharge of the effluent from larger treatment facilities as well. A number of textbooks and reports have addressed the basic guidelines for design of leaching facilities for septic systems and rapid infiltrations basins (RIBs) for wastewater treatment facilities (e.g. USEPA, 2002; Metcalf and Eddy, Inc., 1991). These analyses particularly address flow constraints, such as design loadings and mounding constraints.

In addition, a number of reports and publications have addressed the impacts of wastewater effluent disposal facilities on water quality for groundwater and surface water (e.g. Miller and Ortiz, 2007; Hess et al., 1996) and efforts have been initiated to develop innovative approaches to reduce contaminant and nutrient loads due to these facilities (e.g. Costa et al., 2002; Littleton et al., 2003). Since these facilities introduce a variety of pollutants (including organics, nutrients, metals, and a variety of other constituents) into the subsurface, they can impact groundwater quality. Understanding subsurface transport and transformation is especially important in light of the concerns regarding the potential impacts that nutrients and emerging contaminants may have on ground water and surface water supplies. These subsurface disposal facilities are widely used, particularly in areas where the impacts of waste on surface water resources are a concern. As such, it is especially important to understand these impacts.

To illustrate the effects these facilities can have on groundwater, Figure 10.1 shows a schematic that illustrates groundwater flow patterns that may exist downstream of a typical wastewater effluent disposal facility located near a river. The region that is potentially impacted by the effluent disposal is illustrated in the plan view in Figure 10.1(a) and profile view in Figure 10.1(b). For this example, the wastewater effluent flows down through the vadose (unsaturated) zone immediately below the basins and then towards the river as indicated in the sketch (which is not to scale). The physical characteristics of the transport and flow paths down-gradient of the facility are affected by local site characteristics and surface-water/ground-water interactions, as well as the nature of the discharge and associated mounding. After entering the groundwater, wastewater effluent moves down-gradient through the groundwater in the form of contaminant plume that consists of a complex mixture of inorganic and organic chemicals. These chemicals will likely experience some dispersion and mixing with the surrounding groundwater. The chemicals will also experience of variety of transformations. Thus, the potential impacts of these chemicals on ground-water and adjacent surface-water bodies will depend on the nature of the transformations in the groundwater. Unfortunately, the processes governing the transport and transformations of constituents associated with subsurface disposal facilities are complex and difficult to characterize. Basic approaches are still needed to understand the transport, transformations, and attenuation experienced by the nutrients and contaminants of concern as they move through the subsurface.

Accordingly, the objective of this chapter is to address a number of considerations related to subsurface disposal of wastewater effluent, with particular consideration to the implications of geochemical transformations along flow paths down-gradient of subsurface disposal facilities. First, some background is provided on previous work that has been completed to address geochemical transformations that may occur downstream of subsurface discharges from wastewater treatment facilities. In particular, results found by United States Geological Survey (USGS) researchers for a field site in Cape Cod, Massachusetts are reviewed and considered in relation to other active wastewater effluent discharges. Then, some of the key geochemical transformations down-gradient of wastewater treatment facilities are described in more detail, and some approaches for modeling these transformations are described. Finally, a case study is presented to demonstrate the use of inverse modeling techniques to analyze some chemical transformations related to a subsurface effluent discharge site. This analysis is intended to illustrate the key geochemical transformations that are important to consider with respect to subsurface effluent discharges. These results are used to recommend some key areas that need to be addressed in regards to subsurface discharges.

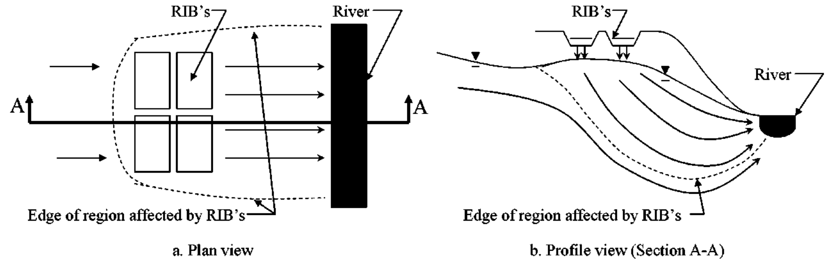


Figure 10.1: Schematic illustrating nature of flow in groundwater due to wastewater effluent discharge located near a river (note - figure is not to scale)

10.2 BACKGROUND

The processes associated with wastewater effluent discharges in groundwater depend on the chemical characteristics of these discharges and on the key processes that may occur in this subsurface. It is important to note that these processes are extremely complex and a full description of the possible processes would be beyond the scope of this paper. Accordingly, this section is intended to provide some general background information on the nature of these processes, with the intention of supporting the topics to be considered in this chapter. Specific background topics include the characteristics of wastewater effluent, some of the key transformations and reactions that impact wastewater effluent in groundwater and some previous research completed at the Cape Cod Toxic Substances Hydrology Research Site which provides insight into some of these transformations.

10.2.1 Wastewater Effluent Discharges

Although treatment facilities remove a large portion of the contaminants from the waste stream before discharge to the environment, the discharges still introduce organic matter, inorganics and nutrients (e.g. metals, and nitrogen and phosphorus), organic chemicals and detergents, bacteria and microorganisms, and potentially toxic constituents into the environment. (LeBlanc, 2006; Barber, 1995) For details on the nature of wastewater effluents, the reader can refer to any reference on wastewater treatment (e.g. Metcalf and Eddy, Inc., 1991). A brief overview of some typical properties is provided for the purposes of this chapter. First, for septic systems, the wastewater effluent discharges typically introduce concentrations of 140 to 200 mg/l of BOD_5, 20 to 60 mg/l of total nitrogen (mostly in the form of ammonium) and 10 to 30 mg/l of total phosphorus into the groundwater. Effluent discharges from conventional secondary wastewater treatment facilities normally introduce less than 10 to 30 mg/l BOD_5, 10 to 30 mg/l of total nitrogen, and 3 to 10 mg/l of total phosphorus. Of course, advanced treatment processes are commonly employed that are capable of reducing these concentrations to lower values. For example, essentially all plants are now required to reduce phosphorus to less than 0.1 to 0.2 mg/l. Nevertheless, the complex mixture of organic and inorganic chemicals discharged into groundwater warrants careful consideration when evaluating impacts.

10.2.2 Transformations and Reactions Down-gradient of Effluent Wastewater Discharges

Transformations in groundwater down-gradient of wastewater effluent discharges may include biogeochemical transformations, aqueous complexation reactions (including reactions involving both the effluent and any mixed groundwater), and reactions involving the soil matrix (including surface complexation reactions and potentially precipitation/dissolution reactions).

Biogeochemical Transformations

The wide variety of organic and inorganic contaminants in the wastewater effluent can lead to especially complex biogeochemical transformations. These processes and interactions have been addressed by a number of investigators (e.g. Flipse, 1984; LeBlanc, 1984, Robertson et al., 1991, Sumner and Bradner, 1995, Wilhelm et al, 1994; and others). Some of the reactions may involve consideration of different oxidation-reduction zones, and quite often involve bacterially mediated biodegradation processes. As such, concentrations of O_2 are especially important to consider for aquifers that have been contaminated by partially and fully treated effluent from wastewater disposal facilities and septic systems.

As described by Stumm and Morgan (1981), any closed aqueous system containing organic matter may result in an environment that accommodates a succession of oxidation-reduction reactions, leading to a sequence of decreasing levels in oxidation state and normally involve complex interactions with bacteria (e.g. Harvey et al., 1984). The organic matter (presented in simple fashion as CH_2O) can serve as the electron donor for a sequence of electron acceptors, including constituents such as O_2, NO_2 and NO_3, $MnO_2(s)$, $Fe(OH)_2(s)$. Related processes may include aerobic respiration, nitrification, manganese reduction, and iron reduction. As an example, for septic system effluent, Wilhelm et al (1994) noted that movement in the presence of oxygen in the unsaturated zone can lead to oxidation of organic carbon and ammonium. Subsequent movement in anoxic zones may lead to denitrification of NO_3^- to N_2 as well (e.g. Smith and Duff, 1988).

Aqueous Complexation Reactions

In addition to the biologically-mediated oxidation-reduction reactions noted above, the chemical mixture contained within the effluent will experience complexation reactions within the aqueous solution. The speciation for the set of chemical constituents is often estimated by assuming that the reactions proceed to equilibrium and that the mass action laws can be defined in terms of established equilibrium constants. Equilibrium constants for reactions various reactions are often defined in terms of established constants. (e.g. as defined in textbooks such as Morel and

Hering, 1999) The reactions are certainly highly dependent on numerous properties associated with the solution (including ionic strength, activity, redox state, temperature, possible rate limitations, etc.)

Reactions Involving the Soil Matrix

Interactions with the soil matrix can have an important effect on transformations and transport associated with wastewater effluent discharges. These interactions generally include surface complexation reactions and precipitation-dissolution reactions. It is important to note, however, that these interactions will naturally be highly dependent on the properties of the soil matrix as well as the aqueous solution. For example, the interactions that may occur in an aquifer with predominantly carbonates will likely be significantly different than those that would occur in an aquifer with predominantly silicates. Accordingly, for purposes of this discussion, the soil matrix is assumed to consist of silica sand, with hydroxides of various metals (including primarily iron and manganese for this case). This sandy soil type is typical of the glacial outwash environment at the field site in Cape Cod, and relatively appropriate for the field site of the case study discussed in the next section.

Precipitation-dissolution reactions that may be possible will depend on the constituents in solutions (including the redox conditions) and on the properties of the soil as well. As such, a full description of the possibilities is considered to be beyond the scope of this current discussion, and the reader is referred to other references (e.g. Morel and Hering, 1999) for detailed consideration of the possibilities. In general, for silica sands with hydrous ferric oxide coatings (with iron and possibly manganese coatings), it is recognized precipitation-dissolution reactions may involve silicates and manganese and iron hydroxides. It is also recognized that precipitation of phosphorus is highly likely.

Surface complexation reactions for silica sands with hydrous ferric oxide coatings are described in detail in Dzombach and Morel (1990). For many cases, surface complexation reactions often require the use of advanced surface complexation models with varying (strong and weak) site affinities to characterize the chemical interactions with the sediments. Considerations include the specific surface area and available surface site concentrations. Other analyses that have incorporated surface complexation reactions include the research of Parkhurst et al. (2003), Kent et al. (2000), and Kent et al. (2002). These include surface complexation constants for the hydrous ferric oxide complexes and for surface complexes of the various cations. Other investigators have used simple exchange relationships to characterize these interactions (e.g. Stollenwerk and Parkhurst, 1999).

10.2.3 Research at the Cape Cod Toxic Substances Hydrology Research Site

In Cape Cod, MA, a series of investigations have been completed that have provided insight into the various geochemical transformations occurring in groundwater affected by the subsurface discharge of wastewater effluent. This site served as basis for a series of investigations by researchers associated with the Cape Cod Toxic Substances Hydrology Research Program (LeBlanc, 2006). For example, well arrays were also installed at the site for conducting detailed experiments to characterize key physical, chemical, and biological processes. These experiments provided the opportunity to assess the physical, chemical, and biological processes experienced by tracers injected into the various contaminated and uncontaminated zones of the plume (e.g. Kinner et al., 2006; LeBlanc et al., 1991; Smith et al., 2006). Because of its relevance to subsurface wastewater effluent discharges, some of the research at the site is summarized here.

Most of the investigations at the site have focused on addressing the characteristics of the Ashumet Valley (AV) Plume, a large region of contaminated groundwater that resulted from an effluent discharge associated with a wastewater treatment facility. The overall extent of the plume, which is shown in Figure 10.2, exceeds 4.8 Km (3 miles). As initially shown by LeBlanc (1984), the AV Plume introduced a variety of contaminants, including phosphorus and nitrogen compounds, metals, organics, detergents, and bacteria to the groundwater. The source area that existed for this plume is a short distance upstream of a pond, which has received significant quantities of phosphorous and other constituents due to the plume discharge. (LeBlanc, 1984) The subsurface within the plume includes an extensive zone of sorbed organic matter and three geochemical zones separated by steep gradients: an anoxic zone with no dissolved oxygen (along with high concentrations of iron and manganese and negligible nitrate) located immediately down-gradient of the disposal beds, an oxic zone with higher concentrations of nitrate around and further down-gradient of this core region, and a sub-oxic transition zone between these two regions. The anoxic zone extended from the disposal beds beyond Ashumet Pond (as shown in Figure 10.2).

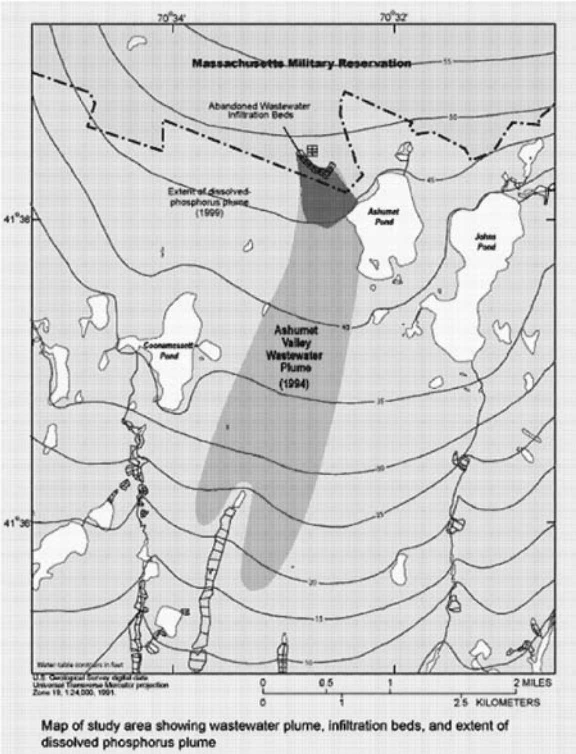


Figure 10.2: Ashumet Valley Effluent Wastewater Plume (graphic courtesy of USGS)

The wastewater disposal facility at this site (operating to secondary treatment levels) was maintained via effluent disposal beds from 1936 until 1995, at which time the discharge was stopped and relocated to another site. Even though the discharge was removed in 1995, the contamination at the time of this discussion (in 2008) remains extensive and an anoxic zone still remains. The persistence of an anoxic zone for an extended period after the removal of the discharge highlights the importance of understanding the processes that consume oxygen as it moves into the contaminated zone beneath the sewage disposal beds. For example, after the cessation of the discharge, biodegradation of sorbed organic matter has continued to deplete oxygen even though the source of organic matter was eliminated (LeBlanc et al., 1999; Barber and Keefe, 1999). The reduction in the oxygen supply (which decreased after the discharge) was found to lead to a decrease in nitrification (Smith et al., 1999a; Repert et al., 2006). Phosphate and metals exhibited little change in mobility (Kent and Maeder, 1999), although Kent and Fox (2004) found that plume changed the chemistry of the aquifer, resulting in a release of naturally-occurring, sediment-bound arsenic back into the water column. Reactive transport simulations using the PHAST model to simulate transport of phosphorous in the sewage plume under pre- and post-cessation conditions at the Cape Cod site (Parkhurst et al., 2003) showed that the high pH and reducing conditions near the disposal beds and substantial phosphorous loads could be maintained for decades into the future. For this model, the extent of decomposition of organic carbon following cessation of sewage disposal, as well as the ability to characterize cation sorption reactions, remained as sources of uncertainty.

In fall 2001, Mathisen et al. (2003) and a number of investigators with the USGS completed an experiment at a field site in the AV Plume. The objective of this experiment was to describe the processes governing oxygen consumption in a contaminated aquifer experiencing natural attenuation. For this experiment, they maintained a three-month, steady injection of uncontaminated groundwater into the contaminated zone of anoxic groundwater (with no dissolved oxygen) directly below one of the former sewage-effluent disposal beds. By sampling from an array of 12 multi-level sampling wells, they were able to monitor the dissolved oxygen and variety of other water quality parameters in the groundwater downstream of the injection. Using the laboratory and field results, a reactive transport model was developed that could characterize the flow, transport, and chemical interactions between the constituents in the water and on the soil surfaces. The simulations were used to show that the restoration of oxygen in this anoxic region is governed by biodegradation processes involving oxidation reactions. These oxidation processes involve the consumption of oxygen and utilization of other constituents released from the sediments (e.g. organic carbon and ammonium). Still, characterization of physical, chemical and biological processes associated with these oxidation processes remains as a significant technical challenge.

10.3 BACTERIALLY-MEDIATED OXIDATION-REDUCTION REACTIONS IN GROUNDWATER AFFECTED BY WASTEWATER EFFLUENT DISCHARGE FACILITIES

Bacterially-mediated oxidation-reduction reactions often have an important effect on the characteristics of various constituents downstream of wastewater effluent discharges. The nature of these reactions often depends on the concentration of dissolved oxygen in the groundwater. When oxygen is present at sufficient concentrations, aerobic respiration and nitrification are two key processes that often occur. Aerobic respiration is commonly represented using the following reaction:

$$CH_2O + O_2 = CO_2 + H_2O \quad (1)$$

where CH_2O serves as a simple representation of organic carbon. While wastewater treatment facilities are effective at reducing the organic matter, some concentrations of CH_2O are still expected to be maintained in the wastewater effluent.

Nitrification, or ammonium oxidation, is often represented in terms of a two-step reaction in which nitrosomonous oxidizes ammonium (NH_4^+) to form nitrite (NO_2^-) and nitrate. The NH_4^+ is often derived via the breakdown of organic forms of nitrogen and is introduced as part of the effluent for the case of effluent wastewater discharges. A common representation takes the form:

$$2NH_4^+ + 3O_2 = 2NO_2^- + 4H^+ + 2H_2O \tag{2}$$

and nitrobacter oxidizes nitrate to form nitrate (NO_3^-) in accordance with:

$$2NO_2^- + O_2 = 2NO_3^- \tag{3}$$

The second step of this process (Equation 3) is generally considered to be a fast reaction.
As such, for the purposes of this discussion, it is reasonable to represent the process in terms of the following simplified reaction:

$$NH_4^+ + 2O_2 = NO_3^- + 2H^+ + H_2O \tag{4}$$

If the CH_2O or NH_4^+ or are depleted and sufficient oxygen is present, it is possible that other constituents (such are Fe^{2+} or Mn^{2+}) may be oxidized, provided that these constituents are present at sufficient concentrations.

Second, if the oxygen has been depleted, other species may serve as the electron acceptor. For example, if sediments include concentrations of iron and manganese (or if these constituents are included in the effluent), iron or manganese reduction may be possible. Iron reduction is represented as

$$Fe(OH)_2 + 1/4\ CH_20 + 2H^+ = 1/4\ CO_2\ (g) + Fe^{2+} + 11/4\ H_2O \tag{5}$$

and manganese reduction is represented as

$$MnO_2(s) + 1/4\ CH_20 + H^+ = 1/4\ CO_2\ (g) + 1/4\ Mn^{2+} + 1/4\ H_20 \tag{6}$$

Subsequent movement in anoxic zones may lead to denitrification of NO_3^- to N_2. Here, CH_2O may also serve as an energy source for denitrifying bacteria, in which case the reaction is represented as:

$$1/4\ CH_2O + 1/5\ NO_3^- + 1/5\ H^+ = 1/4\ CO_2\ (g) + 1/16\ N_2 + 7/20\ H_2O \tag{7}$$

Which takes place under anoxic (or possibly under locally anoxic) conditions. If sulfate is available, sulfate reduction may occur, in accordance with the following representation:

$$\tfrac{1}{4}\ CH_2O + 1/8\ SO_4^{2-} + 1/8\ H^+ = 1/8\ HS^- + \tfrac{1}{4}\ CO_2 + \tfrac{1}{4}\ H_2O \tag{8}$$

Finally, methane formation may take the form:

$$CH_2O + \tfrac{1}{4}\ CO_2\ (g) = 1/2\ CH_4 + CO_2\ (g) \tag{9}$$

For each of the reactions noted in Equations 1 though 9, the likelihood and rates of occurrence will depend on many factors (e.g. thermodynamic considerations, availability of substrates and electron acceptors, favorability of environmental conditions for growth of heterotrophic and/or autotrophic organisms, etc.) It is also recognized that this is not a complete set of oxidation-reduction reactions that can play a role in subsurface biodegradation processes. The occurrence of specific oxidation-reduction reactions will depend on thermodynamic considerations, the nature of the microbial communities that may potentially develop, and other factors. Given the complexity of these processes, identifying the key biodegradation processes that govern transport down-gradient of wastewater effluent discharges can be extremely difficult.

10.4 MODELING OF TRANSPORT AND TRANSFORMATIONS RELATED TO WASTEWATER DISCHARGES

10.4.1 Models for Characterizing Transformations Related to Wastewater Effluent Discharges

A variety of approaches have been used to model biodegradation in the subsurface, and a number of these approaches have been applied to address transformations down-gradient of wastewater effluent discharges. For example, biodegradation kinetics has been modeled using first order decay kinetics, Monod kinetics, and instantaneous reaction kinetics. (Bedient et al., 1999) An example of an established numerical model for characterizing biodegradation (and natural attenuation) includes the BIOPLUME III model, which is a two-dimensional, finite difference model for simulating the effects of advection, dispersion, sorption, and biodegradation on organic contaminants in ground water, with an allowance for a variety of aerobic and anaerobic electron acceptors (Rafai, 1998).

Due to the complex chemical reactions related to wastewater effluent discharges, some of the more recent modeling approaches to characterize these transport processes include the use of multi-component reactive transport models. Often, rate limited reactions are required to characterize some of the processes, such as bacterially mediated oxidation-reduction processes. One example is provided by MacQuarrie and Sudicky (2001a and b), who developed a multi-component reactive transport model that demonstrates many of the key parameters to consider when sewage effluent is discharged into groundwater. The model was used to simulate nitrogen and carbon transport processes in the vadose and saturated zones following introduction into a shallow unconfined aquifer. The approach made use of abiotic reactions with thermodynamic equilibrium constants developed in accordance with Stumm and Morgan (1981) along with biogeochemical kinetic reactions for oxidation processes that were developed using multiple-Monod type expressions as outlined by Chen et al. (1992) and Essaid et al. (1995). Using parameters derived from literature, along with this approximate model, the investigators characterized the processes for a groundwater plume resulting from wastewater contamination at a field site in Ontario. Their research revealed the importance of identifying and quantifying the primary processes governing the fate, transport and complex biogeochemical reactions for cases in which sewage is being introduced into groundwater.

Other multi-component reactive transport simulations include the modeling effort completed by Parkhurst et al. (2003), which provided some insights as discussed previously. This model made use of the PHAST software package to simulate transport of phosphorus in the sewage plume at the Cape Cod site under pre- and post-cessation conditions. This model utilized an organic carbon desorption/reaction process that accommodated the associated reduction of the most thermodynamically favorable electron acceptor (O_2, NO_3^-, MnO_2 or $Fe(OH)_2$). The PHAST software package was also employed by Colman (2005) to characterize transport and transformations of phosphorus, and develop a set of response curves for the purpose of estimating phosphorus plume lengths downstream of active effluent disposal facilities in sand and gravel aquifers. In general, the approaches that make use of multi-component modeling tools can effectively characterize the transport and transformation down-gradient of wastewater effluent disposal facilities, but do have relatively extensive data requirements to sufficiently represent the site-specific processes.

10.4.2 Geochemical Inverse Modeling Using the PHREEQC Package

Given the complexity of geochemical reactions in the subsurface, along with the extensive data needs required to develop multi-component transport models, simpler approaches could be helpful to provide for a basic understanding of key processes that may exist at a given site. Accordingly, this section describes the use of inverse modeling as a basic tool for understanding the key biogeochemical processes that may affect transport of wastewater effluent at a wastewater site. The inverse modeling approach presented here, and employed in the case study presented in this chapter, makes use of a modeling tool provided by the PHREEQC software package. (Parkhurst and Appelo, 1999)

The inverse modeling approach incorporated into the PHREEQC package provides a means for using data from samples collected at two (or more) locations along a flow path to interpret the key processes that may explain the observed changes in concentration. As detailed by Parkhurst and Appelo (1999), the inverse modeling routine PHREEQC essentially solves a set of linear equations associated with mole balances for each of the elements in the flow system. It also provides the ability to incorporate electroneutrality constraints and mixing with a third solution as appropriate. Mole balance equations are set up for the various elements, alkalinity, electrons, water, and for each isotope. In addition, a charge balance equation is included for each aqueous solution, along with an equation to relate the uncertainty terms for pH, alkalinity, and dissolved organic carbon (Parkhurst and Appelo, 1999). The unknowns represent mole transfers of minerals or gases to or from aqueous solution, mole transfers for each valence states of each redox element, and a set of constraints to allow for uncertainty. The constraints allow for the specification of limits on the size of the uncertainty terms and also on the sign of the mole transfer of reactants.

The reader is referred to Parkhurst and Appelo (1999) for a more detailed overview of the capabilities of the PHREEQC package, although a few of the key equations are presented for the purposes of this chapter. First, the mole balance equation, illustrated here for a specified element, takes the form

$$\sum_{i=1}^{I} c_i \alpha_i \left(T_{m,i} + \delta_{m,i}\right) + \sum_{j=1}^{J} c_{m,j} \alpha_j + \sum_{k=1}^{K} c_{m,k} \alpha_k = 0 \tag{10}$$

where α_i is defined as the mixing fraction of each aqueous solution, α_j is defined as the mole transfers from solids or gases into or out of solution, and α_k is defined as the mole transfers between valence states of redox elements. In equation 10, I is the number of aqueous solutions, J is the number of reactive phases, and K is the total number of redox reactions. In addition, c_i is a constant (that is equal to 1 if i is less than I, and -1 if i equals I), $T_{m,i}$ represents the number of moles in solution i, $c_{m,j}$ is the coefficient of master species m in the dissolution reaction for phase j, $c_{m,k}$ is the stoichiometric coefficient of the secondary master species k in redox reaction k, and $\delta_{m,i}$ is an uncertainty term for solution i.

The form of an alkalinity equation is similar to that in Equation 10, while the electron balance takes the following form:

$$\sum_{j=1}^{J} c_{e,j} \alpha_j + \sum_{k=1}^{K} c_{e,k} \alpha_k = 0 \tag{11}$$

where $c_{e,j}$ is the number of electrons released in aqueous redox reaction j, and $c_{e,k}$ is the number of electrons released or consumed in the dissolution reaction for phase k. Also, a charge balance equations takes the form:

$$\sum_{m=1}^{M} \bar{z}_m \delta_{m,i} = -T_{z,i} \tag{12}$$

where $T_{z,i}$ is a charge imbalance in solution i (as determined via a speciation calculation) and $\bar{z}_m$ is the charge assigned to the relevant master species (plus the alkalinity assigned to the master species). Finally, a relationship among the pH, alkalinity, and total dissolved inorganic carbon uncertainty terms is given as

$$\delta_{Alk,i} = \frac{\partial Alk_i}{\partial C_i} \delta_{Ci} + \frac{\partial Alk_i}{\partial pH_i} \delta_{pH,i} \tag{13}$$

where Alk_i is the alkalinity of solution I, and C_i is the total inorganic carbon of solution i, and the partial derivatives are evaluated numerically for each solution.

As described in Parkhurst and Appelo (1999), the resulting system of equations is linearized by letting $\varepsilon_{m,i} = \alpha_i \, \delta_{m,i}$. This step leads to set of linear equations (and inequality constraints) that can be solved using an algorithm that minimizes the sum of absolute values and constraints, as per Parkhurst and Appelo (1999), who follow the approach of Barrodale and Roberts (1980).

Conceptually, as illustrated in Figure 10.3, the approach involves the selection of a set of samples along a flow path, estimation of a complete chemical speciation for each of these samples, and the determination of the transformations that could account for the differences in molar amounts observed for the various elements in the selected samples. Possible transformations include addition (or removal) via precipitation and dissolution reactions, surface exchange reactions, and/or oxidation/reduction reactions.

Procedurally, for a wastewater effluent discharge, the inverse modeling analysis is completed using input defined by samples collected from two wells along a flow path. For instance, the two samples could include a sample of wastewater effluent, and another sample collected from a well located along a flow path down-gradient of the discharge. The software input includes the definition of solution master species, output characteristics, solution data (from field sampling and analyses), inverse modeling parameters, exchange species, and phases. Most of the solution master species and reactions (including stoichiometry and thermodynamic data), exchange species, and phases can be obtained from the PHREEQC database provided by Parkhurst and Appelo (1999), and any additional required input can be provided by the user. The input for the inverse modeling routine includes the solutions and phases to be considered for the inverse analysis, and also the various constituents in the solutions that will be balanced within a specified uncertainty. In general, the approach provides a simple basis for discussing the results of a case study involving a facility that discharges wastewater effluent into groundwater.

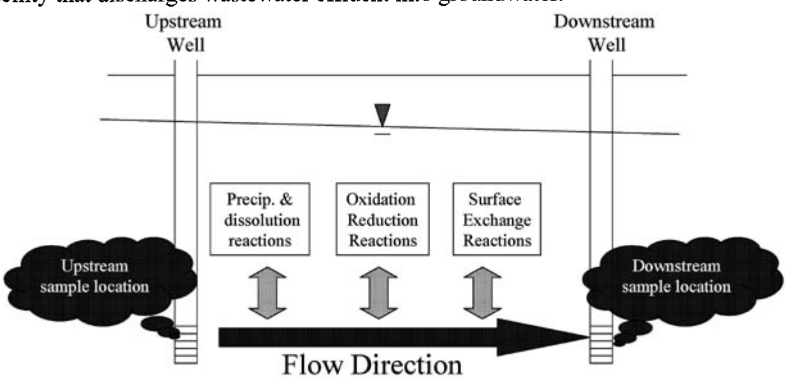


Figure 10.3: Inverse Modeling Concepts

10.5 CASE STUDY – TRANSPORT & TRANSFORMATIONS DOWNSTREAM OF A WASTEWATER EFFLUENT DISCHARGE

Recent research at Worcester Polytechnic Institute (WPI) has included an effort to gain an improved understanding of the transport and transformations that occur in groundwater down-gradient of subsurface wastewater disposal facilities. This initiative includes monitoring and characterization for a number of wastewater effluent discharge facilities. The sites that have been considered included facilities with a range of loading history, including relatively new facilities (5 and 15 years old) and an older facility utilizing infiltration beds as a treatment process.

While a few different treatment facilities are of interest (including two other facilities that have already been sampled), the focus of this paper is on results associated with a wastewater treatment facility in Acton, MA. This facility was selected because the site appears to be well-suited for initiating a long-term sampling and analysis program, and because it would appear to provide a useful basis for comparison for use when evaluating transport and transformations down-gradient of other effluent disposal facilities. It is emphasized that this particular treatment facility is relatively new and it is meeting all required discharge requirements. Accordingly, this case study is provided to illustrate some of the key processes to be considered, rather than a specific assessment.

10.5.1 Site Description

The process for this facility, shown in Figure 10.4, includes screening and grit removal, two sequencing batch reactors (SBRs) followed by a flow equalization, filtration, and ultraviolet (UV) disinfection. (Town of Acton, 2008) The SBRs are considered to be an effective approach for reducing phosphorus and nitrogen releases into the environment. Each SBR provides a batch activated sludge process that allows for programmed cycles for filling and drawing or decanting. The average design flow is 459,421 cubic meters per day (CMD) (121,540, gallons per day, GPD) with a peak design flow of 945,000 CMD (250,000 GPD). Various chemical additions may include soda ash (Na_2CO_3) for alkalinity control, aluminum sulfate and metal salts for phosphorus control, acetic acid for denitrification control, and sodium hypochlorite for odor control.

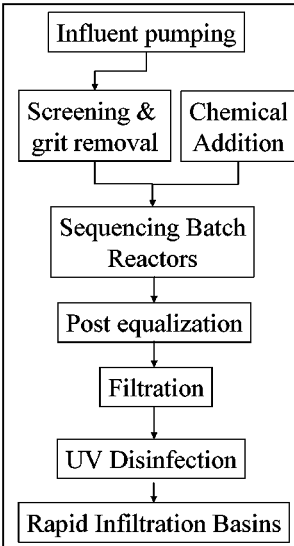


Figure 10.4: Acton WWTF Process Diagram

The effluent beds, shown in Figure 10.5, are designed for a maximum application rate of 61.0 CMD/m^2 (1.5 GPD/ft^2). Soil below the infiltration beds includes 0.91 to 2.44 meters (3 to 8 feet) of fine sandy silt, underlain by 10 feet of fine to medium sandy silt, which is above 5.49 to 6.71 meters (18 to 22 feet) of gray till. The groundwater table is estimated to be at a depth of 3.35 meters (11 feet) below the ground surface under normal conditions, and 2.13 meters (7 feet) below the surface for maximum mound conditions. The hydraulic conductivity of the soil is approximately 9.1 to 15.2 meters/day (30 to 50 ft/day). The elevations of the groundwater table are approximately 35.6 meters (117 feet) at the beds, 48.7 meters (160 feet) at Well G3, and 40.8 meters (134 feet) at the stream.

10.5.2 Field and Laboratory Methods

Characterization of the wastewater effluent discharge for the Acton, MA facility included field sampling and laboratory analyses. The field sampling effort included field visits in the summer and fall of 2007. Sampling included collection of effluent samples, groundwater samples, and surface water samples in the river adjacent to the facility (which are not discussed further since they are not included in the scope of the present discussion). Groundwater samples were collected from monitoring wells located down-gradient of the subsurface disposal beds. The sampling locations can be seen in Figure 10.5. For the initial field visit, groundwater samples were collected with a bailer, after initially purging the wells. In subsequent visits, groundwater samples were collected (after purging the wells as before) using a Geotech Keck SPR sampling pump. The field program included monitoring for dissolved oxygen, temperature, pH, and conductivity. Samples were also collected for alkalinity, pH, cations (total and dissolved), nitrogen compounds (including total and dissolved ammonium, nitrite, and nitrate), phosphorus compounds (total and dissolved phosphate), other anions, dissolved organic carbon, and total sediments.

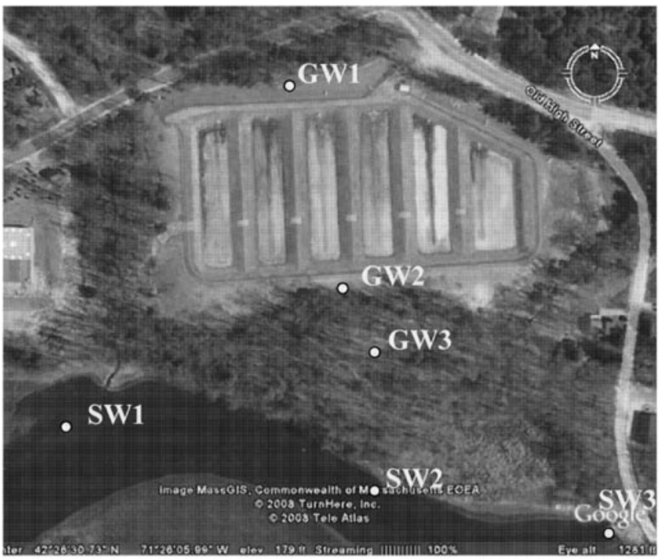


Figure 10.5: Acton WWTF Aerial View – showing sampling locations (background image from MassGIS, Commonwealth of Massachusetts EOAC)

Sample collection methods for this research were developed in accordance with many of the procedures presented in Savoie et al. (1997), as feasible. Laboratory analyses of cations (except for NH_4^+) were completed using a Perkin-Elmer atomic absorption (A/A) spectrophotometer, while laboratory analyses of anions (including

NH_4^+) were completed using colorimetric methods (in most cases making use of a Hach DR/3000 spectrophotometer and associated methods available from Hach). For the metals, filtered and unfiltered samples were digested using nitric acid, allowing for the determination of total and dissolved concentrations. Because of the importance of accurate phosphorus determinations, careful techniques were used when preparing phosphate samples for analysis (including the use of a sulfuric acid-nitric acid digestion method, which was found to yield accurate results for phosphorus). The approaches allowed for the consideration of phosphorous compounds (total and dissolved phosphate) and nitrogen species (including ammonium, nitrite and nitrate) as well.

It is noted that the basic field and laboratory procedures were confirmed quantitatively through initial analyses and consideration of a wastewater-contaminated aquifer located adjacent to the Massachusetts Military Reservation (MMR) in Cape Cod, MA. Due to the extensive research and available data at that site, the site provided an excellent location for validating field techniques and for analyzing data to understanding processes related to wastewater discharges. A multi-level sampler (MLS) well, sampled at different depths, provided a basis for validating field and laboratory analyses under different geochemical conditions. (Mathisen, et al., 2007)

10.5.3 Results of Field Monitoring and Laboratory Analyses

The results of the field sampling and laboratory analyses are shown in Table 10.1. This table includes information on background concentrations, the effluent characteristics, and three monitoring wells (GW1, GW2, and GW3) down-gradient of the treatment facility. Samples collected from GW1 and GW2 were likely directly affected by the wastewater discharge, with little or no contribution from the background groundwater. Samples collected from GW3 were likely affected to some extent by contributions from the background. Since these samples were all collected in one afternoon, the samples only represent a snapshot of the effluent and groundwater characteristics downstream of the plant. It is also noted that the facility is operating effectively within its discharge requirements.

Accordingly, the results are presented here to illustrate trends and relationships on subsurface processes. First, the effluent samples did include concentrations of NH_4^+ and DOC, which decreased significantly in the wells downstream. Thus, some organic carbon and nitrogen are being introduced into the groundwater (as would be expected from a treated wastewater effluent). Second, the PO_4^- concentrations are quite low in this case due to the effective advanced phosphorus treatment processes, and also decrease with distance downstream. As such, phosphorus transport is not considered in great detail for further analyses. In addition, the wastewater effluent is relatively high in oxygen, since the effluent equilibrates with atmosphere as it flows from the sequencing batch reactors to the infiltration beds and then into the subsurface zones beneath the beds. Finally, since the discharge includes various inorganic constituents (that differ from the concentrations in the groundwater), it is recognized that the aqueous complexation reactions, and also surface complexation reactions are likely important.

Table 10.1: Acton sampling results

	Background (μM)	EFFLUENT (μM)	GW1 (μM)	GW2 (μM)	GW3 (μM)
pH	6.41	7.23	6.02	6.08	6.3
Alkalinity	370	1321	369	408	555
TCO_2	682	1484	1150	1160	1170
Ca_t	494	491	324	399	434
Mg_t	177	189	147	156	165
Na_t	813	3664	3987	3540	3256
K_t	187	494	159	197	141
Fe_t	0*	0*	0*	0*	0*
Mn_t	32	1.91	1.2	0.47	0.47
Cl_t	451	3080	3080	3077	2775
SO_4^{2-}	593	683	748	631	578
NH_4^+	0*	25.5	4.8	0.06	7.59
NO_3^-	0*	15.15	13.4	17.2	33.6
PO_4^-	0*	7.9	2.05	3.18	4.74
O_2	--	252	164	260	150
DOC	0*	666	2.36	2.03	1.64

* = below detection limit (listed as zero)

10.5.4 Approach and Inverse Model Development

The inverse model that was developed to characterize the Acton wastewater effluent case study included consideration of two cases to characterize the processes down-gradient of the discharge. This section provides an overview of the approach used for the inverse model development, with consideration to oxidation-reduction reactions, aqueous complexation reactions, surface complexation reactions, and selected precipitation reactions.

First, for the oxidation-reduction reactions, model development included consideration of carbon and ammonium oxidation, manganese reduction, and denitrification. With plenty of carbon, oxygen and nitrogen available, it was anticipated that rate limited oxidation reactions would likely include aerobic respiration (represented in terms of Equation 1) and nitrification (represented in terms of Equation 4). Manganese reduction was assumed to be a possible process since manganese was present in the discharge, and since manganese concentrations were relatively high down-gradient of the facility. This process would lead to the formation of the precipitate MnO_2. Finally, it is recognized that denitrification is a possible process that may exist in the groundwater down-gradient of the discharge. Since this process normally occurs under anoxic conditions (and there weren't enough sampling locations to ascertain whether conditions near the discharge were oxic or anoxic), two cases were considered: Case 1 (which assumed that no denitrification was occurring), and Case 2 (which included the possibility of a denitrification process.) The denitrification was assumed to take the form indicated by Equation 7, and it was only included in the second case considered (Case 2).

Second, aqueous complexation reactions were defined using the constituents noted in Table 10.1 along with the database for reactions and equilibrium constants provided by the PHREEQC software package (Parkhurst and Appelo, 1999). Use of the database provided for a complete computation of chemical speciation/complexation, which could be checked by confirming that electroneutrality was maintained.

Third, surface complexation reactions would formally be defined in terms of surface species with strong and weak affinities for surface sites. However, for the purposes of this analysis, however, these reactions were represented in terms of a simple ion exchange model with the following reactions:

$$CaX_2 = Ca^{2+} + 2X^- \tag{14}$$

$$MgX_2 = Mg^{2+} + 2X^- \tag{15}$$

$$MnX_2 = Mn^{2+} + 2X^- \tag{16}$$

$$NH_4X = NH_4^+ + X^- \tag{17}$$

$$NaX = Na^+ + X^- \tag{18}$$

$$KX = K^+ + X^- \tag{19}$$

$$HX = H^+ + X^- \tag{20}$$

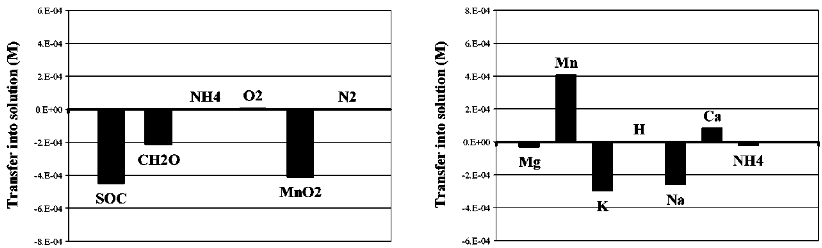
Note that Equation 19 is also included for hydrogen, which helps to account for the role of hydrogen ions in the surface complexation reactions associated with the hydrous ferrous oxide surfaces. It is recognized that anions (such as phosphates and silicates) may also bond to surface sites on the hydrous ferrous oxide coatings for the silica sediments. As such, the anions could also be characterized using surface complexation reactions, or more simply using ion exchange reactions. However, with respect to the total anions in solution, it is expected that the effect of these interactions would be smaller than the effect of precipitation/dissolution reactions. Therefore, anion exchanges with the surface are not considered separately in the current analysis.

Finally, it is recognized that a variety of precipitation/dissolution reactions may exist that can affect concentrations of phosphorus, iron, and manganese. A number of reactions associated with iron and manganese are also microbially-driven (as noted in Equations 5 and 6). While many forms may exist for precipitates with these constituents, a few representative precipitates were specified for these constituents to assess the potential for these processes. These representative precipitates were defined with consideration to the previous modeling work of Parkhurst et al (2003). Potential precipitates of phosphorus include hydroxyapatite ($Ca_5(PO_4)_3OH$) and vivianite ($Fe_3(PO_4)_2{:}8H_2O$). Again, it is recognized that the phosphate concentrations are relatively low, and the use of these simple precipitates provide a simple approach for accounting for the loss of phosphate. One of the potential precipitates of manganese includes pyrolusite, $MnO_2(s)$, which is actually associated with manganese reduction via Equation 5. $Fe(OH)_3(s)$, which is associated with iron reduction via Equation 6, is a possibility but it is anticipated that this reaction is not likely since concentrations of iron were below detection.

With these definitions to define the key processes, inverse analyses using the PHREEQC software model could be used to identify possible transformations and interactions with soils. These results were also compared with the basic conditions from the Cape Cod site (although the analysis recognized the fact that each of these sites would natural have site specific characteristics).

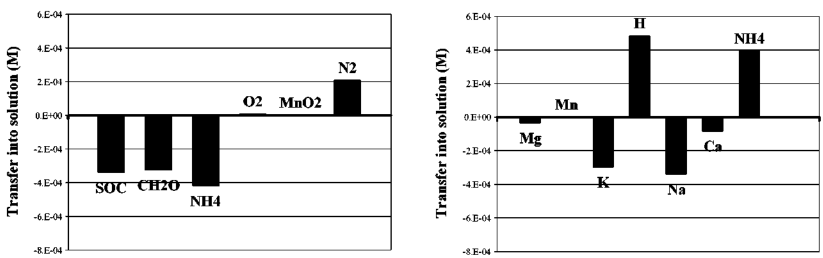
10.5.5 Results for a Basic Inverse Model Using the PHREEQC Package

Inverse modeling was considered for the effluent (EFF1) and for a number of sampled wells (GW1, GW2, and GW3) down-gradient of the Acton WPCF. For the purposes of this paper, inverse model results are presented for the solutions of the effluent (EFF1) and one sampled well (GW2) only. The analysis included an uncertainty of 6% and a tolerance of 1×10^{-10} M. The results reflect a straight analysis of the inverse model, and additional characterization of the specific processes is left for future assessment. Two cases were considered: one case with no denitrification (Case 1) and another case with denitrification (Case 2). The results for the two cases are presented in Figures 10.6 and 10.7, respectively. The figures list mole transfers of various constituents entering the aqueous solution, with consideration to oxidation-reduction reactions and cation exchange reactions.



a. Redox & precipitation reactions b. Cation exchange reactions

Figure 10.6 - Acton WPCF inverse modeling results (Case 1 – no denitrification)



a. Redox & precipitation reactions b. Cation exchange reactions

Figure 10.7 - Acton WPCF inverse modeling results (Case 2 – with denitrification)

The results for Case 1 are shown in Figures 10.6a and 10.6b. As discussed in the previous section, these results allow for carbon oxidation (aerobic respiration) and ammonium oxidation (nitrification) as oxidation-reduction reactions. For this case, aerobic respiration is likely small, since there aren't any significant decreases in oxygen concentrations with distance downstream. The results of the inverse modeling for oxidation-reduction reactions (shown in Figure 10.6a) suggest that a portion of organic carbon is being sorbed by the sediments (indicated as SOC) and the remaining portion (indicated as CH2O) is being utilized in conjunction with oxidation-reduction processes. With this inverse model, the results for cations (shown in Figure 10.6b) suggest that the Mn^{2+} released from the sediments and leading to the precipitation of pyrolusite. Other cations, including Mg^{2+}, Ca^{2+}, K^+, and NH_4^+ are all removed from solution. The phosphorus concentrations are low in the effluent, and any phosphorus is likely sorbed to the sediments and removed by precipitation (set to be in the form of hydroxyapatite for simplicity). Since mole transfers for phosphorus are very small in relation to the scales in Figures 6a and b, they are not shown. The ammonium concentrations indicate that NH_4^+ is being sorbed to the sediments, but is also maintaining a nitrification process that is utilizing some oxygen and accounting for an increase in NO_3^-. Since the manganese reaction in this model is actually being driven by desorption from the sediment, followed by precipitation, the process is considered to be a relatively unlikely possibility, and another scenario was considered.

The results for Case 2 are shown in Figures 10.7a and b. These results allow for carbon oxidation (aerobic respiration), ammonium oxidation (nitrification), and also denitrification. The results for the oxidation reduction reactions (in Figure 10.7a) confirm that ammonium oxidation is a possible process. In this case, however, pyrolusite precipitation is not included and Mn^{2+} is simply defined in terms of exchanges with surface sites on the sediment. As for Case 1, the role of aerobic respiration is likely small, since only small changes on oxygen are observed with distance downstream. The results for cations for this model (shown in Figure 10.7b) indicate that NH_4^+ is released into solution from the sediment, while Mg^{2+}, Ca^{2+}, K^+, and Mn^{2+} are removed from solution. Mole transfers for phosphorus are similar to the results of Case 1 and are not shown since they are very small in relation to the scales in the figures. Although denitrification is known to be a process that most likely occurs under anoxic conditions, denitrification is also believed to occur in locally anoxic cells in variable media (as noted by Sumner and Bradner, 1995). As such, this process is considered to represent a possible process that could account for the oxidation-reduction exchanges occurring in the subsurface. For this case, sediments serve as sources of NH_4^+ which is ultimately oxidized by a nitrification process. In addition, denitrification effectively utilizes NO_3^- while generating $N_2(g)$. As such, with this model, the observed increase in NO_3^- with distance downstream reflects both a gain in NO_3^- from nitrification, and loss in NO_3^- from denitrification. Overall, the results for Case 2 indicate that denitrification may be a possibility. However, to fully confirm that this process is occurring, analyses for the presence of the end products of this reaction (i.e. N_2 or N_2O) would be necessary. Additional analyses would also be necessary to gain a complete understanding of the role of organic carbon in these processes.

Again, it is emphasized that these results are based on a straight application of simple inverse models. Additional monitoring and analyses would be the next steps to fully ascertain the key processes involved in these transformations. Key considerations include clarification of the specific oxidation-reduction processes involving carbon and nitrogen. The final determinations in regards to the key processes clearly depend on both the redox and precipitation processes, along with the surface complexation processes (as represented in terms of cation exchange processes).

10.6 CONCLUSIONS

The results of the analyses presented in this chapter confirm the importance of considering geochemical transformations in groundwater down-gradient of subsurface wastewater effluent discharges. The case study, which included consideration of a facility that is operating effectively within its discharge requirements, illustrated some of processes that may exist downstream of these facilities. For this case, inputs of carbon and nitrogen compounds led to the occurrence of biologically mediated oxidation-reduction processes, including nitrification and possibly aerobic respiration. The nature of these processes is closely related to the concentrations of oxygen as well as the nature of oxygen consumption. The oxidation-reduction processes are site specific, although information gained at a single site can serve to provide insight for the processes occurring at other locations. Additional oxidation-reduction processes such as iron and manganese reduction, as well as denitrification, may also have an impact on the groundwater. Given the impact these processes can have, more research is recommended to fully understand these oxidation-reduction processes.

These results also showed that interactions and constituent exchanges with the soil matrix had an important effect on nutrient transport and transformations down-gradient of the wastewater discharge. For this case study, surface exchange reactions affected cation concentrations and transport. The specific cations released to the aqueous solution can affect the types of oxidation-reduction reactions that occur. For example, the release of ammonium from the soil matrix can directly lead to nitrification, which also consumes oxygen and affects the pH.

The inverse modeling used for these analyses provided a helpful tool for gaining an understanding of key processes. In addition to confirming the presence of nitrification, the inverse modeling results confirmed the presence of the surface exchange reactions and additional geochemical processes that potentially affected nitrogen and carbon. By considering the possibility of a denitrification process, the oxidation-reduction processes could be represented using the inverse model.

It is recognized that the inverse modeling approach could not fully eliminate the possibility of other processes, and that non-uniqueness of solutions remains as a possibility. Additional sample analyses (e.g. to characterize N_2 and N_2O), which could not be completed within the scope of this analysis, would help to confirm the end products of the denitrification process and are being considered for future analysis. Furthermore, additional characterization of the thermodynamic considerations, the nature of the microbial communities, and other factors would help to ascertain the specific processes that may be occurring. Finally, while the region below the filtration beds and above the groundwater table (the vadose zone) was assumed to have a small impact for the purposes of this analysis, future research is necessary to understand the details of transport (and possible gains/losses) in the vadose zone. Finally, it is also recognized that additional research includes detailed three-dimensional modeling to identify the nature of the processes along various flow paths, and also to characterize temporal and special variability.

In general, the results demonstrate that nutrients (including nitrogen and phosphorous compounds) can be transported through groundwater downstream of wastewater effluent disposal facilities. The variety of constituents included in treatment discharges (including the constituents added as part of the treatment process) may further impact the geochemical processes in the groundwater affected by these discharges. As such, consideration of relationships among multiple geochemical processes with a complex suite of constituents remains as an important research need for improving our ability to understand the processes related to subsurface disposal discharges and their effects on the quality of groundwater impacted by these wastewater effluent discharges.

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CHAPTER 11

Use of Wetland Systems for Groundwater Recharge in Two Urban Wastewater Treatment Plants in Orlando, Florida

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ABSTRACT: Polishing wastewater with wetlands may help remove nutrients in the wastewater effluents on one hand, and promote groundwater recharge and ecosystem health on the other hand. There are essentially two general types of domestic wastewater wetlands, including natural and constructed wetlands. Some domestic wastewater facilities utilize a combination of both. According to Florida Department to Environmental Protection, there are 17 natural (both treatment and receiving) wastewater wetlands comprising roughly 2508.5 Km^2 (6,200 acres) and a total of 21 constructed wetland sites comprising roughly 1618.4 Km^2 (4,000 acres) across Florida. Of those, 5 facilities use a combination of both natural and constructed wetland systems, making that a total of 33 permitted domestic wastewater wetland sites in the state of Florida. This chapter aims to introduce two of them, including the newly developed Northwest wastewater reclamation facility (NWRF) and Lake Marden Treatment Wetlands/Augmentation Project that has been started since 2005 and the long-standing Eastern WRF and Treatment Wetlands in Orlando, Florida to illuminate the comparative advantages for minimizing the urbanization effects due to economic development. The treatment wetland at the NWRF was implemented to increase the plant's reclaimed water management capacity from 17 million liters per day (Ml/d) [4.5 million gallons per day (MGD)] annual average daily flow (AADF) to 28.5 Ml/d [7.5 MGD] AADF, thereby matching the plant's treatment capacity. The treatment wetland was constructed as part of a lake augmentation project. The concept of a constructed wetland treatment system was developed to provide additional nutrient removal from the reclaimed water prior to discharge to Lake Marden. Lake Marden, a relict karst (sinkhole) lake, was used as a means to increase recharge to the underlying Floridan aquifer system. On the other hand, Orange County's Eastern Water Reclamation Facility (ERWF) provides wastewater treatment for the County's rapidly developing East Service Area. The EWRF is designed for a capacity of 72.2 Ml/d (19.0 MGD). The EWRF employs the Bardenpho biological nutrient removal (BNR) process. In addition to BNR, the wastewater stream treatment processes at the EWRF include filtration and disinfection. Reclaimed water from the EWRF is distributed for reuse as follows: rapid infiltration/ groundwater recharge: 9.5 Ml/d (2.5 MGD), cooling tower water: 14.2 Ml/d (3.7 MGD), and wetlands enhancement: 19.2 Ml/d (5.0 MGD). Though a different type of beneficial reuse than the NWRF treatment wetlands, the EWRF wetlands enhance the environment in

response to the increasing needs associated with urbanization. The EWRF system has been functioning well for over 20 years and is a successful component of the integrated multiple reuse program. The successful operations of both NWRF and ERWF stand for the milestones of ecological engineering practices.

11.1 INTRODUCTION

The population of the State of Florida increased from 15,982,378 in 2000 to 18,089,888 in 2006, a 13.2 percent increase (US Census Bureau, 2008), and is expected to continue to increase significantly over the next 50 years. As population increases, the need for water supply and reclaimed water management increases. This has created significant challenges in central Florida where traditional water supply sources are reaching their sustainable yield and cost-effective reclaimed water management options are limited. As a result, utilities in central Florida are implementing innovative options that meet both water supply and reclaimed water management needs. It is anticipated that public water supply demands will double in central Florida by 2020 (Vegara, 2000). Currently, water supply demands in central Florida are predominately met by groundwater from the Floridan aquifer. Since the late 1990s, the state water management districts, who have broad authority and responsibility for water supply availability as well as for flood protection, water quality protection, and ecosystem protection and restoration per Chapter 373, Florida Statutes, have expressed increasing concern that the Floridan aquifer was reaching its sustainable water supply limit (Long, 2001). This concern culminated in 2007 when the St. Johns River Water Management District (SJRWMD), South Florida Water Management District (SFWMD), and Southwest Florida Water Management District (SWFWMD), whose jurisdictional boundaries intersect in central Florida near the City of Orlando, developed the Central Florida Coordination Area (CFCA) rules which limit all groundwater withdrawals from the Floridan aquifer, which is the primary potable water supply source for the region, to projected 2013 water demands deemed by the water management districts to be reasonable and beneficial. The CFCA rules also require water use applicants to meet water demands beyond 2013 with Alternative Water Supply (AWS) sources such as reclaimed water, surface water, stormwater, or desalinated seawater. The implementation of most of these AWS sources (exclusive of the reuse or reclaimed water, which Orange County has already implemented) is significantly more costly than traditional groundwater sources.

In the years leading up to the implementation of the CFCA rules, Orange County Utilities was already facing limitations on their groundwater supply allocations from the SJRWMD and SFWMD. Simultaneously, Orange County Utilities was in need of reclaimed water management expansions at its water reclamation facilities to accommodate increased wastewater generation associated with rapid growth in the central-Florida region. River discharges of reclaimed water treated to secondary standards, a common reclaimed water management option in other regions of the country, are generally not permittable in Florida due to surface water quality issues. The water quality issues stem from low river system hydraulic gradients and high temperatures in Florida. Surface water discharges are also becoming increasingly constrained as Total Maximum Daily Loads (TMDLs) are adopted. To implement a surface water discharge, Orange County would be required to upgrade the treatment processes at water reclamation facilities to advanced treatment levels, which would significantly increase the cost of treatment. As such, reuse of reclaimed wastewater and indirect aquifer recharge options (which do not require advanced treatment) are more commonly used in the central-Florida region.

Considering the increasing need for both water supply and reclaimed water management, Orange County Utilities began to implement reclaimed water management options that would enhance the environment and recharge the groundwater system. This integrated approach to water management simultaneously addresses both the water supply and reclaimed water management issues facing Orange County as population in the region continues to increase. One of these innovative options involved implementing treatment wetlands at its Eastern Water Reclamation Facility (EWRF) and Northwest Water Reclamation Facility (NWRF). The locations of these two facilities are presented in Figure 11.1. These two water reclamation facilities treat wastewater collected in unincorporated and some incorporated areas of Orange County, which is part of the greater metropolitan Orlando area. The wetlands at these treatment plants were implemented as part of reclaimed water management facility expansions required to accommodate increased wastewater generation associated with rapid growth in the central-Florida region. While the wetland system of NWRF and was commissioned in 2005, the EWRF system has been functioning well for over 20 years and is a successful component of the integrated multiple reuse program. The successful operations of both NWRF and ERWF stand for the milestones of ecological engineering practices.

The treatment wetland at the NWRF was implemented to increase the plant's reclaimed water management capacity from 17 million liters per day (Ml/d) [4.5 million gallons per day (MGD)] annual average daily flow (AADF) to 28.5 Ml/d [7.5 MGD] AADF, thereby matching the plant's treatment capacity. The treatment wetland was constructed as part of a lake augmentation project that has been successfully operating since 2005. The concept of a constructed wetland treatment system was developed to provide additional nutrient removal from the reclaimed water prior to discharge to Lake Marden. Lake Marden, a relict karst (sinkhole) lake, was used as a means to increase recharge to the underlying Floridan aquifer system. Aquifer recharge was selected as the preferred reclaimed water management method in part because the potential reuse irrigation demand within the County's service area near the NWRF was insufficient to meet the 11.5 Ml/d [3.0 MGD] AADF reclaimed water management system capacity deficit, and in part to integrate the County's water and reclaimed water systems. Regional evaluations were indicating that projected groundwater withdrawals from the Floridan aquifer by various water supply utilities near the NWRF were going to result in unacceptable drawdowns in the Floridan aquifer and surficial aquifer groundwater table in the same area. Independent evaluations performed by Orange County indicated that one way to potentially avoid or prevent these potential drawdown constraints was to find a means of increasing recharge to the Floridan aquifer. It was determined that augmenting Lake Marden could meet the County's full 11.5 Ml/d [3.0 MGD] AADF reuse capacity deficit and would facilitate sustaining groundwater levels in the area thereby offsetting the potential effects of groundwater withdrawals.

On the other hand, Orange County's Eastern Water Reclamation Facility (ERWF) provides wastewater treatment for the County's rapidly developing East Service Area. The EWRF is designed for a capacity of 72.8 Ml/d [19.0 MGD]. The EWRF employs the Bardenpho biological nutrient removal (BNR) process. In addition to BNR, the wastewater stream treatment processes at the EWRF include filtration and disinfection. Reclaimed water from the EWRF is distributed for reuse as follows: rapid infiltration/ groundwater recharge: 9.5 Ml/d [2.5 MGD], cooling tower water: 14.2 Ml/d [3.7 MGD], and wetlands enhancement: 19.2 Ml/d [5.0 MGD]. The treatment wetlands at the EWRF were also implemented to meet the increasing reclaimed water management needs of Orange County's rapidly growing service area. However, conversely to the treatment wetlands at the NWRF, only a small percentage of the water applied to the EWRF wetlands likely recharges the underlying Floridan aquifer. Thus, the EWRF wetlands provide benefits at the same time as follows: 1) They increase valuable wetland and wildlife habitat for threatened and endangered species in the area; 2) They increases recharge to the surficial aquifer system, which potentially helps to maintain adjacent wetland systems; 3) They enhances the environment of the Econlockhatchee River, which has been designated an Outstanding Florida Water by the State of Florida; and 4) They provides baseflow during low-flow periods to the Econlockhatchee River and downstream rivers. This chapter aims to introduce both NWRF and EWRF by a comparative way and discuss the monitoring outcomes in the interfaces of engineered systems and natural environments. The following sections start with the Lake Marden and augmentation project followed by the EWRF wetlands.

11.2 NORTHWEST WATER RECLAMATION FACILITY WETLANDS

In 1997, Orange County expanded the treatment capacity of its NWRF from 13 million liters per day (Ml/d) [3.5 MGD] annual average daily flow (AADF) to 28.5 Ml/d [7.5 MGD] AADF. As part of this expansion, Orange County completed a Reuse Feasibility Study (PBS&J, 1997) that evaluated alternatives to expand the facilities reuse/reclaimed water management capacity to match its treatment capacity. The reclaimed water management system at that time consisted of 13 on-site rapid infiltration basins (RIBs) with a rated capacity of 17.0 Ml/d [4.5 MGD] AADF resulting in an 11.0 Ml/d [3.0 MGD] AADF water management capacity deficit (i.e., more treated water was being produced than could be used). The primary alternatives investigated for using this water were: 1) public access (agricultural or landscape) reuse irrigation and 2) aquifer recharge via new on-site RIBs, new off-site RIBs, and the augmentation of Lake Marden.

Surface water discharge was not investigated as a reclaimed water management option because the NWRF is located in a high recharge area consisting of land-locked lakes and karst features. In addition, surface water discharges in central Florida are discouraged by regulatory agencies due to surface water quality issues. The water quality issues stem from low river system hydraulic gradients and high temperatures in Florida. Surface water discharges are becoming increasingly constrained as Total Maximum Daily Loads (TMDLs) are adopted.

Deep well injection (injection of reclaimed water into the non-potable boulder zone beneath the Floridan aquifer system) is not a technically feasible option in central Florida. Direct potable reuse (injection of reclaimed water into the potable aquifer) is a technically feasible option, but currently cost-prohibitive when compared to other available options due to stringent treatment requirements. This option is also difficult to permit and would likely face public perception issues. For these reasons, injection-based alternatives were deemed infeasible at the time of the study.

11.2.1 Planning Alternatives

Public Access Reuse Irrigation

Public access reuse (PAR) irrigation is a widely used reclaimed water management option in central Florida that involves irrigating publically accessible landscaping or agricultural areas with public access quality reclaimed water. Public access quality reclaimed water is wastewater that has been treated to standards set by the Florida Department of Environmental Protection (FDEP) that are acceptable for safe human exposure. Simply summarized, these standards include secondary treatment with high level disinfection and filtration. Public access reuse quality water must have a Total Suspended Solids (TSS) concentration less than 5 mg/L. The NWRF was upgraded in 2000 to meet the PAR water quality standards.

From a water supply standpoint, the intent of PAR irrigation is to reduce groundwater withdrawals by using reclaimed water for irrigation in lieu of potable water. From the standpoint of operating a wastewater facility, it provides a reclaimed water management option, but one with challenges related to the effect of dry and wet-weather events. During dry weather, irrigation demands increase and reclaimed water production decreases. This results in the need to implement a supplemental source to meet demands during periods of supply deficit. During wet weather, irrigation demands decrease and reclaimed water production increases. As such PAR irrigation does not provide the required reclaimed water management during wet periods and would require the implementation of a reclaimed water management option that is not dependent on consumer demands and would accommodate wet-weather conditions (i.e., RIBs, treatment wetlands, or lake augmentation).

Orange County, Florida first considered agricultural irrigation due to the abundance of nurseries, greenhouses and ferneries in the portion of its service area where NWRF is located. The NWRF is in a relatively isolated area of unincorporated Orange County and is constrained by service areas of adjacent reclaimed water utilities; therefore, the potential for residential, commercial, and other types of irrigation are limited. The 1997 Reuse Feasibility Study (Post Buckley Schuh & Jernigan, 1997) identified approximately 3.0 Ml/d [0.8 MGD] AADF of potential agricultural PAR in the vicinity of the NWRF. Though reducing withdrawals in the area by up to 3.0 Ml/d [0.8 MGD] AADF, it was ultimately determined that the irrigation needs in the area were not sufficient to meet Orange County's entire reclaimed water management capacity deficit.

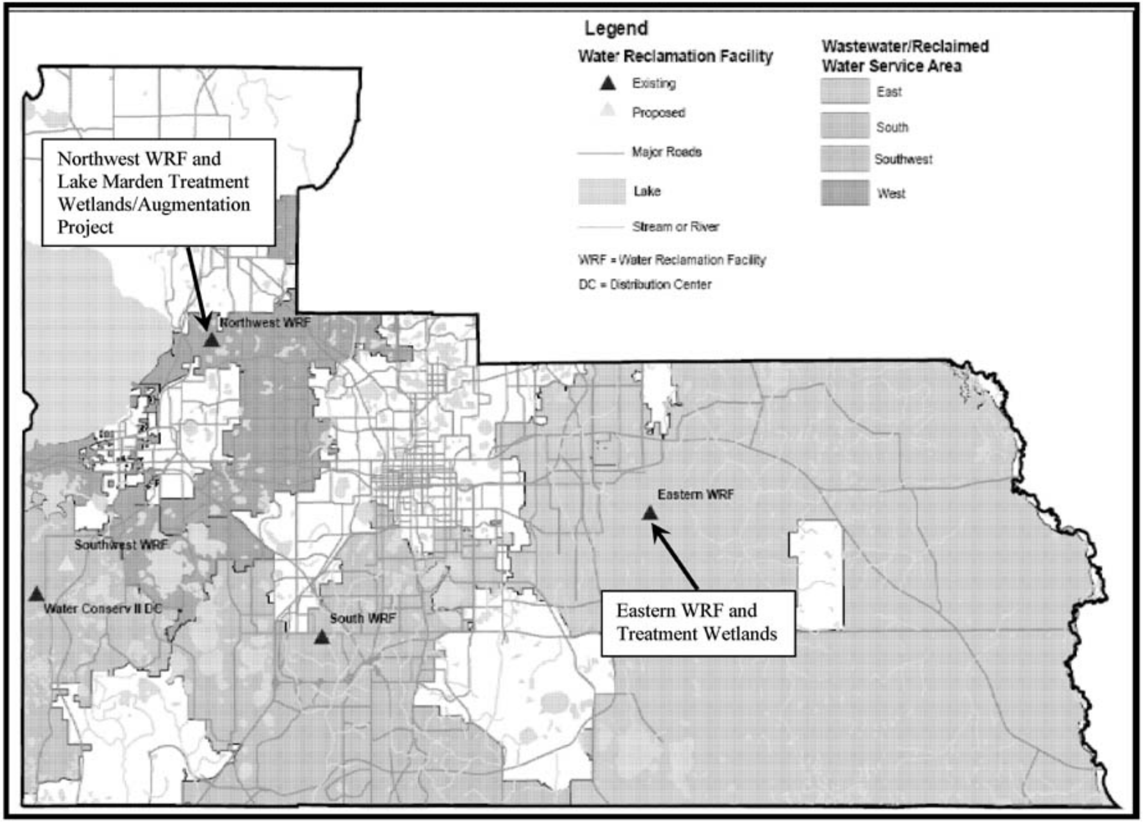

Figure 11.1: Location Map (PB Americas, Inc., 2008)

Aquifer Recharge

Simultaneous to solving its reclaimed water management capacity deficit at NWRF, Orange County was also keeping an eye on regional water resource issues. In western Orange County, the intermediate confining unit (also known as the Hawthorn Formation) has high leakance characteristics that connect the surficial aquifer water table to the underlying Floridan aquifer potentiometric surface levels. As such, hydraulic changes to either system directly affect the other system. Indications were that projected withdrawals from the Floridan aquifer by various water utilities near the NWRF were going to result in unacceptable drawdowns in the Floridan aquifer and the surficial aquifer groundwater table in the same area. These issues became particularly evident when Orange County began to renew its Consumptive Use Permit from the St. Johns River Water Management District for the County's Western Regional Water Supply Facility located 2 miles east of the NWRF.

Orange County's independent evaluations of the water resources of east-central Florida indicated that one way to potentially avoid or mitigate these potential drawdown constraints was to find a means of increasing recharge to the Floridan aquifer. For this reason, and because of the low potential for public access reuse irrigation in the area, the focus for reclaimed water management capacity at NWRF became aquifer recharge.

Aquifer recharge can be implemented directly or indirectly. Direct recharge is the intentional application of water directly into a target aquifer system (i.e., direct potable reuse). As previously discussed, direct recharge options were deemed infeasible at the time of this study. Indirect recharge is the artificial recharge or intentional application of water in a manner that will indirectly recharge a groundwater aquifer. This involves allowing the recharged water to pass through a different medium, such as an overlying aquifer system (i.e., the surficial aquifer system), lake or wetland, or other natural or anthropogenic feature, before it recharges the target aquifer system (i.e., the Floridan aquifer system).

The Floridan aquifer system is a highly prolific aquifer that is used as the primary potable water supply for central Florida. In many areas, Floridan aquifer groundwater meets primary and secondary drinking water standards using only disinfection for treatment and pH adjustment for distribution system water quality considerations such as pipeline corrosion. Some areas of the Floridan aquifer also require aeration for the removal of hydrogen sulfide that is naturally occurring at varying levels in the Floridan aquifer. Brackish regions of the Floridan aquifer, where they are closer to the coast or in areas which connate water exists, are not as commonly used as a reliable source for drinking water supply.

Reclaimed water used to recharge the underlying Floridan aquifer is also of high quality that meets most primary and secondary drinking water standards. Reclaimed water applied via indirect recharge methods is diluted as is passes through the surficial aquifer or lakes and wetlands. Wetlands provide additional treatment via nutrient removal by the vegetation; however, it is not commonly believed that the surficial aquifer or lakes provide a significant level of treatment. However, the Floridan aquifer is generally biologically active in the central Florida region, which is evident by the biota that are sometimes found dwelling in the Floridan aquifer, and provides additional treatment as water travels through the aquifer system. As such, the already high quality reclaimed water applied via recharge is treated further as it passes through the aquifer system.

Indirect recharge can be implemented for a variety reasons including, but not limited to: 1) reduce, stop, or reverse water table and/or potentiometric drawdowns or springflow reductions associated with groundwater withdrawals; 2) create seawater intrusion barriers; 3) store water underground for future water supply; 4) mitigate flood impact in wet-weather; 5) eliminate surface water discharges of reclaimed water; and 6) provide wet-weather management of excess reclaimed water for a reuse system. A variety of methods have been developed to artificially recharge groundwater systems. As part of the 1997 Reuse Feasibility study, RIBs and lake augmentation were deemed the most feasible options at that time.

Additional RIBs

One aquifer recharge method examined was the implementation of additional RIBs. RIBs are a widely used land-application based indirect recharge option. They are commonly used as a reclaimed water/wet-weather management and/or water supply recharge option throughout inland counties of central Florida. RIBs involve discharging water into basins that allow water to percolate/infiltrate into the ground, through surficial aquifer system sediments. Ideally, the infiltrated water then flows to the underlying Floridan aquifer system through discrete conduits or as diffuse leakance through the intermediate confining unit (also known as the Hawthorn Formation). Figure 11.2 presents an idealized geologic cross section showing the implementation of indirect recharge via RIBs.

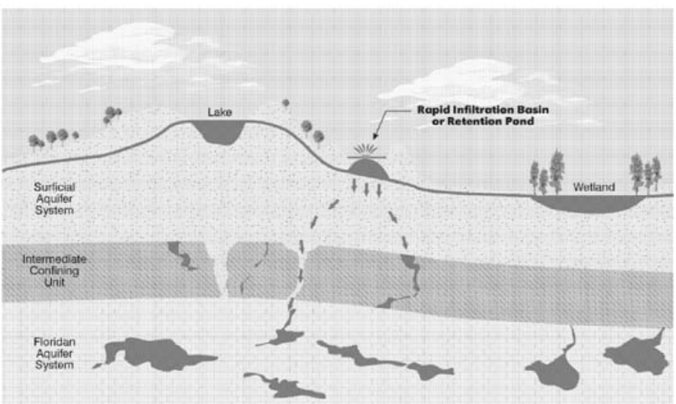

Figure 11.2: Indirect Recharge via RIBs (PB Water, 2004)

RIB capacity in the vicinity of the NWRF tends to be constrained because a relatively thin layer (10 to 15 feet on average) of high permeability sands limits the conveyance capacity of the surficial aquifer (e.g., local hydrogeologic characteristics). The NWRF property is approximately 700 acres, much of which was used for the plant, the original 13 RIBs, and a transfer station located within the site. Of the original 700 acres, only between 100 and 200 acres of land was available for development. It was determined that providing the needed 11.5 Ml/d [3.0 MGD] AADF annual average daily flow of additional reclaimed water management capacity through RIBs on the original site was not feasible. Nearby lands were also investigated for expansion of the RIB system at the NWRF. An insufficient amount of land to accommodate the 11.5 Ml/d [3.0 MGD] AADF capacity deficit was identified near the existing site.

Augmenting Lake Marden

Another aquifer recharge method examined was to augment the on-site Lake Marden directly as a surface water discharge. Lake augmentation involves directly discharging reclaimed water into a lake to augment or supplement the lake. Lake augmentation differs from surface water discharge because the lakes selected for augmentation are land-locked with no stream or river discharge. Non-discharging lakes in central Florida typically have high leakance characteristics (i.e., are karst features) and provide recharge to the underlying Floridan aquifer system through the Intermediate Confining Unit. This tends to limit the application of augmentation to regions of central Florida with isolated land-locked lakes that have suitable hydrogeologic characteristics (e.g., high leakage). Figure 11.3 presents an idealized geologic cross section showing the implementation of indirect recharge via lake augmentation.

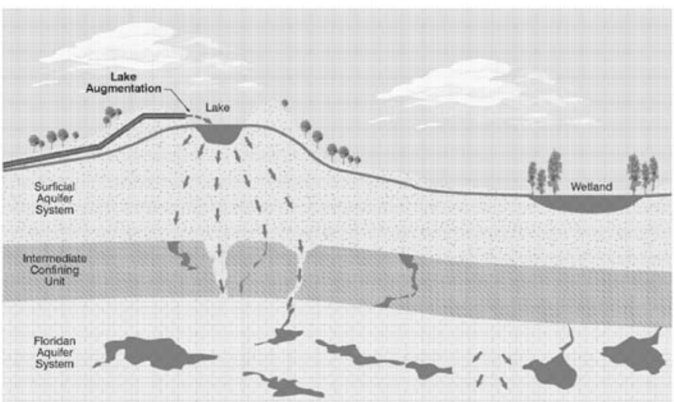

Figure 11.3: Indirect Recharge via Lake Augmentation (PB Water, 2004)

Lake Marden, the target lake for this study, is wholly owned by Orange County, so it was exempt from federal surface water discharge requirements. This made the permitability of the project considerably more feasible than a surface water discharge at a lake not wholly owned by Orange County. To investigate the potential for this option to meet the reclaimed water management deficit, groundwater flow modeling was performed. The results of the groundwater flow modeling, which were the basis for the design and permitting for the project, indicated that the entire 11.5 Ml/d [3.0 MGD] AADF of needed reclaimed water capacity could be recharged through lake augmentation.

11.2.2 Treatment Process

At the time of the project, the NWRF was a 28.5 Ml/d [7.5 MGD] AADF permitted capacity two-stage carrousel extended aeration domestic wastewater treatment facility consisting of influent screening, grit removal, odor control, anoxic tanks (denitrification), aeration, secondary clarification, chemical feed facilities, up-flow sand filtration units, and high-level disinfection (e.g., secondary treatment with high-level disinfection). Figure 11.4 presents a process flow diagram for the NWRF. The NWRF was already producing PAR quality reclaimed water at the time of the reclaimed water management system expansion.

Despite being exempt for surface water discharge rules, the FDEP was initially concerned that augmenting a lake that directly recharged the underlying Floridan aquifer, which was the primary source of potable water for the area, would result in adverse impacts to surface and groundwater quality in the area. The FDEP was particularly concerned about nitrogen levels in the aquifer. Ambient nitrogen concentrations in the Floridan aquifer system are generally low (less than 1.0 mg/L). Increasing nitrogen levels at the springs over the past several decades in central Florida, particularly in the Wekiva Basin where the NWRF is located, have been receiving increasing concern from regulatory agencies. The regulatory agencies have indicated that potential sources of nitrogen in the aquifer could be from agricultural and residential fertilizers and reclaimed water from water reclamation facilities and septic tanks. As a result of this concern, the FDEP required the implementation of a constructed wetland treatment system to provide additional nutrient (i.e., nitrogen and phosphorus) removal prior to the discharge of reclaimed water to Lake Marden.

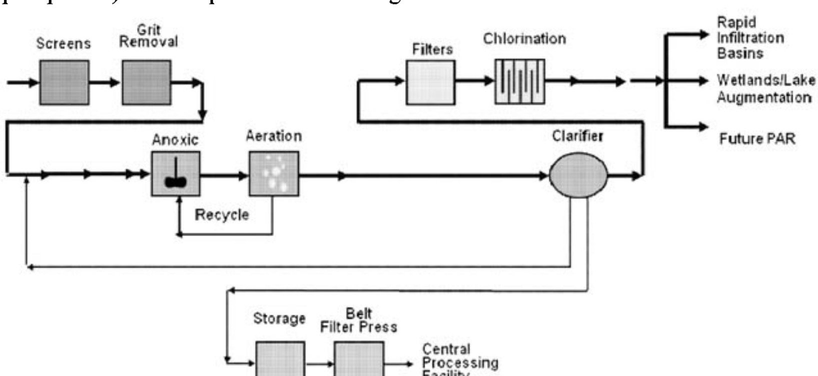

Figure 11.4: NWRF Process Flow Diagram (Orange County Utilities, 2006)

The FDEP required that water discharged to Lake Marden have a concentration of Total Nitrogen (as N) less than 3 mg/L. This is similar to the water quality limitation for Total Nitrogen required by Florida Administrative Code (F.A.C.) for discharge from created or treatment wetlands. This is the reason created or constructed wetlands were chosen as a feasible engineering option to couple with the Lake Marden augmentation project to achieve the target water quality limits.

If Orange County had not implemented a treatment wetland as part of a lake augmentation project, treatment process upgrades may have been necessary at the NWRF. In addition, if an applicant for a lake augmentation project does not own the lake, the project would likely be subject to a Water Quality Based Effluent Limitations (WQBEL) permitting determination. A WQBEL determination could also result in the need for treatment process upgrades at the WRF providing reclaimed water at the project. These upgrades would likely be higher than the upgrades required for a project for which the applicant owns the lake but does not implement a treatment wetland. Treatment plant upgrades would have increased the overall cost of the Lake Marden Project.

11.2.3 Wetlands

As mentioned, the FDEP required that the water discharged to Lake Marden have a concentration of Nitrogen (as N) less than 3 mg/L. The existing WRF was designed to produce reclaimed water with a Nitrate (as N) concentration less than 12 mg/L (which is the regulation for rapid rate land applications such as RIBs and slow rate land applications such as irrigation). It was determined that the required nutrient removal could be achieved with a created wetland on an approximately 100 acres of remaining undeveloped land on the plant site.

Created wetlands, also known as treatment wetlands, artificial wetlands, or constructed wetlands, are a relatively common reclaimed water management option implemented in Florida. Created wetlands involve directly discharging reclaimed water into man-made or natural wetlands. The FDEP reports that there are 33 active treatment wetlands utilized for reclaimed water management at water reclamation facilities in Florida. A treatment wetland can be man-made, natural, or a combination of man-made and natural.

When used for reclaimed water management, the FDEP requires that reclaimed water discharged to a treatment wetland, which is defined as a wetland within the landward extent of waters of the state, must meet secondary treatment by nitrification with a total ammonia (as N) concentration not exceeding 2.0 mg/L as a monthly average. Discharge of reclaimed water to a receiving wetland, which is defined as a wetland within the landward extent of waters of the state, not being used for treatment, must meet the following additional water quality requirements:

- Carbonaceous Biochemical Oxygen Demand $CBOD_5$: 5 mg/L
- TSS: 5 mg/L
- Total Nitrogen (as N): 3 mg/L
- Total Phosphorus (as P): 1 mg/L

Discharge of reclaimed water to purely created (man-made) wetlands does not have the above requirements; however, receiving, treatment, and created wetlands must all meet the following water quality requirements for water discharged from the wetlands if the wetlands are contiguous to or discharge to another water body:

- Total Nitrogen (as N): 3 mg/L (on an annual average basis), of which no more than 0.02 mg/L may be un-ionized ammonia
- Total Phosphorus (as P): 0.2 mg/L (on an annual average basis)

The above applies unless Water Quality Based Effluent Limitations (WQBELs) have been established for the receiving water body. Additional requirements regarding the water quality of discharges from wetlands used for reclaimed water management are specified in the FDEP rules.

Lake Marden is wholly owned by Orange County and does not discharge to another water body, and is therefore not a water of the state. It also does not have any established WQBELs. A created (man-made) wetland was being proposed for the treatment. For these reasons, only the regulatory water quality limitations for water discharged from a wetland were required for the Lake Marden Project. The FDEP requires that treatment wetlands provide a minimum detention time of 14 days (in addition to the water quality limitations). The Lake Marden wetland was designed to treat 11.5 Ml/d [3.0 MGD] AADF of reclaimed water with a detention time of approximately 22 days. This was achieved with an approximately 70-acre wetted perimeter and an average wetland depth of approximately 2 feet (the depth was variable based on existing topography). The planted vegetation in the Lake Marden wetland varied depending on the depth of water in a given area. Figures 11.5 and 11.6 present the planted wetlands.

In addition to the water-quality based design of the wetland, the water-resource aspects of the wetland were also an important consideration. The approximately 100 acres of available land were located at the top of a sandy hill. Due to this, there was initially a concern that lateral seepage and associated unacceptable impacts to adjacent land uses might be an issue. Seepage can be conceptualized using a water balance of the wetland as follows:

$$\Delta S = I - O$$

where: I = Inflows $[L^3/T]$
O = Outflows $[L^3/T]$
ΔS = Change in storage $[L^3/T]$

This can be expanded to include components of the water balance as follows:

$$\Delta S = P + WI - E - WO - S$$

where: P = Precipitation $[L^3/T]$
WI = Wetland Inflow $[L^3/T]$

E = Evaporation [L^3/T]
WO = Wetland Outflow [L^3/T]
S = Seepage from the wetland [L^3/T]

Once placed into operation, only seepage through the bottom of the wetland is unknown in the above equation (ΔS, P, WI, WO are monitored and E is estimated from well-documented publications). As such, actual seepage from the wetland can be empirically calculated by substituting the known values into the above equation.

As part of design, S and WO were both unknown. The target project capacity was 11.5 Ml/d [3.0 MGD] AADF; however, it was unknown if the location could accept higher or lower application rates than the target capacity. As a result, WI was also unknown. The number of unknowns required the application of groundwater flow modeling. The predicted capacity of the wetland and the seepage from the wetland were estimated using a detailed local-scale numerical groundwater flow model nested with a regional numerical groundwater flow model. It was initially estimated that the selected on-site location could accept the target capacity. However, it was estimated that seepage would be over 33 percent of the annual average capacity of the system and may result in unacceptable impacts to adjacent land uses.

Figure 11.5: NWRF Wetland Vegetation - littoral zone (Deleon and Rivera, 2006)

Figure 11.6: NWRF Wetland Vegetation – central area (Deleon and Rivera, 2006)

To alleviate this issue, a bentonite slurry wall was planned within the berm of the wetlands, through the top layer of sandy surficial aquifer system soils, and keyed into a lower permeability silty-sand and clayey-sand surficial aquifer soil layer. This reduced the model predicted seepage to 10 percent of the annual average permitted capacity of the system and eliminated all potential offsite impacts. Minimizing seepage also reduces the potential for the wetland to temporarily dry out, which could compromise the health of the wetland vegetation and the functionality of the wetland.

The FDEP required the wetlands to be designed to capture a 25-year/96-hour design storm, which is approximately 13 inches in the NWRF area. The Lake Marden wetlands were also designed to accommodate a peak day factor for reclaimed water application of 2 times the annual average permitted capacity. This was done to accommodate increased reclaimed water supply during wet-weather. These criteria were incorporated into the design of the wetland berms. During peak flow conditions and design storms, the wetlands detention time will be reduced. Continued compliance with discharge regulations during peak flow conditions and storms was an additional consideration in the design of the wetlands.

The wetlands were designed with a series of vertical discharge pipes connected to a manifold system at the influent end of the wetland. This allowed for an even distribution of water across the width of the wetland. Even distribution is preferred to minimize "dead" or ineffective wetland zones and short circuiting at the influent end of the wetland. The manifold system also provides some aeration as water is discharged into the wetland. The effluent structure was a large drop inlet with boards to control the wetland stage. The effluent structure discharged to a gravity pipeline that conveyed reclaimed water to nearby Lake Marden.

The wetland was segregated into six sections or cells. The six cells were designed in three pairs from the influent end to the effluent end (along the primary flow path) of the wetland system. The cells were segregated into three sections along the flow path to accommodate the natural change in topography at the wetland site. The bottom elevations of the three pairs of cells were stair-stepped in two-foot increments. The three sections were separated by berms and connected with drop inlet structures with boards (to control internal water levels) and culverts to convey water between cells. The three sections were split into two "trains" to allow the two halves of the wetland to be operated separately or in tandem. This allows for redundancy in case cells need to be taken off line for maintenance or other purposes. The parallel cells were hydraulically connected with slide gates. This allowed water levels to be equally maintained in the two sets of parallel cells when operated in tandem. Figure 11.7 presents the layout of the Lake Marden wetlands and demonstrates the segregation of the wetland into cells. This picture was taken prior to the planting of the wetland vegetation and the system being placed into full operation. The first pair of cells in the wetland system was planted with a variety of deep marsh species with a dominance of bulrush. The second and third pairs of cells were planted with mixed marsh vegetation designed to enhance the wildlife habitat.

11.2.4 Management Challenges

The Lake Marden Augmentation Project was the first aquifer recharge project of its kind in Florida, so obtaining permits for the proposed facilities was one of the most challenging aspects of implementing the project. As previously mentioned, this wetland system associated with the project was ultimately located at the top of a large on-site hill primarily comprised of sandy soils. This location was the only portion of the existing property large enough to accommodate the required treatment wetland. In addition to concerns from the state government, nearby homeowners and an adjacent borrow pit/landfill owner protested Orange County's wastewater operational and environmental resource permits, alleging that the project would increase water levels in the area and potentially inundate their properties. Of particular concern was the location of the proposed wetlands. Local residents were concerned that locating the wetland at the top of a hill comprised of sand would result in reclaimed water seeping out of the wetlands and onto their property.

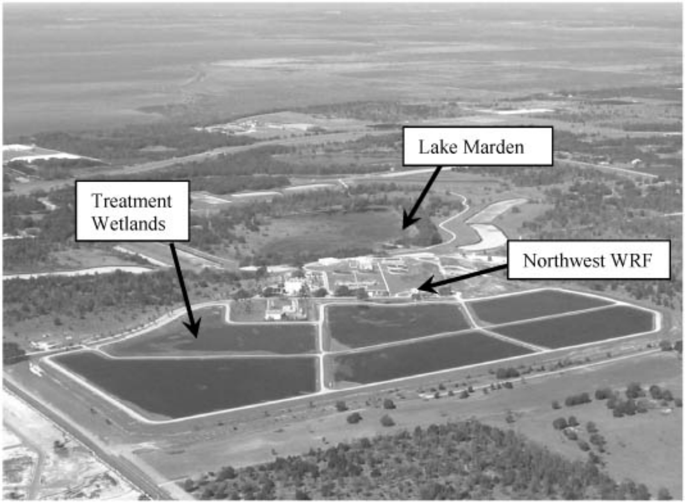

Figure 11.7: NWRF Treatment Wetland Layout (Mather, 2005)

This was addressed through the implementation of the slurry wall previously discussed. Numerical groundwater flow modeling was used to develop the preliminary design of a slurry wall within the outer berm of the treatment wetlands to reduce the potential for off-site seepage of reclaimed water. The modeling was used as the basis of design and to provide reasonable assurance to the permitting agency that the project would not result in adverse hydrologic impacts to the area. In fact, it was demonstrated that the project resulted in beneficial effects to the hydrology of the area by offsetting potential drawdowns in the surficial and Floridan aquifers that may be occurring as a result of groundwater withdrawals. The beneficial effect of the Lake Marden Project was also included as part of the reasonable assurance provided to the St. Johns River Water Management District in the County's Consumptive Use Permit to withdrawal groundwater from the Floridan aquifer. Ultimately, in 2003, all permit protests were resolved and the Lake Marden Augmentation Project received a permit from the FDEP and construction began.

11.2.5 Construction

During construction, there were a number of challenges encountered prior to placing the project into operation. The first challenge was the installation of the slurry wall within the exterior berm. As previously mentioned, the slurry was designed to be constructed through the highly permeable sands and keyed into a clayer, siltier sand layer beneath the overlaying sands. However, the depth of the overlaying sands was highly variable (between 10 and 25 feet) throughout the perimeter of the wetland. The square footage of the required slurry wall was ultimately greater than initially anticipated (or bid by the contractor). This resulted in additional cost and construction delays.

The second challenge related to the planting of the wetland vegetation. The contractor was required to control water levels within the wetland (prior to being placed into operation) at suitable levels to allow for proper planting. However, the interior berms separating the stair-stepped wetland cells were permeable and allowed water to seep into the lower elevation cells. This increased the difficulty of planting and resulted in the loss of vegetation. As a result, it took significantly longer than anticipated to achieve wetland vegetation grow-in. The following types of wetland vegetation were planted in the NWRF treatment wetlands: 1) Scirpus californicus – Giant Bulrush, 2) Eleocharis Interstincta – Knotted Spikerush, 3) Potamogeton sp. – Pondweed, 4) Nymphaea Odorata – Fragrant Water-lily, 5) Sagittaria Latifolia – Duck Potato, 6) Pontederia Cordata – Pickerelweed, 7) Thalia Geniculata – Fire Flag, 8) Juncus Effusus – Soft Rush, 9) Spartina Bakeri – Sand Cordgrass, and 10) Panicium Hemitomon – Maidencane.

11.2.6 Start-up and Operation

Construction of the Lake Marden Project was completed in February 2005 and it was placed into operation in March 2005. The first year of operation was used to allow grow-in of the wetland vegetation. During this period, reclaimed water was conveyed to the wetlands, but water was restricted from discharging from the wetlands. In March 2006, water was initially discharged from the wetland system to Lake Marden. The quantity of water discharged to Lake Marden was incrementally increased over many weeks. This allowed the response of the system to be monitored and operations to be adjusted based on system performance and hydrologic conditions. This small incremental increase in loading rate also allowed gopher tortoises along the shores of Lake Marden to relocate as the lake stage slowly increased. This approach to the gopher tortoise relocation was required by the Florida Fish and Wildlife Conservation Commission permit.

Much of 2006 was used to correct operational issues that were uncovered during the initial operating period (i.e., system controls). Since being placed into full operation, the County has been conducting a load test to determine the operational capacity of the system. This load test is not complete and data from the test are not available at this time. However, the County has been successfully operating the Lake Marden treatment wetland and lake augmentation system at approximately 11.5 Ml/d [3.0 MGD] AADF since placed into full operation. Though data from the load test of the wetland and lake augmentation system are not yet available, data presenting the quality of reclaimed water being conveyed to the wetland are presented in Figures 11.8 and 11.9.

Today, many wildlife including but not limited to sandhill crane, multiple types of heron, multiple types of egrets, ibis, redtail hawk, multiple types of turtles, and alligators thrive in the wetlands. Based on the success of the Lake Marden Project, Orange County is currently implementing a second treatment wetland/lake augmentation project on a newly acquired parcel (with lake) adjacent to the NWRF.

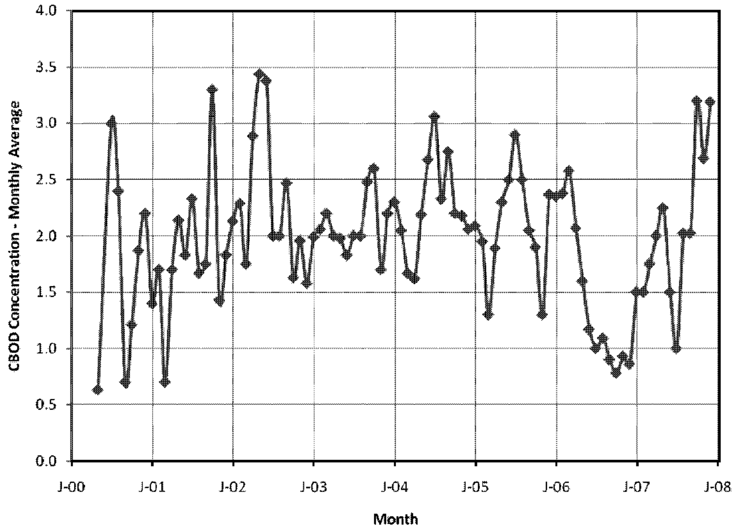

Figure 11.8.: NWRF Monthly Average Reclaimed Water $CBOD_5$ Concentration (mg/L)

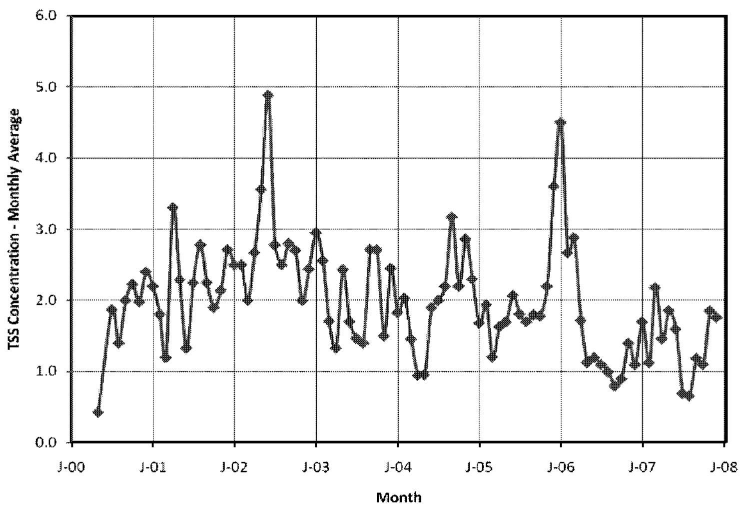

Figure 11.9: NWRF Monthly Average Reclaimed Water TSS Concentration (mg/L)

11.2.7 Benefits of NWRF Treatment Wetland/Lake Augmentation System

A majority of the reclaimed water applied to the NWRF Lake Marden treatment wetlands/lake augmentation project recharges the underlying Floridan aquifer. The only potential system loss is due to evapotranspiration; however, in western Orange County that is generally balanced by precipitation on a long-term annual average basis. Water from the Lake Marden system that recharges the Floridan aquifer offsets groundwater level surficial and Floridan aquifer groundwater level drawdowns potentially associated with groundwater withdrawals. In particular, offsets in the surficial aquifer water table serve to maintain or enhance lake and wetland levels in the area, which is demonstrated by pre- and post-project photographs of Lake Marden (Figures 11.10 and 11.11). In addition, recharge of the Floridan aquifer helps to maintain and enhance spring discharges in the area. This beneficial recharge was recognized by the St. Johns River Water Management District in Orange County's Consumptive Use Permit renewal to withdraw groundwater from the Floridan aquifer and provided the necessary reasonable assurance required for an increase in groundwater allocation.

11.3 EASTERN WATER RECLAMATION FACILITY WETLANDS

Orange County's Eastern Water Reclamation Facility (ERWF) provides wastewater treatment for the County's rapidly developing East Service Area. The EWRF initially utilized RIBs for reclaimed water management. The on-site RIBs have a permitted capacity of 9.5 Ml/d [2.5 MGD] AADF. In 1987, the EWRF began to provide a large industrial customer, the Orlando Utilities Commission's (OUC's) Stanton Energy Center (SEC), reclaimed water for its power plant cooling processes. Though SEC provided a large volume relatively consistent customer that increased the reclaimed water management capacity at the EWRF, SEC's demands vary depending on power usage fluctuations and climatic conditions (SEC uses a storage reservoir to receive the reclaimed water which also collects direct rainfall on the reservoir). Due to this, the EWRF needed an additional reclaimed water management option to accommodate days when reclaimed water generation exceeded SEC demands and the capacity of the on-site RIBs.

In 1988, Orange County placed into operation a combination of created and natural wetlands to provide a portion of its needed reclaimed water management at the EWRF. At the time, Orange County's PAR system (landscape irrigation) was not yet developed in their East Service Area and projections for reuse demand would not meet the County's short-term reclaimed water management needs. In addition, landscape irrigation is highly variable depending on climatic conditions and would not serve to meet the County's wet-weather management needs at the EWRF. RIB capacity in eastern Orange County tends to be constrained by hydrogeologic conditions. As such, insufficient land was available to provide the needed RIB recharge capacity. Due to the potential limitations of RIBs and landscape irrigation, treatment wetlands were selected as the preferred reclaimed water management option.

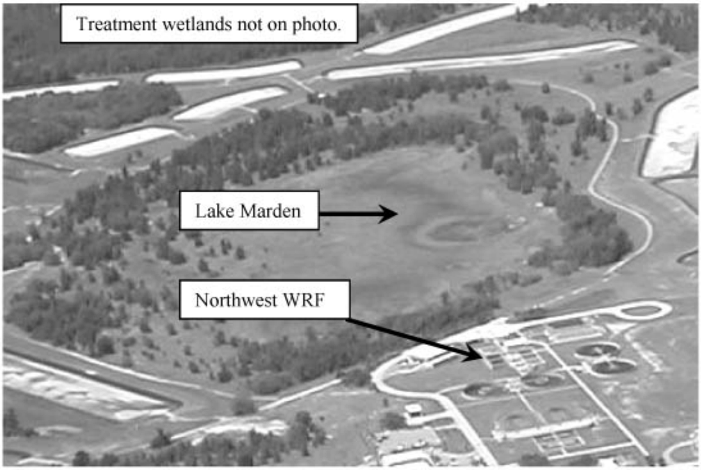

Figure 11.10: Lake Marden 2001: Pre-Project (Mather, 2001)

Figure 11.11: Lake Marden 2006: Post-Project (Rivera, 2006)

11.3.1 Wastewater Treatment and Reuse System

The EWRF originally consisted of a 9.5 Ml/d [2.5 MGD] AADF Modified Ludzack-Ettinger (MLE) process. The facilities included a pre-treatment structure for screening and grit removal, two Carrousel™ - type anoxic-aerobic tanks, secondary clarifiers, and chlorination. The current treatment system at the EWRF primarily consists of a five-stage Bardenpho® advanced biological nutrient removal process with final clarification, sand filtration, and chlorination. Figure 11.12 presents a process flow diagram for the EWRF. It has a total permitted capacity of 71.9 Ml/d [19 MGD] AADF and produces water that exceeds advanced treatment standards. The plant is currently being expanded to 79.5 Ml/d [21 MGD] AADF.

As previously discussed, the reclaimed water management system at the time the treatment wetlands were brought on line consisted of RIBs and power plant cooling reuse. Since the wetlands were brought on line, the power plant cooling reuse system was expanded (and is in the process of being expanded again) and the County has implemented a rapidly expanding PAR system consisting primarily of residential, commercial and golf course irrigation. The power plant and irrigation demands combined with the management capacity of the RIBs and wetlands provide the County with a robust reclaimed water management system for the EWRF.

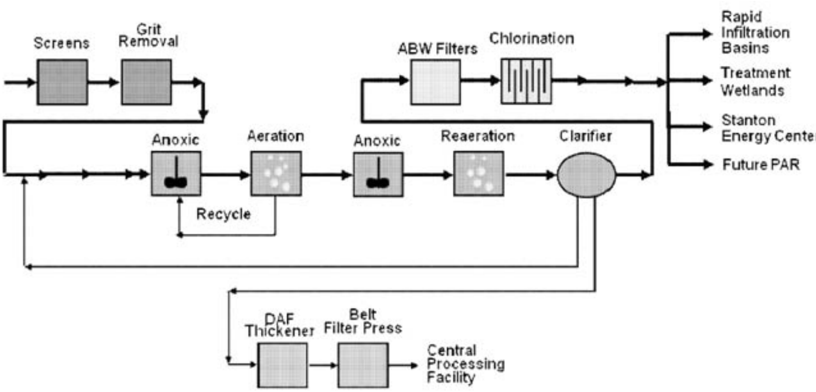

Figure 11.12: EWRF Process Flow Diagram

11.3.2 Wetlands System

The EWRF treatment wetlands consist of a unique combination of overland flow, created wetland, and natural wetland areas (FWRJ, 2002). Reclaimed water is first conveyed through an overland flow area that feeds a 35-acre created wetland that serves to evenly distribute flow through the system. The initial created wetland then discharges to a natural cypress-dominated swamp that serves as a treatment wetland. Once the water flows through the treatment wetland, it is recollected and redistributed through a second created wetland that is approximately 45 acres. The second created wetland also serves to evenly distribute flow into the next segment of the wetland system. From the second created wetland, water flows into a natural hardwood swamp and then to another natural cypress swamp. The final cypress swamps are the exit wetlands. Flow from the exit wetlands is discharged to a tributary of the Econlockhatchee River. Figure 11.13 presents a layout of the EWRF wetland system.

Figure 11.14 presents a picture of the piping manifold that discharges reclaimed water into the initial distribution created wetland of the EWRF wetland system.

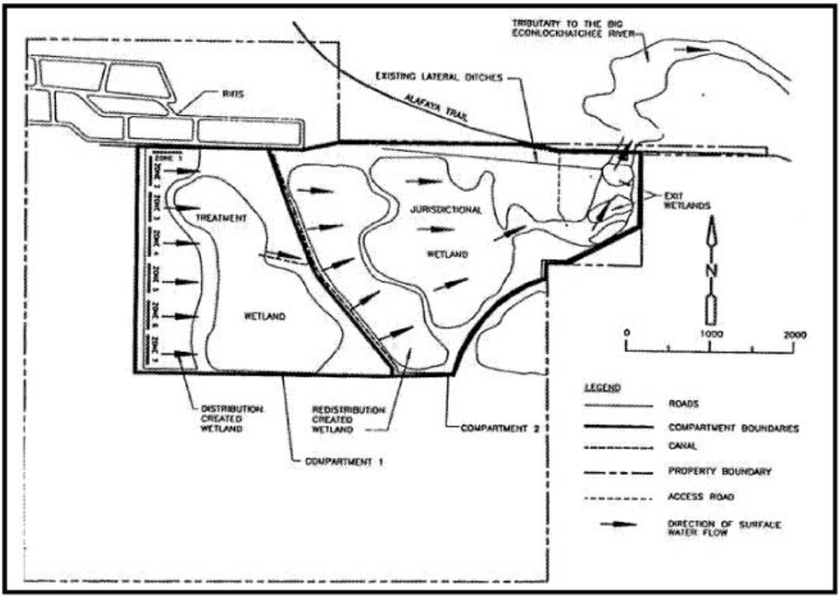

Figure 11.13: Orange County EWRF Treatment Wetland Layout (FWRJ, 2002)

Figure 11.14: Orange County EWRF Treatment Wetland Distribution Created Wetland and Influent Piping Manifold

The natural wetlands integrated into the EWRF wetlands did not require any vegetation planting. The natural treatment wetland is primarily dominated by cypress and loblolly bay. The natural jurisdictional wetland is a mixed forested system consisting primarily of cypress, sweet magnolia, and red maple. The "exit" wetlands are cypress domes primarily dominated by cypress trees. The constructed wetlands were originally planted with 15 tree and herbaceous plant species. However, the location of the constructed wetlands (interdispursed amongst other natural wetlands) proved to be conducive to natural wetland species colonization. After many years of operation, approximately 185 diverse tree and herbaceous wetland plant species adapted to the original design and naturally colonized in the constructed wetland (Schwartz, 2008). The distribution constructed wetland was originally dominated by herbaceous vegetation, but is now predominately a forested system consisting of trees and shrubs with a healthy groundcover. The redistribution constructed wetlands was originally an herbaceous wetlands, but now also exhibits significant shrubs and healthy groundcover. In addition to healthy vegetation, the EWRF wetlands are home to a variety of wildlife including wading birds such as herons and egrets, hawks, turtles and alligators.

The EWRF treatment wetlands were initially permitted as an experimental exemption to the treatment wetland rules. However, in 1998 after almost a decade of operation, it was determined that the wetlands were a highly effective reclaimed water management option and the experimental exemption was removed and the system was permitted based on the Wetlands Application Rule, Chapter 62-611, F.A.C. The wetlands were permitted with the following influent (e.g., water entering the wetlands) water quality limitations (on an annual average basis):

- $CBOD_5$: 5 mg/L
- TSS: 5 mg/L
- TN (as N): 3 mg/L
- TP (as P): 1 mg/L

The wetlands were also permitted with the following discharge or effluent (e.g., water exiting the wetlands water quality limitations (on an annual average basis):

- TN (as N): 2.0 mg/L
- TP (as P): 0.2 mg/L

The permitted capacity of the system is 23.5 Ml/d [6.2 MGD] AADF.

11.3.3 System Operation and Monitoring

There are seven zones of discharge at the influent end of the wetland. Discharge to the wetlands is rotated to provide for alternating wet and dry periods; however, the wetlands are not permitted to dry out for extended periods. This is required to maintain the viability and functionality of the wetland vegetation. It is estimated that between 0.2 and 1.9 Ml/d [0.5 MGD] AADF is required to maintain the hydration of the wetlands. The only regular maintenance performed for the wetland is periodically cleaning the redistribution channel (FWRJ, 2002).

There are seven water quality sampling stations in the wetland that are monitored quarterly for dissolved oxygen (DO), pH, $CBOD_5$, TSS, total phosphorus (TP), total nitrogen (TN), sulfate (SO_4), Fecal Coliform, and Chlorophyll a. Figures 11.15 through 11.18 present the quality of reclaimed water being conveyed to the wetlands for selected parameters. Biota is also monitored annually in the wetland system. At the final discharge station downstream from the exit wetlands, TP and TN are monitored weekly. Finally, the Econlockhatchee River is monitored quarterly for CBOD5, TSS, DO, TN, TP, and Chlorophyll *a* at five sampling stations.

11.3.4 System Performance

During its first ten to fifteen years of operation, the wetlands successfully propagated native plant communities. Key water quality results from monitoring during this period of operation are as follows (FWRJ, 2002):

- An insignificant increase in CBOD5 was detected, but concentrations were below 3 mg/L;
- No change in TSS was detected;
- The reclaimed water did not cause a microbiological contamination of surface waters;
- Ambient iron levels in the wetland decreased due to dilution from the reclaimed water;
- Metal concentrations in the wetland did not exceed Florida Class III quality criteria;
- Nitrate plus nitrite concentrations reduced from the influent to the effluent ends of the wetland;
- Ambient TP concentrations in the wetland were diluted by the application of reclaimed water;
- pH increased within the wetland; however, this may also be the effect of photosynthesis; and
- DO concentrations in the wetland were maintained during the operational period.

In summary, the wetlands have shown no decrease in surface outflow water quality and all discharge water-quality performance goals were met or exceeded during the initial ten to fifteen year operational period. The wetlands have been performing satisfactorily since that time as well. In addition, water-quality data indicate that discharge from the wetlands is having no adverse affect on the water quality of the Econlockhatchee River.

A water balance based on the flow and water quality results performed after the initial ten to fifteen year operational period indicates that reclaimed water was the largest hydrologic inflow to the wetland system and surface discharge was the largest outflow component from the system. Other components such as lateral groundwater flow and soil storage were relatively minor components of the annual average water budget (FWRJ, 2002).

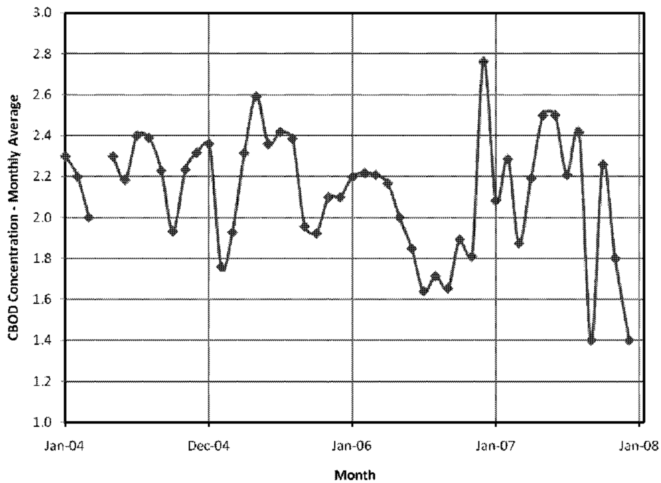

Figure 11.15: EWRF Monthly Average Reclaimed Water $CBOD_5$ Concentration (mg/L)

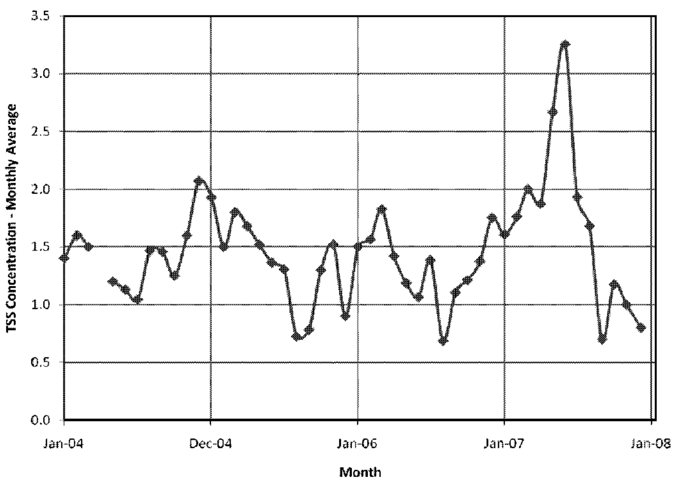

Figure 11.16: EWRF Monthly Average Reclaimed Water TSS Concentration (mg/L)

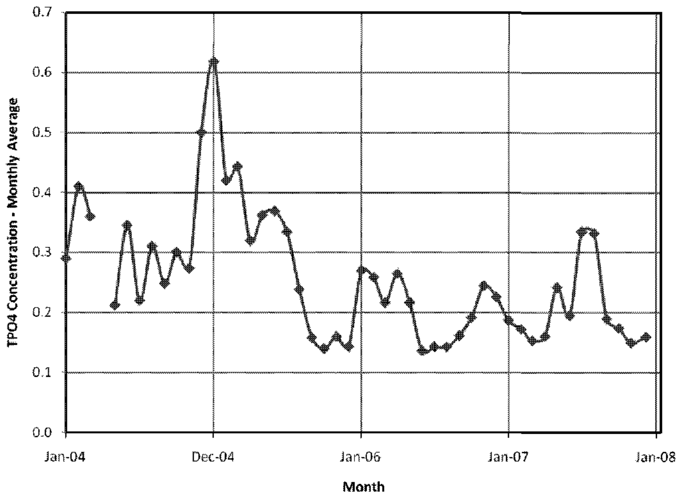

Figure 11.17: EWRF Monthly Average Reclaimed Water TPO_4 Concentration (mg/L)

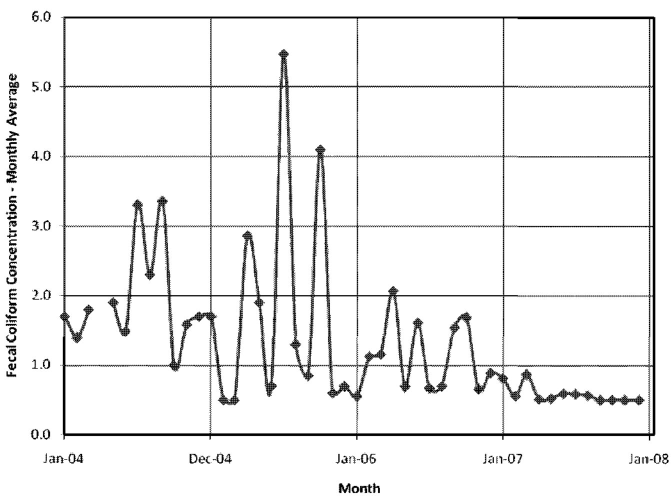

Figure 11.18: EWRF Monthly Average Reclaimed Water Fecal Coliform Concentration (mg/L)

11.3.5 Benefits of EWRF Treatment Wetlands

The general hydrogeologic characteristics in the vicinity of the EWRF treatment wetlands and NWRF treatment wetlands/lake augmentation project are similar. The hydrogeology in these areas can be generally characterized as follows: 1) Surficial Aquifer: Generally consisting of sands, silts, and clays varying in thickness and consistency; 2) Intermediate Confining Unit (Hawthorn Formation): A semi-confining unit generally consisting of clayey sediments of varying thickness and consistency; and 3) Floridan Aquifer: A highly transmissive aquifer generally consisting of limestones and dolomites. The Floridan aquifer in Orange County is generally considered to consist of multiple stratigraphic layers known as the Upper Floridan aquifer, the Middle Semi-confining Unit, and the Lower Floridan aquifer.

Though the general hydrogeology is similar, the specific characteristics of these aquifers are very different at the NWRF than at the EWRF. Near the NWRF, located in western Orange County, the surficial aquifer has a highly permeably sandy soil layer near land surface and a relatively deep depth to water table. In addition, the Intermediate Confining Unit near the NWRF is relatively thin and highly leaky with many karst features. The surface water drainage basins in the area are land-locked with no discharge. Water applied to land application systems such as RIBs, treatment wetlands, or lake augmentation projects are generally routed to the karst, land-locked features characteristic of the area. Once conveyed to these karst features, water passes through the surficial aquifer and intermediate semi-confining unit into the underlying Floridan aquifer.

Near the EWRF, located in eastern Orange County, the surficial aquifer tends to be comprised of less permeable silty-sands and clayey sands. The depth to water tends to be shallow (up to a few feet below land surface) and the intermediate confining unit is relatively thick and of low permeability. The surface water drainage basins in the area discharge to tributaries and rivers and some wetlands in the area are perched. Due to these conditions, a low percentage of water applied to the surficial aquifer system recharges the Floridan aquifer.

Due to the differences discussed above, the NWRF and EWRF wetlands function differently. As previously discussed, a majority of the water applied to the NWRF Lake Marden treatment wetlands/lake augmentation project recharges the underlying Floridan aquifer thereby offsetting groundwater level drawdowns potentially associated with groundwater withdrawals.

Conversely, only a small percentage of the water applied to the EWRF wetlands likely recharges the underlying Floridan aquifer. The EWRF wetlands provide other benefits as follows:

- Increases valuable wetland and wildlife habitat for threatened and endangered species in the area;
- Increases recharge to the surficial aquifer system, which potentially helps to maintain adjacent wetland systems;

- Enhances the environment of the Econlockhatchee River, which has been designated an Outstanding Florida Water by the State of Florida; and
- Provides baseflow during low-flow periods to the Econlockhatchee River and downstream rivers.

Though a different type of beneficial reuse than the NWRF treatment wetlands, the EWRF wetlands enhance the environment in response to the increasing needs associated with urbanization. The EWRF system has been functioning well for over 20 years and is a successful component of the integrated multiple reuse program.

11.4 GROUNDWATER MONITORING

The NWRF treatment wetland/Lake Marden augmentation project has an extensive monitoring network that includes wetland influent and effluent flow and quality, rainfall, wetland water level and quality, groundwater level and quality, and Lake Marden stage. As previously discussed, OCU is currently performing a several year-long operational load test of the Lake Marden system. Data and analyses results from the loading test are not yet available for publication. However, Table 11.1 presents several years of water quality data for the reclaimed water conveyed to the wetlands.

Table 11.1: NWRF Average Reclaimed Water Quality Concentrations (mg/L) for Select Permit Parameters for Water Conveyed to the Wetlands

Parameter	Permit Limit – Annual Average (mg/L)	2000	2001	2002	2003	2004	2005
$CBOD_5$	20	2.0	1.8	2.3	2.0	2.3	2.1
TSS	5	1.8	2.2	2.7	1.9	2.0	1.9
TN	12	3.8	2.3	4.3	4.3	3.2	4.7
TP	N/A*	0.9	1.0	1.2	1.3	1.0	0.6

* The NWRF does not have a concentration limitation on TP.

Based on the water quality data presented in Table 11.1, TN concentrations in the reclaimed water conveyed to the wetlands is typically between 2 and 5 mg/L. Preliminary data indicates the TN concentration in the water discharged from the treatment wetlands to Lake Marden is regularly less than 1.0 mg/L. The permit limitation on the concentration of TN in the water discharged from the wetland is 3.0 mg/L.

As previously discussed, the permitted water quality concentration limitations of water conveyed to the EWRF wetlands correspond to advanced treatment requirements (5-5-3-1). As presented in Table 2, the quality of the reclaimed water being conveyed to the wetlands meets the permit requirements.

In addition to the water quality permit limitations for water conveyed to the wetlands presented in Table 11.2, the EWRF wetlands also have water quality limitations for TN and TP for the water discharged or exiting from the wetlands. The permit limitations for these parameters are 2.2 mg/L and 0.2 mg/L, respectively. The concentration of TN and TP in the water discharged from the EWRF wetlands is typically below 1.0 mg/L and 0.1 mg/L, respectively, and well within the limits specified in the permit.

Table 11.2: EWRF Average Reclaimed Water Quality Concentrations (mg/L) for Select Permit Parameters for Water Conveyed to the Wetlands

Parameter	Permit Limit – Annual Average (mg/L)	1995	1996	1997	1998	1999	2000	2001
$CBOD_5$	5	1.0	2.0	1.0	1.6	1.5	1.1	1.4
TSS	5	1.0	2.0	2.0	1.5	1.2	1.0	1.4
TN	3	1.7	2.0	1.5	2.4	2.4	2.7	2.46
TP	1	0.07	0.09	0.27	0.25	0.29	0.38	0.20

** Incomplete data set

11.5 CONCLUSIONS

Central Florida is projected to experience significant population increases over the next 20 to 50 years. As population increases, the need for water supply and reclaimed water management increases. Public water supply demands are anticipated to double in central Florida by 2020 (Vergara, 2000). In turn, the portion of public water supplies that are returned to a water reclamation facility for treatment and management will also significantly increase. Facing limited traditional water supplies and cost-effective reclaimed water management options, central-Florida utilities are implementing innovative alternatives that address both issues in an integrated manner. One such innovative option is the implementation of treatment wetlands. Orange County Utilities, Florida has successfully implemented treatment wetlands at two of its water reclamation facilities.

The treatment wetlands at the NWRF are coupled with a lake augmentation project. Reclaimed water discharged from the treatment wetlands is routed directly to Lake Marden. This project increases valuable wetland and wildlife habitat, enhances the ecosystem of Lake Marden, and recharges the underlying Floridan aquifer via vertical diffuse leakage from Lake Marden through the Hawthorn semi-confining unit. The resulting aquifer recharge offsets drawdowns associated with groundwater withdrawals from the Floridan aquifer in the region.

The treatment wetlands at the EWRF increase wetland and wildlife habitat, increase recharge to the surficial aquifer system which potentially hydrates adjacent wetland systems, enhances the environment and increases the baseflow of the Econlockhatchee River. Enhancing the baseflow of the Econlockhatchee River could offset future surface water public supply withdrawals in rivers downstream of the Econlockhatchee River currently being implemented in central Florida.

Both the NWRF and EWRF wetlands increase the reclaimed water management capacity at these two facilities. The wetlands also diversify the reclaimed water management options available to these facilities by creating alternatives that can accommodate both a base-loaded or moderate wet-weather peak flow loading condition. The wetlands at the EWRF have been successfully operated for approximately 20 years and serve as an example of a proven technology in the hydrologic and hydrogeologic conditions characteristic of eastern Orange County. The NWRF treatment wetlands have been operating for several years. Additional flow, groundwater level, and water quality data are needed to fully evaluate this coupled treatment wetland/lake augmentation system. OCU is currently implementing a loading test of this system that includes collecting the required data. Though the loading test is not complete, preliminary results indicate the project is operating successfully. Based on the preliminary results of the Lake Marden treatment wetland/lake augmentation system, OCU is currently designing a second treatment wetland/lake augmentation project at Lake Cora Lee. This project will also serve as an expansion of the NWRF reclaimed water management system and will further offset drawdowns associated with groundwater withdrawals in the region.

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CHAPTER 12

Low Impact Development Practices: Designing to Infiltrate in Urban Environments

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ABSTRACT: Low Impact Development (LID) is predicated on achieving a water balance for runoff, infiltration, and evapotranspiration. To meet infiltration needs associated with LID, many structural practices are used including: bioretention, infiltration trenches/wells, infiltrating wetlands, level spreader, vegetated filter strip systems, permeable pavement, swales, and water harvesting systems. Each of these practices can potentially infiltrate substantial amounts of runoff. Moreover, designers can alter these practice designs to increase infiltration volumes. Some design and construction features discussed in this chapter include the incorporation of sumps, or storage wells, in the bottom of LID infiltration practices and innovative means of excavation such that underlying soil saturated hydraulic conductivity is better preserved. An important concern for a system that relies on infiltration is whether the rate of infiltration is retained with time. A study at Villanova University indicated that while there is substantial infiltration rate fluctuation seasonally, there was no statistical evidence of annual infiltration rate decreasing. An examination of several types of permeable pavement systems in North Carolina showed that the type of permeable cover was not a factor in substantial and statistical runoff reduction. This allows LID designers to specify permeable pavement types based upon cost and

aesthetics rather than hydrologic performance. Lastly, in addition to providing runoff volume reduction many Low Impact Development techniques, such as standard grass swales examined in Maryland, provide peak runoff mitigation, albeit mostly for small to medium-sized events. All the case studies presented herein substantially reduced runoff, thereby increasing amounts of infiltration. Both of which are main goals of Low Impact Development in urban environments.

12.1 INTRODUCTION

This chapter highlights water quantity and water quality goals for Low Impact Development (LID) principles and practices, with a special focus on infiltration. Structural engineering practices associated with LID are examined and special design features that may be used to encourage infiltration are mentioned. The demonstrated ability of each of these Best Management Practices (BMPs) to infiltrate is shown. Designers and stormwater managers will be able to use this chapter as a guide for LID BMP selection and understand the infiltration potential for each practice in urban environments.

12.1.1 Low Impact Development

Low Impact Development, or LID, is fundamentally predicated on achieving a hydrologic balance (Davis 2005). A direct result of urbanization is the elimination or decreased permeability of permeable surfaces, which alters the balance of runoff, infiltration and evapotranspiration. Construction impacts have been shown to reduce infiltration rates of in situ soils by a factor of 10 (Gregory et al. 2006, Pitt et al. 2008). Conventional pavement eliminates infiltration altogether. Conventional measures, such as wet ponds, used to reduce peak flows generated by the decrease in permeability only marginally improve infiltration and do not come close to replicating an undisturbed green field's infiltration and evapotranspiration. The goal of LID is to treat runoff with practices – both structural and non-structural – such that the annual hydrologic balance (volumes of runoff, infiltration, and evapotranspiration) of the post-developed condition is similar to that of a pre-developed or target condition.

A necessary consequence of achieving this hydrologic goal is pollutant load reduction to the storm drain network. Less water leaving a development will typically mean lower pollutant loads. Moreover, many of the structural practices incorporated in LID employ pollutant removal mechanisms, such as filtration, chemical sorption, and biological processes, which reduce pollutant concentrations. It is often the pollutant load reduction aspect of these practices that causes their selection by stormwater designers and managers. The purpose of this chapter, however, is to highlight how many LID techniques increase infiltration from developments that would otherwise by runoff dominated.

The list of practices associated with LID is substantial and includes non-structural techniques. A partial list is found in Table 12.1. This chapter focuses on structural practices associated with Low Impact Development that have an infiltration component, including: bioretention, infiltration wells/ trenches, infiltrating wetlands, level spreader – vegetated filter strips, permeable pavement, swales, and water harvesting – irrigation systems. Each practice's appearance and function will be described as will special design features that can be used to enhance infiltration. A review of some studies that indicate infiltration volumes associated with each practice will also be highlighted.

Table 12.1: Example Low Impact Development Practices

Tree Preservation	Rooftop Detention	Riparian Buffers	Pollution Prevention
Strategic Grading	Smaller Culverts	Infiltration Trenches & Swales	Maximizing Street flow
Site Finger Printing	Permeable Surfaces	Infiltration Wetlands	Landscape Storage
Flat & Wide Swales	Water Harvesting	Green Roofs	Soil Aeration
Amended Soils	Rain Barrels/ Cisterns	Reforestation	Shoulder Filter Strips
Bioretention/ Rain Gardens	Level Spreader – Vegetated Filter Strips	Curb and Gutter Elimination	Impervious surface reduction

12.1.2 Regulating Infiltration

Maintaining predevelopment ground water recharge functions is one of the design objectives of the LID approach to stormwater management. As of 2008 only a small number of states and local jurisdictions had requirements and design criteria for maintaining ground water recharge. Maintaining ground water recharge is an emerging issue in stormwater management. It is important to note that an infiltration BMP can be used to recharge or restore both the base flow and groundwater components of the hydrologic cycle.

The design criteria for the use of infiltration BMPs to maintain base flow and groundwater recharge are not currently well defined. The design elements for which design criteria need to be standardized include: 1) the volume of annual runoff required to maintain pre-development recharge, 2) how the BMP should be designed to optimize recharge, and 3) what limitations, if any, pollutant inputs and the native subsoils place on the ability of the infiltration BMP to meet recharge requirements. The components of the pollutant input to the BMP determines whether infiltration is an acceptable practice. Current EPA Policy encourages the use of certain LID techniques for infiltration in residential areas (USEPA Memorandum Jun 11, 2008). In areas where groundwater recharge is critical, novel designs can be incorporated that would have the subsurface base of the infiltrating BMP wider than the surface. Watershed studies in Pennsylvania have related recharge to preconstruction recharge and water consumption, and show that capturing as little as 1.25 to 2.5 cm (0.5 to 1.0 in.) of runoff from each storm exceeds this goal (Delaware County Planning Commission, 2005)..

12.2 INFILTRATION BMPs ASSOCIATED WITH LOW IMPACT DEVELOPMENT

12.2.1 Bioretention

Since its initial development and trial applications over a decade ago, the bioretention system, also referred to as "raingardens" and "bioinfiltration," has rapidly become one of the most versatile and widely used stormwater BMPs throughout the US and many parts of the world. It has recently become identified as a preferred site practice for green building design and Leadership in Energy and Environmental Design (LEED) certification. Bioretention cells often double as required landscape features.

General features of a bioretention system include 0.7-1 m of a sand/soil/organic media for treating infiltrating stormwater runoff, a surface mulch layer, various forms of vegetation, orientation to allow 15-30 cm of runoff pooling and associated appurtenances for the inlet, outlet and overflow. Figure 12.1 shows a diagram of a typical bioretention system. Several bioretention installations in the Mid-Atlantic region are presented in Figure 12.2.

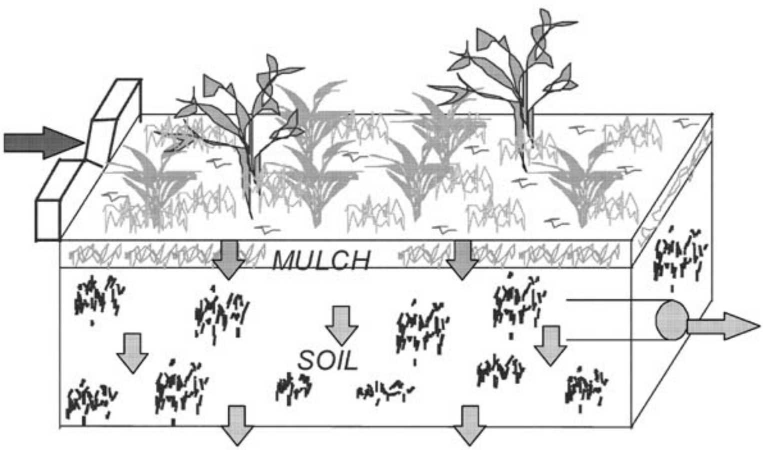


Figure 12.1: Diagram of Bioretention Facility

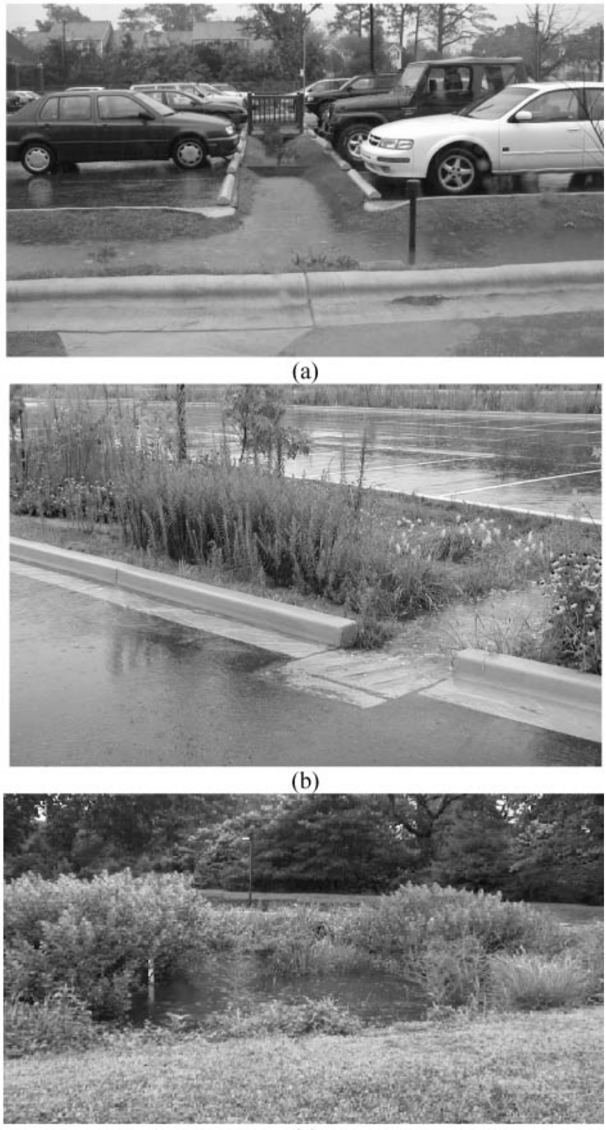


Figure 12.2: Photos of Bioretention in a) North Carolina, b) Maryland, c) Pennsylvania

Incorporating both filtration and infiltration, initial research into bioretention has shown that these facilities substantially reduce runoff volumes and peak flows. Bioretention incorporates biological treatment practices, such as vegetative uptake and nitrification and denitrification. When a precipitation event occurs, the bowl of the bioretention cell fills, sometimes only partially, and water infiltrates the designed soil media, which filters the passing stormwater. Depending upon antecedent moisture conditions, a fraction of the filtered water will adhere to soil particles and eventually flow into the underlying drainage system. Bioretention outflow, that is water that enters the storm drain network, is the summation of overflow associated with the bowl filling and under drain effluent. The remaining volumes of water that temporarily are stored in the media either infiltrate or evapotranspire.

Several researchers have examined the benefits of creating a sump in the bottom of the bioretention cell (Dietz and Classen, 2006; Kim et al., 2003; Hunt et al., 2006). Sumps, or internal water storage zones (Davis et al., 2009), are designed to temporarily store water so that it can later infiltrate. Creating sumps is accomplished by either elevating the underdrains or installing an upturned elbow, as seen in Figure 12.3. Design guidance on IWS sizing is in the process of being established, but is not yet available.

Figure 12.3: An elevated underdrain (as seen during construction) creates an IWS

Another option to increase infiltration from bioretention that is beginning to be used by designers is preparing the underlying soil by scarifying the soil rather than the typical "smearing" excavation technique employed by most contractors (Figure 12.4). Brown and Hunt (2009) examined these two excavation techniques and demonstrated that the scarifying excavation technique (1) compacted soils less and (2) had higher measured hydraulic conductivities. This study is further described later in this chapter.

Figure 12.4: (a) The Rake, or scarifying, method of excavation versus (b) the typical smearing excavation technique. Technique (a) has been shown to significantly improve infiltration

Bioretention hydrology has been well tested across the US and beyond. Volumes of runoff and infiltration/evapotranspiration have been determined. Only a few studies, however, have been able to measure and estimate the separation of infiltration and evapotranspiration. Infiltration and evapotranspiration (ET) processes are important in the functioning of bioretention systems. Infiltration and evapotranspiration together can account for the fate of 50% to 90% of inflow, depending on in situ soil type, media depth and type, and drainage configuration (Heasom et al., 2006; Hunt et al., 2006). To date, however, only limited information has been published related to how these processes function within a bioretention system, or how they can be optimized for hydrologic benefit and pollutant removal. A field study in Louisburg, NC (Li et al. 2009) showed ET accounting for the fate of 10-15% of all inflow water on an annual basis. Li et al. also showed that proportionally larger bioretention cells, relative to their contributing drainage area, infiltrated more water than their more undersized counterparts.

Infiltration has not been found to decrease with time. Infiltration rates from a bioinfiltration device were monitored for four years in Villanova, PA (Emerson and Traver, 2008). The rate of infiltration did not degrade, supporting the idea that bioretention cells can maintain infiltration rates for at least several years. Emerson and Traver (2008) did find that seasonality existed in infiltration rate, with reduced infiltration occurring in colder months.

12.2.2 Infiltration Wells and Trenches

Infiltration trenches and wells are a group of related practices similar to bioretention that convey water in shallow cells, as shown in Figure 12.5. They are specifically intended to infiltrate water into shallow ground water. These systems are typically filled with larger stone or other very porous media, and are deeper then bioretention cells. Similar to bioretention, these systems rely on infiltration practices; however there is no expectation of evaporation as a removal mechanism.

(a) (b)

Figure 12.5: Villanova University Infiltration Sites (a) Infiltration Trench under construction. (b) Infiltration well or pit circa 1900's

Infiltration trenches and wells collect the water from impervious areas usually directly piped from impervious areas. Pretreatment is strongly recommended for these systems to prevent sediments from clogging the geotechnical layers that separate the porous media from the surrounding soils. In contrast to bioretention, these systems are deeper, and have a larger pore space (or volume) to temporarily store runoff while waiting for the water to infiltrate. Construction practices to avoid compacting the soil are the same as bioretention, and washing of the fill media is required to remove fine materials. To increase infiltration capacity requires either an increase in surface area or volume. Many manufacturers now offer crate type units with larger void space to replace the stone media. Other ideas may include an irregular outside pattern to increase surface areas. Similar to bioretention, special construction methods that compact or smear soils less can increase infiltration volumes.

In contrast to bioretention, all the captured volume infiltrates as the water is not directly exposed to the surface environment, and there are no plants supporting transpiration. Infiltration from these sites is more three dimensional than from bioretention practices. Infiltration occurs through both the sides and the bottom of the media depending on design characteristics, and is related to depth (Figure 12.6). The infiltration trench in Figure 12.5a has shown changes in performance over time. It has been theorized that the bottom is clogged due to fine entrapment against the geomembrane on the bottom. It should be noted that the inflow to the infiltration BMP is poorly treated, and the drainage area to BMP footprint is approximately 24 times larger than is generally recommended to rapidly age the site. Recent results do indicate that the infiltration through the sides is still operational, and if sized correctly would be considered a successful design (Emerson and Traver, 2008). The change in infiltration rates due to temperature was also observed at this site, and is discussed in more detail later in this chapter.

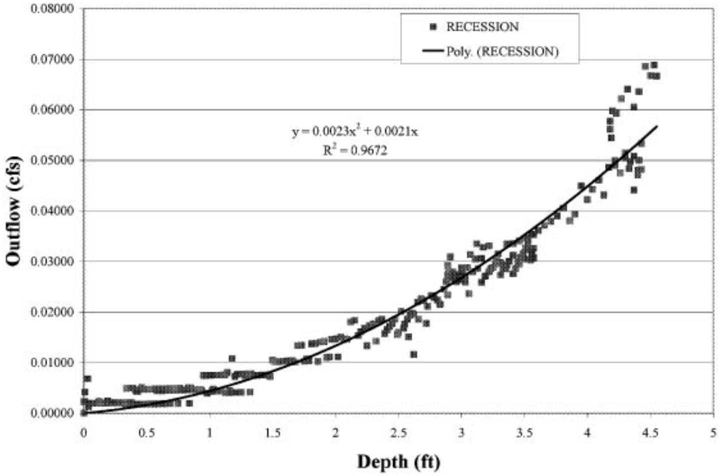


Figure 12.6: Infiltration Trench Outflow versus Depth Curve (Dean 2005)

In contrast, the historic seepage pit shown in Figure 5b is estimated to have been in operation from the 1890s to the 1970s when it was disconnected. The pit collected runoff from four story slate roofs, and was found to be still operational when tested (Welker et al. 2006). Clearly the role of suspended solids is the key to long term performance of these practices.

12.2.3 Infiltrating Wetlands

Infiltrating wetlands are typically small, "pocket" wetlands located in permeable soils with relatively high water tables. The presence of seasonally high water tables usually prohibits the use of other LID techniques such as bioretention and infiltration trenches. Deeper water zones of these wetlands are designed to retain water during droughty periods, but much of the wetland is subject to occasionally drying out due to infiltration and ET losses. These wetlands would typically be located in flat, sandy soils that adjoin the Mid-Atlantic and Southeastern coasts of the United States. The facilities are wetter than bioretention cells, because they do not rely on a drainage infrastructure or special fill media. Examples of infiltrating wetlands are shown in Figure 12.7.

(a) (b)

Figure 12.7: Two infiltrating wetlands located in North Carolina (a) River Bend and (b) Wilmington. In (b) the inundated deep pool in the foreground is in contrast to the remaining portion of the wetland which is not saturated at the surface

Infiltrating wetlands fill like "conventional" wetlands during precipitation events. However, unlike conventional wetlands where nearly all detained runoff is discharged through an outlet structure, much of the runoff captured in the infiltrating wetland exits by infiltration. This occurs because water is ponded in the wetland above the water table elevation. The rates of infiltration are based upon the driving head of water stored in the wetland and the wetland's water table elevation at the time and immediately following the storm event. Assuming an in situ soil infiltration rate of 0.1 in/hr, an internal NCSU study showed that as much as 20% of all incoming water can infiltrate on an annual basis (Jones, 2008).

As the perimeter of infiltrating wetlands increases, there is more potential to infiltrate water into the surrounding soil. Infiltration is mostly governed by the perimeter, as limited infiltration occurs vertically, due to the presence of a high water table. Because designers have some control over the shape of the wetland, maximizing the perimeter: surface area ratio is expected to increase infiltration volumes. Designers can also restrict flow rates through the detention structure forcing captured runoff to have more residence time in the wetland, thus giving the captured runoff more time to infiltrate. There have been limited studies on infiltrating wetlands. Lenhart and Hunt (2009) examined an infiltrating wetland in River Bend, NC, and measured a runoff volume reduction of 54% for 24 hydrologic events. In fact, the wetland usually had available storage for each storm, as illustrated in Figure 12.8.

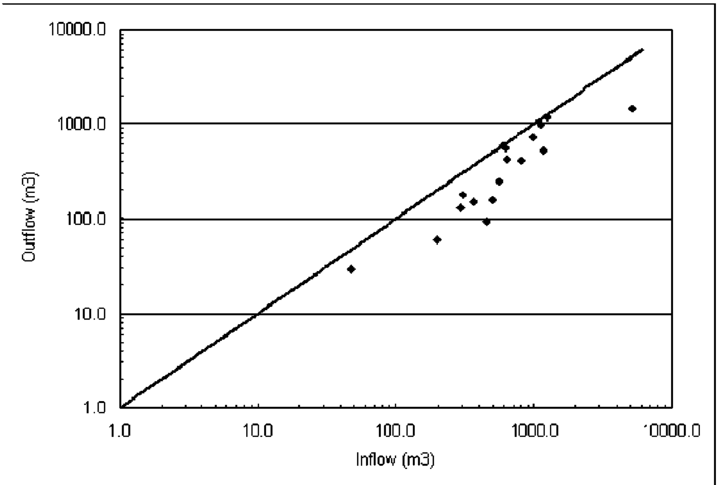


Figure 12.8: Volume reduction provided by an infiltrating wetland in River Bend, NC. (Source: Lenhart and Hunt 2009)

12.2.4 Level Spreader / Vegetated Filter Strips

Level spreaders are hardened structures (often concrete footer walls) that spread runoff evenly over the same grade before flow enters a downslope vegetated filter strip, often a riparian buffer. This allows for infiltration and some eventual evapotranspiration. Moreover, if the vegetated filter strip is comprised of turf grass, there is a perceived high acceptance by landowners (Figure 12.9).

(a) (b)

Figure 12.9: Level spreader – grassed vegetated filter strip systems in (a) Charlotte, NC, and (b) Apex, NC

Level spreader and graded grass filter strips may often be the most appropriate BMP in locations with a seasonally high water table near the surface, which is a common occurrence in some of the flatter topographic regions along the southeast and mid-Atlantic coasts. Their use, conversely, will be more limited in steep slope applications typically found in hillier, mountainous topography. Level spreader systems consist of three parts: the forebay, the channel, and the vegetated filter strip (Figure 12.10).

Designed filter strips have been shown to infiltrate substantial quantities of runoff when used in conjunction with level spreaders (Line and Hunt 2009, Hunt et al. 2009). Substantial infiltration is due to vegetated filter strips being graded by design, ensuring their "levelness" and downward slope. This allows water to remain in sheet flow for much longer periods than those associated with a naturally-occurring topography. An extensive study by Hathaway and Hunt (2008) showed that level spreaders upslope of riparian buffers did not provide diffuse flow in any of the 24 level spreader/riparian buffer systems examined. In many cases the topography of the riparian buffer forced water to re-concentrate, effectively bypassing most of the riparian buffer's hydrologic benefits. Grassed filter strips that are evenly graded perpendicularly from the level spreader tend to keep flow from concentrating, thus allowing for increased infiltration.

Filter strips can be amended with compost or other soil amendments to make these systems more permeable (Hunt et al., 2009). The ratio of the filter strip area to contributing watershed area can also be increased. Because the use of level spreader – vegetated filter strip systems has not been extensive in urban areas yet, little design guidance is available as to the how flatter, amended, and relatively larger vegetated filter strips function.

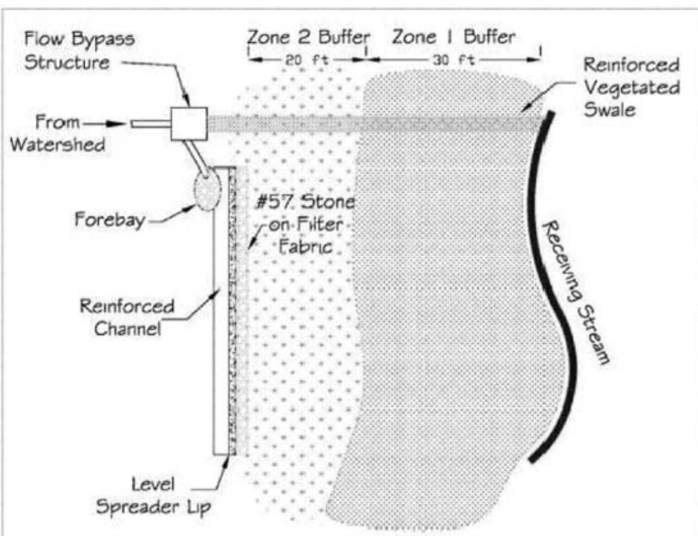


Figure 12.10: Schematic of a level spreader system upslope of a riparian buffer

Studies by Yu et al. (1993), Line and Hunt (2009) and Hunt et al. (2009) show that level spreader – vegetated filter strip systems can be effective stormwater management practices. Yu et al. (1993) examined a level spreader-vegetated filter strip system for 8 events in Virginia. While Yu and his colleagues did not report runoff volume reduction, they did show substantial reduction in effluent concentrations of TSS. Line and Hunt's (2008) examination of a LS-VFS (5.2% slope) receiving highway runoff from a 0.35 ha (0.86 ac), 49% impervious watershed showed a runoff volume reduction of 49%. The watershed to VFS ratio at that site was 28:1. In comparison, Hunt et al. (2009) examined a level spreader – vegetated filter strip in a residential neighborhood in Charlotte, NC, that received stormwater from a 0.87 ha (2.15 ac) watershed with a watershed to VFS ratio of 9:1. This system, for 23 monitored events, had an 85% volume reduction. The difference in performance was probably the result of (1) the lower watershed to VFS area ratio, (2) the substantially flatter slope (5.2% to 1.25%), and (3) the soil amended VFS. The marginal benefits of a relatively larger VFS size, amended soil and flatter slope appear to exist, but still need to be quantified.

12.2.5 Permeable Pavement

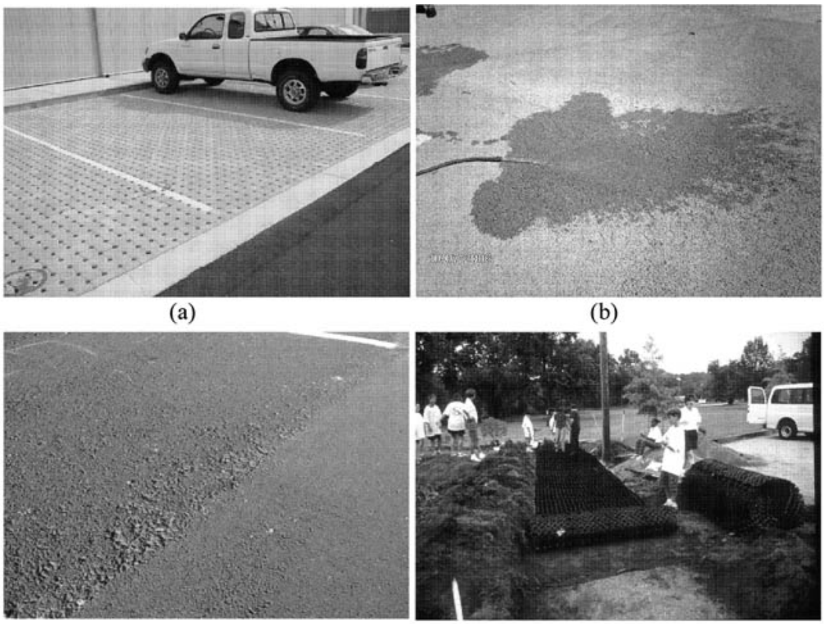
Permeable pavement is pavement which has openings which allow water to pass through it rather than forcing runoff to shed off it. Examples include permeable asphalt, concrete, or open celled pavers. Under the surface layer is a gravel storage layer ranging in depth from 10 to 30 cm (4 to 12 in.) and, in some cases, an underdrainage system. When it rains, rainfall passes through the pavement and partially fills the gravel storage layer, where it infiltrates over an extended period of time. There are 5 types of permeable pavements, Permeable Asphalt (PC), Permeable Concrete (PC), Permeable Interlocking Concrete Pavers (PICP), Concrete Grid Pavers (CGP), and Plastic Grid Pavers (PG). Pictures of four pavement types are shown in Figure 12.11.

Unlike traditional surfaces, permeable pavement allows water to pass through its surface. All permeable pavement types essentially perform in the same manner. After water migrates through the surface, it temporarily collects in the gravel storage layer (Figure 12.12). Depending upon the rainfall intensity, rainfall volume, and existing soil infiltration rate, water then either exits the bottom of the permeable paver via soil infiltration or under drain pipe, or water inside the pavement will build up until runoff occurs. Very intense rainfall rates can produce runoff from permeable pavement, particularly on concrete grid paver systems filled with sand. For runoff that passes through the pavement, many pollutants can be trapped inside the pavement or removed as the water passes out of the pavement into the surrounding soil.

Permeable pavements can be specifically designed to optimize infiltration. Designers can adjust the following parameters:

- Depth of storage layer
- Surface Infiltration

- Underdrain need
- Underdrain Configuration
- Location of pavement for best In Situ Soil (to a lesser extent)



(a) (b) (c) (d)

Figure 12.11: Four types of permeable pavement: (a) Permeable Interlocking Concrete Pavement (PICP), (b) Pervious Concrete, (c) Pervious Asphalt, and (d) Plastic grid pavement

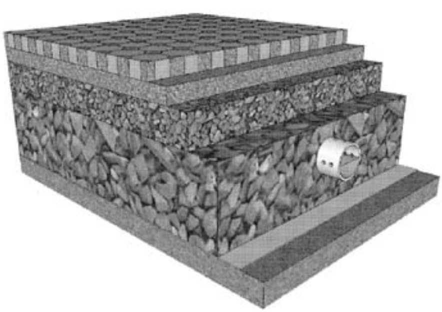


Figure 12.12: Schematic cross section of PICP type permeable pavement

12.2.5.1 Depth of Storage Layer

On average, each inch of gravel base can store 1/3 inch of runoff. So, a 22.5 cm (9-inch) gravel storage layer holds 7.5 cm (3 inches) of rainfall at a given moment, assuming the drainage layer is flat. For structural reasons, described in AASHTO (1993) a gravel storage layer is needed for most permeable pavement types (with the notable exception of pervious concrete). Provided the underlying soils allow some infiltration to occur, deeper storage layers therefore allow more water to infiltrate during large storm events.

12.2.5.2 Surface Infiltration

The type of pavement used has a minor effect on surface infiltration, with pavements employing a sand or sandy soil fill having lower surface infiltration rates than pavements designed with pea gravel fill or pervious concrete or pervious asphalt. The difference, however, among pavement types long term is not significant (Bean et al., 2007a). Rainfall intensities of 1 – 2 inches per hour may cause runoff from CGP filled with sand and PG filled with sand. Rainfall intensities of 4 inches per hour may cause runoff from the other pavement types, per Bean et al.'s study.

12.2.5.3 Underdrain Need

The need for underdrains is dependent upon the in situ soil. If the soil infiltration rate is sufficiently reduced, an underdrain is required. It is reasonable to expect the pre-construction infiltration rate to be decreased by a factor of 10 to 20 following construction due to soil compaction by heavy equipment (Gregory et al., 2006; Pitt et al., 2008). For given design needs, an infiltration rate like this may be too low and underdrains will be used.

12.2.5.4 Underdrain Configuration

Collins et al. (2008) found that an underdrain that creates a storage zone in the bottom of the pavement base layer can reduce outflow volumes. A specific study, however, has yet to be conducted on this design feature. Water that initially pools internally in the pavement (1) does not drain and (2) can slowly infiltrate the subbase, increasing times to peak, reducing runoff volumes, and lowering peak outflow rates. A design option for underdrains is to size them so that they have limited outflow rates. That is, use underdrains with a small diameter. Another option is to cap the underdrains with a restrictive orifice, or hole. While this might not substantially reduce outflow volumes, it would dramatically reduce peak flows and increase times to peak for a given storm event. Doing this is akin to using a small orifice to dewater a pond or wetland over a 2- to 3- day period.

12.2.5.5 Pavement Location in "Best" In-situ Soil

A developed site may have surprisingly varied underlying soils. Some of which may be considered impermeable while others will have some permeability. If the designer is able to identify locations with somewhat permeable underlying soils, the permeable pavements will potentially infiltrate a substantial amount more. The infiltration volume has been shown to be potentially very high, provided many of the previously discussed design and siting features have been followed. One site in Swansboro, North Carolina, with an underlying sandy soil and 50 cm deep gravel base infiltrated all of the precipitation that fell on it, including 5 storms exceeding 50 mm (Bean et al., 2007b). Another study of pervious concrete in Villanova, PA, had similar results. Over a multi-year period all storms less than 5 cm were completely infiltrated at the Villanova site, routinely capturing a minimum of 95% of the yearly average 115 cm regional rainfall average (Kwiatkowski et al., 2007). The site has a low drainage area to bed surface ratio, is directly connected to surrounding downspouts, and again shows the same seasonal effects due to the impact of temperature on viscosity (Braga et al., 2007). Contrastingly, a site with tighter soils and underdrains "only" reduced annual runoff volumes by approximately 40% (Collins et al., 2008) in Kinston, NC. This lower value matched what Gilbert and Claussen (2006) found for a paver installation in Connecticut. Clearly, because so much water can infiltrate from a surface that is designed for automobiles, questions regarding infiltrating contaminants arise.

12.2.6 Swales

Grass swales are shallow, grass-lined, typically flat-bottomed channels that were originally implemented for stormwater conveyance (Barrett et al. 1998). They are an inexpensive way to convey water from a source to a treatment practice or an exit from the property. Swales are commonly used on highway projects because they represent a simple, aesthetically pleasing technique for conveying runoff along linear systems. They are usually turf, but can sometimes grow wetland vegetation (Figure 12.13). While recent studies have revealed grass swales as an effective stormwater management technology, good performance data and mechanistic understanding of swale design parameters are not widely available, in large part because of the complexity of swale operation. In addition to longitudinal flow in the direction of the swale, swales may receive flow laterally through vegetated side slopes, which could improve incoming water quality. Infiltration throughout the swale surface area can reduce total runoff volume. However, the multiple points of water input and discharge can complicate performance analyses

The standard highway swale is designed to convey runoff from the largest storm events away from the roadway. Because of this, highway swales commonly are not specifically designed for smaller storm events (0.5 – 2.5 cm) that produce the majority of annual runoff through the swale (Schueler, 1994). The hydrologic effectiveness of grass swales as a BMP is dependent on attenuation of peak runoff flow by vegetation, reduction of total runoff volume through infiltration, and increase in travel time (Jenson, 2004).

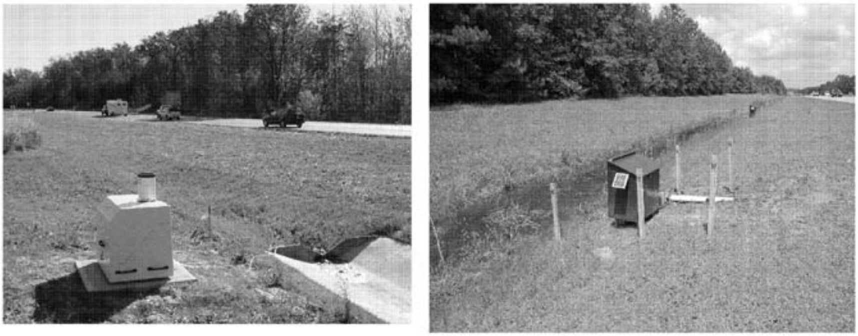


Figure 12.13: Two monitored swales, including a "dry" swale in Maryland (a) and a wet swale in North Carolina (b)

In cases where the swale intersects the SHWT, it can go "wet." If vegetation is allowed to follow its natural course, then this swale will eventually become a "wetland swale." Little research has been done on this type of swale's effectiveness, however it is assumed that a wetland swale would have better nutrient (particularly nitrogen) removal rates than standard swales. Wetland swales do have a higher roughness coefficient (used to calculate swale geometry). Exact roughness coefficients have not been measured, but a Manning's n of 0.050 may be assumed to be a reasonable approximation for a fully grown wetland swale. A second impact of wetland swales is that much of the swale's cross section is now occupied by vegetation. In a standard low-cut grass swale essentially all of the swale's cross-section is open space. With a wetland swale, when vegetation is overgrowing the bottom, an estimated 10% of the cross sectional area (internal NCSU study) is now taken up by plant mass. The higher roughness coefficient and the decreased amount of free space will lead a wetland swale's cross-section to be larger than an "equivalent" dry swale.

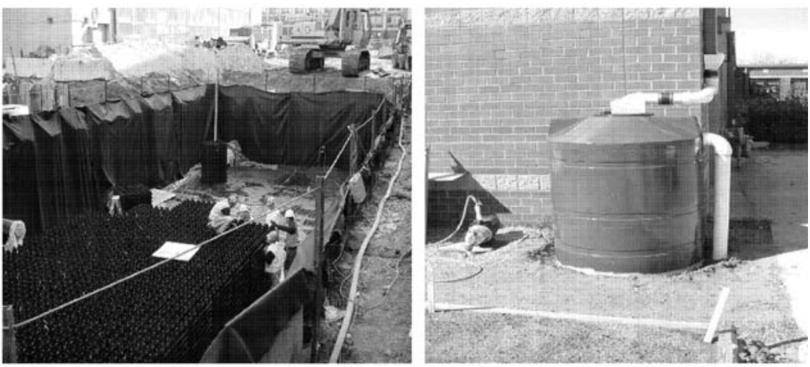
Like vegetated filter strips, the underlying soil of swales can be amended or even simply core-aerated, to improve infiltration. Commonly seen as a construction sediment erosion control measure, swales can have small check dams built along their length, forcing water retention in the swale. More expensive options include importing a sandy fill soil to either replace existing underlying soil or to mix with in situ soils. The marginal effectiveness of none of these techniques has been reported in literature.

Peak flow attenuation in grass swales was examined by Wu et al. (1998) using the peak discharge factor (PDF), defined as the ratio of peak discharge of runoff to total rainfall amount. Grass swales reduced the PDF by 11-22% when compared to direct highway runoff (Wu et al., 1998). Reduction of total runoff volume through infiltration in grass swales has been reported as 30-47% (Rushton, 2001) and 33%

(Backstrom, 2003). Grass swale efficacy appears to also be dependent on storm characteristics. Several studies found that during small storms, reduction of total runoff volume was significant. However, as would be expected due to soil saturation, during large or intense storms, the total volume of runoff discharged from the grass swales was equal to or sometimes larger than that entering the swale (Schueler, 1994; Yu et al., 2001; Rushton, 2001).

12.2.7 Water Harvesting – Irrigation Systems

Cisterns and water harvesting systems capture rainfall (usually from rooftops) in a large sealed container and store it for later use. Cisterns can be either above or below ground (Figure 12.14). A cistern is sized to optimize a balance of water demand met, the frequency of a dry cistern, the amount of water (and nutrients) captured, and cost (payback period). Various solutions might capture the majority of runoff, but be quite costly. In other cases, it is important for the cistern to rarely go dry, so an otherwise over-sized cistern might be most appropriate. Cisterns harvest rainwater from rooftops and temporarily store water for uses such as irrigation, washing vehicles, washing laundry, and flushing toilets. Cistern water is most easily used for non-potable (non-drinkable) purposes; however, with special treatment, harvested rainwater can even be consumed.



(a) (b)

Figure 12.14: Cistern examples: (a) A multi-hundred thousand gallon underground cistern in Cambridge, Mass., on the campus of MIT (courtesy of Nitsch Engineering) and (b) a multi-hundred gallon above ground cistern in Wilmington, NC

The water harvesting system consists not only of a cistern, but also a pipe network diverting rooftop runoff to the cistern, an overflow bypass for when the cistern is full, and a pump and distribution network to deliver water to its intended use (Figure 12.15). Tanks, or cisterns, can be quite small, less than 380 L (100 gallons), to well over 380,000 L (100,000 gallons) for a commercial or institutional site. The cisterns may rest on the surface or be located entirely below ground. Tanks are made of plastic, metal, or concrete, depending upon the cistern's size and location. Economies of scale certainly exist. Large-scale water harvesting is practiced frequently in Florida and in parts of the Southeast. Golf course ponds have been used to capture stormwater runoff and re-use the runoff for green and other course irrigation. Modeling has shown that in certain instances, the payback period for cistern systems is less than 10 years

One common small scale cistern is the rain barrel. Rain barrels are typically less than 380 L (100 gallons) in size and can be used for limited water needs, such as in a garden. While rain barrels serve an excellent demonstration and awareness purpose, they rarely contribute a significant amount to runoff reduction due to their small size. Runoff is routed to a cistern during a rain event. Either the runoff will fill the cistern, at which point overflow occurs, or the storm will be completely captured. In the intervening period between storm events, stored water in the cistern is used to meet a series of demands, such as irrigation, vehicle washing, and toilet flushing. The use of cistern water vacates a portion of the cistern for subsequent precipitation events. The amount of harvested water that infiltrates depends upon the end use of the water. Water used to irrigate clearly results in larger infiltration volumes than water used to flush toilets, unless that latter is directed to a septic field. Special care can be taken when using harvested cistern water so that irrigation is maximized. For example, washing vehicles on a lawn allows water to infiltrate while washing a vehicle on connected impervious surfaces does not afford much infiltration opportunity. In most water harvesting operations, water is drawn from the cistern via a pump.

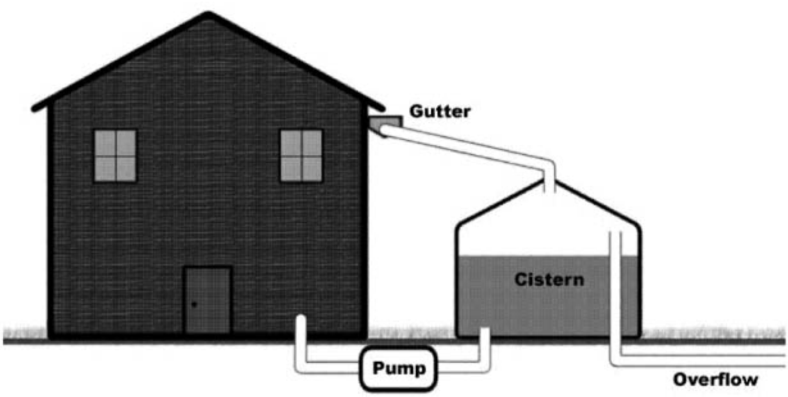


Figure 13-15. Water Harvesting system layout schematic

Capturing rain water in a cistern by itself does not produce infiltration. It is the demands/uses of this captured water that cause water harvesting systems to produce infiltration. Several uses, such as toilet flushing or use in washing machines, typically do not result in infiltration. However, two common uses for non-potable water do result in potential infiltration: irrigation and washing vehicles. It is assumed that cistern water harvested for irrigation is applied in a manner such that it infiltrates the ground. Vehicle washing that flows onto the permeable landscape (grass) resembles irrigation and is most apt to occur on residential property.

Limited field research has been conducted on the stormwater benefits of water harvesting. However a few modeling studies have shown that water harvesting can capture a substantial amount of runoff if a reliable demand exists (Guo and Baetz, 2007; Coombes and Barry, 2007; Jones and Hunt, 2009). Several field installations of water harvesting systems were monitored for multiple year periods, including one for vehicle washing and another used to irrigate (Jones and Hunt, 2009). Results of the monitoring study showed that the rainwater harvesting systems were underutilized (Figure 12.16), which was suspected to result from poor estimation of water usage and public perception of the harvested rainwater. Captured water volumes that could potentially infiltrate were less than 20% of annual rainfall due to under-utilization.

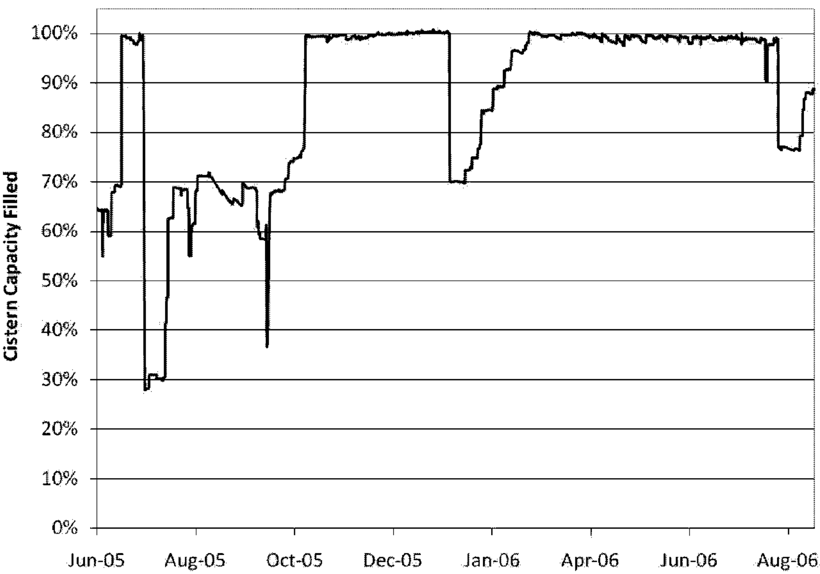


Figure 12.16: Cistern water level over a 14-month period, showing the system rarely less than 50% full

12.3 APPLICATIONS OF LOW IMPACT DEVELOPMENT BMPs

Four illustrative case studies are presented in this section. The first study is an examination of swales in Maryland and their ability to increase infiltration and reduce peak flows; the second study examines a possible excavation technique studied in North Carolina that could be incorporated as part of bioretention and other infiltration BMP construction; the third study compares hydrologic performance among four different types of permeable pavement systems examined in North Carolina; and the fourth study focuses on accounting for infiltration on a seasonal temporal basis in permeable pavement and bioinfiltration in Pennsylvania.

12.3.1 Maryland State Highway Administration/ University of Maryland Swale Study

Researchers at the University of Maryland- College Park (Stagge, 2006) examined two sales along MD Route 32 near Savage, Maryland, which was a four-lane (two in each direction) limited access highway. Two swales were constructed in the highway median to receive runoff laterally from the southbound roadway lanes. The first, known as MDE, is a swale constructed based on state guidelines, with a 15.2 m sloped (6%) grass pretreatment area between the roadway and the swale channel (MDE, 2000).

The second swale, just to the north, known as SHA, was similarly constructed, but lacked a pretreatment filter area. Both swales converged at an inlet where water flow measurements were made. Both were constructed with identical cross-section designs (side slopes of 3:1 (33%) and 4:1 (25%) on either side of the swale), a 0.61 m bottom width, and approximately 1.4% longitudinal slope. Topsoil used in the swales had a grading distribution of 20-75% sand (2.0-0.050 mm) by mass, 10-60% silt (0.050-0.002 mm), and 5-30% clay (less than 0.002 mm). Grass used for the swales and pretreatment area was initially composed of 90% tall fescue, 5% Kentucky bluegrass, and 5% perennial ryegrass.

Because runoff entering the swale was distributed along its length, a third sampling area was designed and constructed to collect and sample runoff directly from the highway (known as Direct), south of the swales. In this area, a concrete channel was constructed directly along the roadway shoulder parallel to the roadway. This allowed an accurate representation of instantaneous swale input flow from the roadway surface without disrupting flow into the swales. Sampling zones were designed so that all three swales had nearly identical roadway drainage areas. Table 13-2 presents specific design parameters for these 3 channels.

Table 12.2: Design characteristics for three sampled channels (Stagge, 2006)

	Direct	SHA Swale	MDE Swale
Roadway Area (ha)	0.271	0.224	0.225
Swale Area (ha)	0	0.169	0.431
Total Area (ha)	0.271	0.393	0.656
Modified Area (ha) $=A_S+C_RA_R$ (Eq. 7)	0.257	0.382	0.645
Channel Material	Concrete	Grass	Grass
Channel Slope	0.2%	1.6%	1.2%
Channel Length (m)	168	198	137
Pretreatment Slope	-	-	6%
Pretreatment Length (m)	-	-	15.2 (from roadway to channel center)

Overall, the swale research system was designed as an input/output study. Stagge (2006) considered the flow from the direct highway runoff to be equal to the influent characteristics for each swale. This assumed influent was compared to flow measured at the outlet of each swale. Flow characteristics were, thus, directly calculated for each storm event. Additionally, the performance characteristics for each swale were directly compared to one another. An area-based normalization allowed a standard comparison between swales with differing grass pervious areas.

12.3.1.1 Results

Stagge (2006) anaylzed 24 rainfall events over 19 months. Sampled total rainfall depths varied between 0.15 and 17.32 cm, with a mean of 2.44 cm. Storm durations varied between 0.2 hours and 29 hours with a mean of 9.36 hours. Only 13 (SHA) and 12 (MDE) storm events produced measurable flows at the outfall structure of the grass swales; the remaining 11 and 12 events were completely captured and assimilated by the swales, with no measurable outflow. Overall, the monitoring data suggested that the grass swales are effective at completely capturing 50% of storm events, reducing the total runoff volume for 20% of events, but have little effect on total volume for the largest 30% of storm events.

Both swales were very successful in reducing the distribution of peak flow, as shown in Figure 12.17. Regression lines indicate that the population distribution of peak flows for each of the three channels are lognormally distributed, with the majority of peak flows in the lower ranges and a small occurrence of high peak flows caused by large storm events. Both swales demonstrated complete attenuation of flows for nearly 50% of the monitored storm events, as discussed above, due to infiltration and evapotranspiration. The nonparametric Wilcoxon test (Wilcoxon, 1945) concluded that both swales exhibited significant peak flow attenuation as compared to the input.

12.3.1.2 Study Findings

Stagge (2006) found the effect of the grass swales on total volume reduction to not be constant and showed three distinct treatment modes. For the lowest intensity rainfall events, the swales completely captured all runoff, such that no measurable flow occurred at the swale outfall. Using data on storm events in the state of Maryland (Kreeb and McCuen, 2003), grass swales using these design parameters should completely capture two-thirds of storm events in Maryland. For events with higher rainfall intensities, these swales were effective at reducing the total runoff volume through infiltration; however, they begin to lose effectiveness above a threshold limit that corresponded to a rainfall depth of approximately 3.3 cm. This depth corresponds to a relatively large storm event, which is expected to occur in less than 14% of events in Maryland. The cumulative effect of these three treatment conditions is that swales successfully reduce the total runoff volume by an average of 46-54%.

12.3.2 Increasing Infiltration with Alternative Excavation Techniques

Brown and Hunt (2009) examined how the construction of bioretention cells impacts the in-situ soil's ability to exfiltrate stormwater, thus promoting groundwater recharge, as opposed to discharging stormwater through underdrains. Their study looked at different construction methods to promote a higher infiltration rate to the in-situ soil layer. More exfiltration from the bottom of the bioretention cell to the in-situ soil would decrease outflow to the storm drain network. Brown and Hunt (2009) explored two phases of the construction process. The first was to determine how different backhoe excavation techniques affect soil integrity, and the second was to determine the impact of excavating in wet soil versus dry soil. The two excavation techniques and two soil moistures were tested in two different soil types – predominantly clay and predominantly sand.

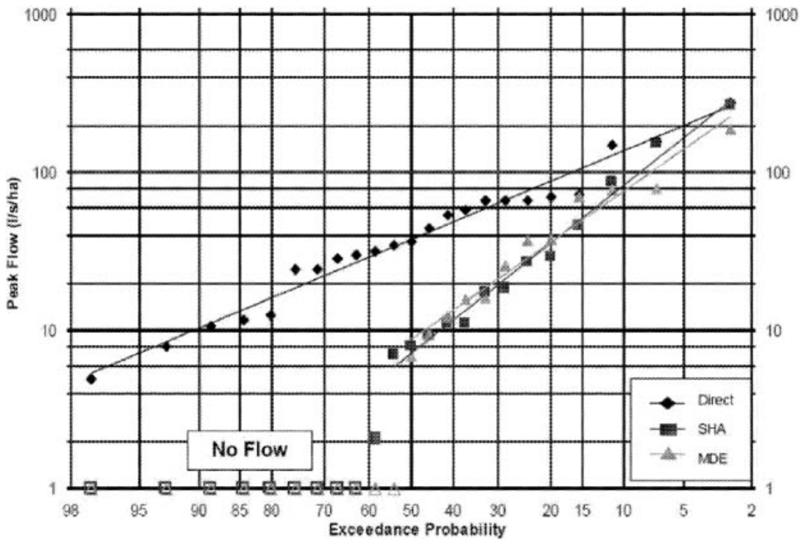


Figure 12.17: Peak flow distribution for all storm events for influent (Direct) and swale discharge (SHA and MDE)

An expert excavator who understood the importance of using consistency in the excavation techniques for the purposes of research was contracted for this project to construct bioretention cells. The excavation techniques were a "rake" versus a "scoop" approach. Examples of the two methods were earlier presented in Figure 12.4. The "rake" approach used the teeth of the backhoe bucket to scarify and till the surface, where the "scoop" technique had more smearing and compaction associated with it. Due to the maximum compaction levels occurring between 20 to 30 cm (8 to 12 in) during construction in Gregory et al. (2006), emphasis was placed on using consistency in technique for excavating the final 30 cm (12 in) of soil.

The second phase studied was excavating in different soil moisture conditions – relatively wet soil versus relatively dry soil. In order to test the difference of moisture conditions, excavation in dry soil took place after several consecutive dry, warm days. To test for excavation in wet soil, the top layer of soil was excavated, leaving approximately 30 cm (12 in) to the proposed bottom layer of the bioretention cell. An earthen berm was built around the testing area, and it was manually irrigated overnight to saturate the soil. By removing the top layer of soil, the soil at the proposed bottom layer of the bioretention area became saturated quickly so final excavation could proceed on the following day. This was designed to replicate finishing excavation the day after rainfall. NC design standards recommended a fill media depth of 0.6-1.2 m (2-4 ft), and 0.76 m (30 in) is recommended for nitrogen treatment (NCDWQ, 2007). Using this as guidance, a typical fill media depth of 0.6-0.9 m (2-3 ft) was used for this study to make sure the excavation depth was consistent with typical bioretention cell construction.

12.3.2.1 Results

Brown and Hunt's (2009) results are displayed in Table 12.3 for the sandier soil location (Nashville, NC) and Table 13-4 for the more clayey soil location (Raleigh, NC). At both locations, with both of the soil types tested, infiltration and saturated hydraulic conductivity increased when the raked method was used. This finding was accompanied by increased bulk density associated with the scoop method. As the compaction and bulk density increased, water movement through the soil slows down. At the Raleigh locations, constructing in wetter soils had a negative impact.

Table 12.3: Results from soil tests for hydraulic conductivity (K_{Sat}), infiltration, and dry bulk density for Nashville site.

		K_{Sat} (cm/hr)			Infiltration (cm/hr) (n=3)		Dry Bulk Density (kg/cm^3)		
Site	Type	Average	Std Dev	n	Average	Std Dev	Average	Std Dev	n
"Dry" / (sandy-clay)	Scoop	3.98	3.58	4	3.01	1.54	1.74	0.068	4
	Rake	7.31	6.08	6	6.71	5.01	1.70	0.018	6
"Wet" / (sandy)	Scoop	7.93	6.77	5	43.6	16.8	1.67	0.035	5
	Rake	21.6	7.45	6	61.9	26.5	1.61	0.035	6

Table 12.4: Results from soil tests for hydraulic conductivity (K_{Sat}), infiltration, and dry bulk density for Raleigh site.

		Ksat (cm/hr)			Infiltration (cm/hr) (n=3)		Bulk Density (kg/cm^3)		
Site	Type	Average	Std Dev	n	Average	Std Dev	Average	Std Dev	n
"Dry" / (clay)	Scoop	1.81	2.13	4	0.43	0.50	1.50	0.080	6
	Rake	2.29	2.88	6	0.78	0.52	1.63	0.075	6
"Wet" / (clay)	Scoop	0.62	0.77	5	0.24	0.06	1.37	0.103	6
	Rake	4.37	5.65	6	1.20	0.27	1.17	0.054	6

12.3.2.2 Study Findings

Based on the data Brown and Hunt (2009) collected, excavating the final 30 cm (12 in) using the teeth on the bucket to rake the surface instead of using the bucket to scoop and make the surface smooth improved the soil properties that govern infiltration. The rake method scarified the bottom layer in the bioretention cell creating more pore spaces and a lower bulk density, which helped promote the soil's ability to exfiltrate water from bioretention cells to the underlying soils. The potential for exfiltration was reduced when using the scoop method because it created more compacted soils with higher bulk densities.

In particular, when examining the rake method in wet conditions, the hydraulic conductivity was significantly less at the sandy soil site (p-value = 0.044) and the infiltration rate and hydraulic conductivity were significantly less at the clay soil site (p-values = 0.004 and 0.051, respectively). Under dry conditions, there was no statistical significance associated with excavation technique, but the trend showed improved infiltration and hydraulic conductivity when using the rake method. Brown and Hunt (2009) recommended that due to little extra cost associated with the rake method and this technique's apparent benefit, the rake technique should be specified by designers in future bioretention construction projects.

12.3.3 Choosing Permeable Pavement Types per Hydrology

Collins et al. (2008) examined a permeable pavement parking lot in eastern North Carolina consisting of four types of permeable pavement and standard asphalt from June 2006 to July 2007 for hydrologic differences in pavement surface runoff volumes, total outflow volumes, peak flow rates, and time to peak. The 20-stall employee parking lot was constructed in January 2006 at the City of Kinston Public Service Complex in eastern North Carolina. The lot was comprised of six 6 m by 18 m pavement sections: two standard asphalt sections each containing two parking stalls and four different permeable pavement sections each containing 4 parking stalls (Figure 12.18). The four permeable sections were comprised of the following types of pavement:

- Pervious concrete (PC),
- Permeable interlocking concrete pavers with 12.9% open surface area and openings filled with No. 78 stone (PICP1)
- Concrete grid pavers with 28% surface open areas and opening filled with sand (CGP)
- Permeable interlocking concrete pavers with 8.5% surface open areas and openings filled with No. 78 stone (PICP2).

The site investigation found that at a depth of approximately 50 cm, the in-situ soil type changed from sandy loam to sandy clay loam.* *All permeable sections overlaid a washed ASTM No. 78 stone aggregate bedding layer and a washed ASTM No. 5 stone base course layer, the depths of which varied slightly based on the product specifications of the overlying pavement types. The base course layer was designed to support the expected parking lot traffic loading, estimated as 60 vehicle passes per day. For ease of installation, the excavation depth beneath permeable pavements was kept consistent, so the aggregate storage layer was adjusted for all sections to meet the strength requirements for the section which limited pavement design. Each pavement section was designed to be hydraulically separate from the other sections.

Due to the low permeability of the in situ soils, perforated corrugated plastic pipe (CPP) underdrains (d=10cm) were installed at the bottom of the each permeable pavement aggregate base course layer to drain water from the system, thereby creating separate "cells." The aggregate subbase of each permeable pavement section sloped to the corrugated underdrains in the center of each pavement section at a 30:1 side slope. The pavement cells were unlined, to allow for some potential exfiltration of water into the subsoil. The entire parking lot, excluding the entrance ways, was designed with a 0.42% surface and sub-grade slope to provide drainage and allow for monitoring.

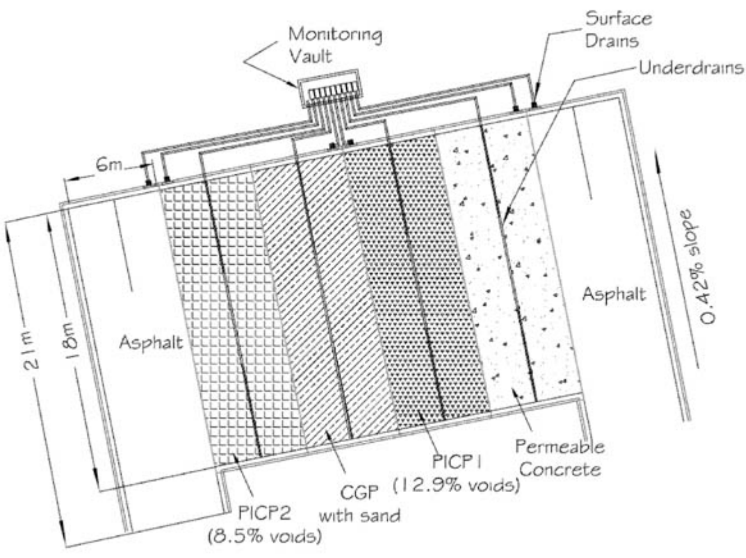


Figure 12.18. Permeable pavement parking lot design (plan view)

12.3.3.1 Results

When compared to asphalt, all four permeable pavement sections dramatically reduced surface runoff volumes. The marked reductions in permeable pavement surface runoff were similar to results found by Bean et al. (2007b), Brattebo and Booth (2003), and Valavala et al. (2006) on unclogged pavement sites. Mean runoff reductions from rainfall depth were 34.7, 99.9, 99.3, 98.2, and 99.5% for asphalt, PC, PICP1, CGP, and PICP2, respectively (Table 12.5). Surface runoff volumes of all pavements were statistically different from one another ($p<0.01$). Expressed in order of highest runoff generation, pavements performed as follows: Asphalt>>CGP>PICP1>PICP2>PC. All pavement surface runoff volumes were positively correlated to rainfall depth and intensity ($p<0.05$). Simultaneous runoff from all five pavements was generated during only two rainfall events: Tropical Storm Ernesto and the 22 November 2006 Storm. Rainfall depth during these events exceeded 180 mm and 130 mm, respectively.

The majority of the total outflow for all permeable pavement sections occurred not as runoff, but as subsurface drainage (Table 12.6). The PICP1 and CGP cells generated significantly less outflow volumes than the other pavement sections ($p<0.001$) and no statistical difference between these cells was observed. The total outflow volumes of the PC cell, PICP2 cell and asphalt were not statistically different from one another. Average percent volume reductions from rainfall volume were 35.7, 43.9, 66.3, 63.6, and 37.7% for asphalt, PC, PICP1, CGP, and PICP2 cells, respectively. The response of the PICP1 cell was likely attributed to an increased subsurface storage volume in the bottom of the pavement, and a consequential increase in exfiltration, due to an elevated underdrain outlet. However, the hydrologic response of the CGP cell is presumed due to water retention within the pore spaces of the sand filling the pavement surface openings. Outflow volume reductions for the permeable system that did occur can be attributed to retention within the pavement void fill media and aggregate subbase and some small degree of subsoil infiltration. Because the system was designed to drain, the degree of infiltration was likely limited. The one exception was the PICP1 cell, in which an elevated underdrain left some water ponded in the system after each storm event, allowing more time for infiltration.

Table 12.5: Percent surface runoff reductions from rainfall depth

Parameter	Asphalt (n=44)	PC (n=40)	PICP1 (n=41)	CGP (n=40)	PICP2 (n=40)
	---Percent Runoff Reduction from Rainfall (%)---				
MEAN	34.65	99.86	99.33	98.17	99.51
MEDIAN	29.43	99.94	99.37	98.67	99.68
MIN	-2.73	99.03	97.76	91.11	96.94
MAX	84.80	100.00	100.00	100.00	100.00
STDEV	18.71	0.22	0.58	1.83	0.65

Table 12.6: Percent outflow volume reductions from rainfall depth

Parameter	Asphalt (n=54)	PC (n=51)	PICP1 (n=50)	CGP (n=50)	PICP2 (n=46)
	---Percent Outflow Reduction from Rainfall (%)---				
MEAN	35.70	43.88	66.29	63.63	37.68
MEDIAN	30.20	49.30	65.55	67.24	35.55
MIN	-2.73	-9.44	20.70	10.48	-9.13
MAX	92.33	92.09	100.00	99.96	84.02
STDEV	21.76	23.68	23.22	26.46	23.85

12.3.3.2 Study Findings

With respect to runoff reduction and peak flow mitigation, Collins et al. (2008) found all pavements performed substantially and statistically significantly better than asphalt ($p<0.001$). Although hydrologic differences among the pavements did exist, they were small in comparison to the overall improvements from asphalt. The PICP1 and CGP cells were able to store up to 6 mm of rainfall without yielding any outflow, a volume accounting for roughly 30% of the median rainfall event that occurred during the study. It is likely that the response of the PICP1 cell was due to an increased subsurface storage volume resulting from an elevated outlet pipe; whereas, the CGP cell response was likely an effect of the sand fill media. The sand layer of the CGP cell likely retained the greatest amount of runoff, and was most effective in mitigating peak rainfall intensities. Sand, which has been viewed as a detriment because of increased surface runoff, appears to have the benefit of holding additional water, which then slowly leaks or evaporates. Collins et al. (2008) suggested that incorporating a sump beneath permeable pavement applications, provided underlying soils were supportive, would appreciably improve the performance of permeable pavement systems.

12.3.4 Villanova University Seasonal and Long Term LID Practice Infiltration Study

Variations in the infiltration processes for a Bioinfiltration Traffic Island (BTI) and Pervious Concrete Infiltration Basin (PCIB) built on the campus of Villanova University were the focus of Emerson and Traver (2008). Continuous monitoring data was collected at both sites. Emerson and Traver's work focused on the long-term and seasonal changes of the infiltration process at the PCIB and BTI infiltration BMPs. Their study's objective was to provide information and an understanding of the infiltration process itself which may be relevant and applicable to other LID infiltration-based applications.

12.3.4.1 Pervious Concrete Infiltration Basin

The Pervious Concrete Infiltration Basin (PCIB) was a retrofit of an existing conventional asphalt paved common area between two dormitories and was constructed in 2002. The PCIB consisted of three underground storage beds filled with crushed stone aggregate (~9 cm average diameter) to create a void space of approximately 40%. Areas of standard concrete overlay the storage bed and are surrounded by a perimeter of pervious concrete. The majority of the runoff that the BMP receives comes from the slate roof tops of adjacent four-story dormitories which are directly connected to the subsurface storage beds. One of the storage beds (~180 m^2 or ~1,900 ft^2) had been instrumented with various monitoring equipment including pressure-temperature transducer. The depth and temperature data from the storage bed, collected over a two-year period, was used as the basis of Emerson and Traver's study. The storage bed was located a minimum of 4.6 m (15 ft) above the local groundwater levels. A soil sample from the base of the lower storage bed was silt (ML) according to the Unified Soil Classification System (USCS) (Kwiatkowski 2004). The soil would likely be classified as a Loam according to the USDA textural triangle.

12.3.4.2 Bioinfiltration Traffic Island

As in the case of the PCIB the Bioinfiltration Traffic Island (BTI) studied by Emerson and Traver (2008) was also a retrofit. In 2001 a curbed turf area located in a dormitory/student apartment parking area was converted to an infiltration BMP. The area enclosed by a curb was first excavated to approximately two meters below the curb elevation. The excavated soil, a silt (ML) according to the USCS, was then mixed with an imported sand (poorly graded sand (SP)) at approximately a 1:1 ratio. The resulting soil mixture was placed back into the excavated cavity to a depth of approximately 1.2 m (4.0 ft) which left a shallow surface depression (bowl) (Heasom et al., 2006). The open surface depression had an overflow weir located at 52 cm (1.7 ft) above the lowest point in the bed. The bed was relatively flat with an average depth prior to overflow of 25 cm (0.82 ft). The shallow depression occupies 144 m^2 (1,550 ft^2) when full, and captures runoff from a 0.52 ha (1.3 ac) watershed which is covered by approximately 35% directly connected impervious surfaces. This results in a directly connected impervious cover to BMP area ratio of 10:1. The composite soil mixture is classified as a silty sand (SM) and would be considered a sandy loam according to the USDA textural triangle. The mixture contained approximately 30% fines passing the No. 200 sieve. Machinery was never allowed to drive over the BMP footprint to avoid any compaction of the underlying and mixed soil. The shallow depression was covered in bark mulch and planted with vegetation. The groundwater elevation adjacent to the BMP generally remained approximately 4.3 m (14 ft) below the base of the storage bed.

Among other hydrology-related parameters, the depth of ponded water within the storage bed had been monitoring continuously using an ultrasonic transducer at a five-minute interval for more than four years. Continuous temperature data was collected at the soil surface for two years at the BMP. Both infiltration BMPs ponded relatively soon after the onset of rainfall and consequent inflow. Soil moisture meters located at shallow depths (0.3 to 0.6 m) beneath both BMPs indicated that the soil promptly reached a steady moisture content that sustained until after the end of ponded conditions. These moisture contents were generally reached prior to recession of the ponded depth. This indicated to Emerson and Traver (2008) that the steady ponded recession rate as measured by the BMP instrumentation was representative of late-time or steady infiltration governed by the saturated hydraulic conductivity. Since the geometry of the storage basins are relatively flat; it was assumed that the infiltration process was adequately represented by a vertical, one-dimensional approximation. Therefore the ponded recession rate, corrected for the storage bed porosity, is assumed to be a representative estimate of the hydraulic conductivity of the soil surface within the BMP. This estimate was used to normalize the performance of the BMPs. Changes in the recession rate over time represented changes in the hydraulic conductivity. The influences of the shallow ponded depths were considered negligible. This analysis did not account for potential changes during matric suction-dominated early-time infiltration and the inherent imprecision caused by air entrapment.

12.3.4.3 Results

Emerson and Traver's (2008) analysis of the PCIB data resulted in 15 separate estimates of hydraulic conductivity; the results had a mean and standard deviation of 7.2 and 2.0 cm/day respectively. The BTI data included 123 estimates with a mean and standard deviation of 13 and 4.9 cm/day, respectively. The estimates of hydraulic conductivity area plotted over time to determine if there are any significant variations apparent in the data, as is shown in Figures 12.19 and 12.20. In contrast to other longevity based assessments of infiltration BMP performance, the results for both BMPs show no obvious signs of a systematic change in performance over the period of record. What is noteworthy, however, was the amount of variation and apparent cyclic/seasonal pattern. This cyclic variation can best be seen in the seven-point moving average (solid line) fit to the data from the BTI, as shown in Figure 12.20.

Emerson and Traver (2008) proposed a linear regression model to quantify the temperature dependency of hydraulic conductivity estimates at the two infiltration BMPs. A variation on this method was then used to determine what portion of the observed temperature dependency could be attributed to temperature-induced viscosity effects. The linear regressions for both BMPs were prepared with the ponded water temperature as the independent variable and the hydraulic conductivity estimates as the dependant variable. Then the average temperature for all observations was found for each BMP. This average temperature was used with the equations developed by linear regression models to provide a best estimate of hydraulic conductivity at the average temperature.

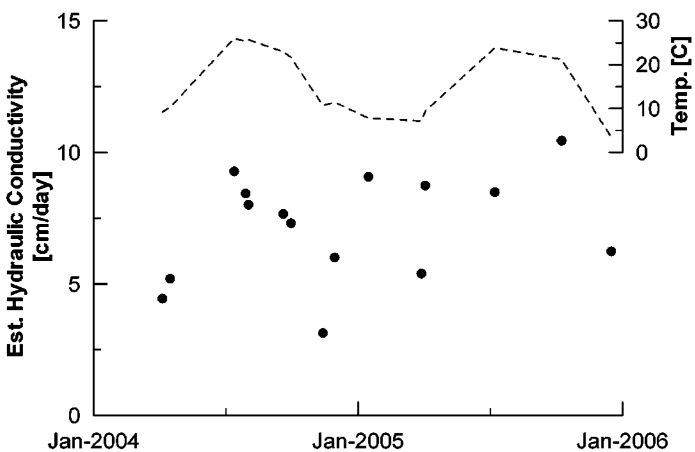


Figure 12.19: PCIB hydraulic conductivity in Villanova, PA. (Emerson and Traver, 2008)

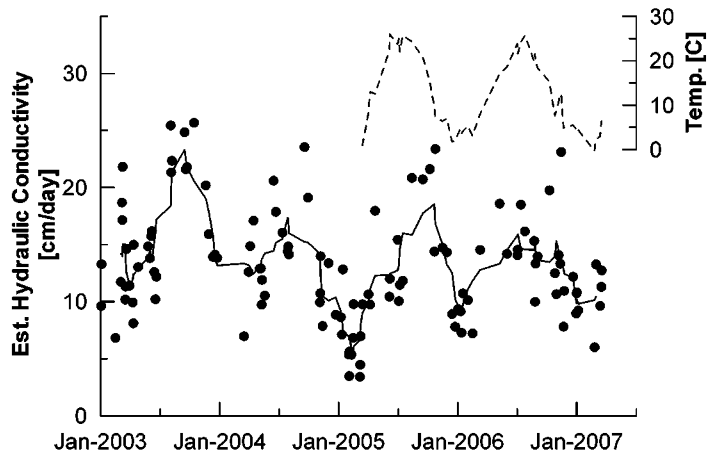


Figure 12.20: BTI hydraulic conductivity in Villanova, PA. (Emerson and Traver, 2008)

For the PCIB the slope of the observed and predicted temperature dependency were remarkably similar with the predicted dependency being slightly higher, and well within the 95% confidence interval of the observed regression as shown in Figure 3-21. The predicted temperature dependency determined at the BTI was also higher than the observed. The slope of the predicted variation slightly exceeded the 95% confidence interval for the observed slope. This difference implied that the BMP is showing less seasonal variation than would be expected based on viscosity changes. Seasonal changes in evaporation, transpiration, and biological soil activity likely played some small role in the observed seasonal variation of infiltration BMPs. However, makeshift evaporation experiments conducted during warm months in the bottom of the BTI provided no measurable or even noticeable estimates of evaporation rates.

Emerson and Traver's (2008) results show that there is a stronger temperature dependency at the BTI, and that on average the infiltration process at the BTI is faster than at the PCIB. This is not surprising due to the differences in the soil texture between the two BMPs. The composite soil and even the original soil prior to amendment at the BTI are coarser than that found at the PCIB. Coarser soil is typically associated with having a higher intrinsic permeability, and therefore will have a higher hydraulic conductivity on average and be more sensitive to changes in the fluidity of water. The estimates provided here confirm this, with the estimate of the intrinsic permeability of the BTI being approximately twice as large as the PCIB.

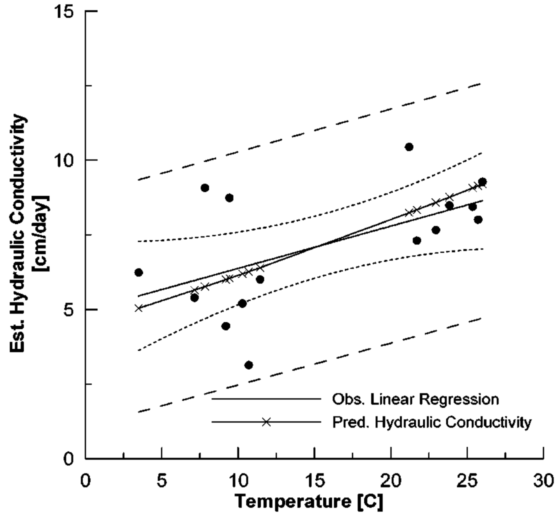


Figure 12.21: Hydraulic conductivity as a function of temperature (Emerson and Traver, 2008)

12.3.4.4 Longevity of Infiltration BMPs

The strong seasonal and cyclic variations of the infiltration BMPs make it imperative to account for temperature when determining if any long-term systematic changes are present. Visual inspection of the data did not show any obvious long-term changes. Therefore, two multiple linear regressions were performed to determine if there are any subtle systematic changes over the period of record. The regressions use both temperature and age of the BMP as the two independent/explanatory variables; therefore, only the last two years of the BTI data set were used. The multiple linear regression for the BTI provided no improvement in the r^2 over the original linear regression. Additionally the slope with respect to temperature remained essentially unchanged. The multiple linear regression provided a positive slope with respect to age of 0.79 cm/day/yr; however, the 95% confidence interval for this estimate were fairly well centered around zero (-1.4 to 2.1 cm/day/yr). Therefore, Emerson et al. (2006) found no significant evidence of a systematic change at the BTI. Based on a visual examination of the data the authors found it likely that similar results would be found if temperature data for the entire 4.25 year data record existed. While it is encouraging that the BMP showed no evidence of a systematic decrease over their relatively short data records, the true test of their longevity will come over the next decades.

12.3.4.5 Study Findings

Emerson and Traver's (2008) analysis of the continuous data record from two infiltration BMPs on the campus of Villanova University provided valuable insight into the long-term and seasonal performance of stormwater infiltration. The results showed no discernable systematic decrease in performance over the period of record examined. The steady-ponded recession rate was found to vary roughly two-fold over the course of year. The variation follows a cyclic pattern with its highest values typically occurring in late summer and the lowest in late winter. This variation is similar to that which is expected due to changes in the hydraulic conductivity resulting from temperature-induced viscosity changes of the ponded water. The seasonal changes were significant and resulted in event ponding times which varied over the course of a year between 50 to 80 hours and 80 to 120 hours on average for the PCIB and BTI, respectively.

12.4 CONCLUSIONS

Low Impact Development is predicated on achieving a water balance for runoff, infiltration, and evapotranspiration. To meet infiltration needs associated with LID, many structural practices are used including: bioretention, infiltration trenches/wells, infiltrating wetlands, level spreader – vegetated filter strip systems, permeable pavement, swales, and water harvesting systems. Each of these practices has been shown to potentially infiltrate substantial amounts of runoff. Moreover, designers can alter these practice designs to increase infiltration volumes, if desired. Some design and construction features discussed in this chapter include the incorporation of sumps, or storage wells, in the bottom of LID infiltration practices and innovative excavation methods such that underlying soil saturated hydraulic conductivity is better preserved. An important concern for any system that relies on infiltration is whether the rate of infiltration is retained with time. A study at Villanova University concluded that while there is substantial infiltration rate fluctuation seasonally, there was no statistical evidence of annual infiltration rate decreasing. An examination of several types of permeable pavement systems in North Carolina showed that no matter the type of permeable cover, runoff was substantially and significantly reduced. This allows LID designers to specify permeable pavement types based upon cost and aesthetics rather than hydrologic performance. Lastly, in addition to providing runoff volume reduction, many Low Impact Development techniques, such as simple swales examined in Maryland, provide peak runoff mitigation, albeit mostly for small to medium-sized events. All the case studies presented herein substantially reduced runoff, thereby increasing amounts of infiltration. Both of which are main goals of Low Impact Development.

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CHAPTER 13

Environmental Effects of Pervious Pavement as a Low Impact Development Installation in Urban Regions

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ABSTRACT: Pervious pavement systems can be used to reduce stormwater runoff volume and are efficient at removing solids from runoff; however, the pollutant removal efficiency for nutrients, metals, and organic contaminants is yet to be determined due to either a lack of data or inconsistent results. Groundwater recharge through the use of pervious pavement systems has not been proven, although runoff infiltration to underlying soils has been shown under certain conditions. The potential for groundwater contamination through the infiltration of runoff through pervious pavement is dependent on the stressor of interest, its mobility, its concentration in runoff, and its partitioning in runoff (dissolved or particle-bound). Every site is different and care should be taken to examine site conditions, underlying soil characteristics, and local climate prior to determining if the installation of pervious pavement would be an appropriate best management practice for stormwater management at a particular location.

13.1 INTRODUCTION

Low impact development (LID) is an alternative stormwater management strategy intended to ensure that post-development hydrologic conditions more closely match those of the pre-constructed site by implementing runoff control near the source. This differs from more traditional approaches in that stormwater is controlled at the source, rather than managing runoff through collection and conveyance offsite (U.S. EPA, 2000). There are several LID approaches used singularly or in combination to manage the stormwater including: storage, infiltration, groundwater recharge, volume reduction, runoff frequency reduction, stormwater retention/detention, reduction of impervious surfaces, and lengthening of flow paths and runoff time (U.S. EPA, 2000). Prince George's County (Maryland) publicized the overall LID approach to site design during the 1990s, but many of the designs elements have been in use for years (Maryland Department of Environmental Resources, 1993, 1999).

Stormwater controls typically termed "best management practices" (BMPs) or simply management practices are an essential element in the EPA National Pollutant Discharge Elimination System (NPDES). The NPDES system requires that non-point source water pollution be controlled for all new development and permits be given by the states. The Phase I program regulates communities with more than 100,000 people and Phase II regulates smaller municipalities. LID is one way to help ensure compliance with the NPDES program by using stormwater BMPs to decrease surface runoff. While LID practices may have become popular on merit alone, the stormwater regulations in many states accelerated the installation pace.

Pervious pavement is an infiltration-based BMP. Generally speaking, a pervious pavement system consists of several strata beginning with a pervious surface, followed by a small gravel filter course, then a reservoir or subbase course of larger gravel. A geotextile frequently separates the reservoir course gravel from the underlying soil. Figure 13.1 shows a typical pervious pavement cross-section. Not all installation designs have a geotextile at the soil interface, but it is often used to prevent soil migration into the gravel subbase. Other installations have a second geotextile at the filter course-reservoir course interface, and the geotextile located here is also used to prevent migration among gravel layers. There are many types of pervious pavements. Currently, pervious asphalt, pervious concrete, and permeable interlocking concrete paver systems appear to be the most popular. Table 13.1 presents the pervious pavement types discussed here, along with the abbreviations that will be used for the rest of this chapter.

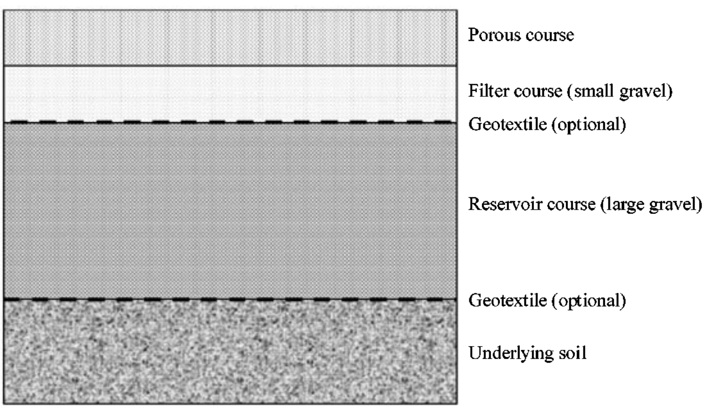

Figure 13.1: Cross-section of a typical pervious pavement system

Table 13.1: Pervious pavement types, abbreviations, and descriptions

Pavement type	**Abbreviation**	**Description**
Porous asphalt	PA	Open-graded asphalt with a reservoir for storage below
Porous asphalt friction course	PAFC	A thin layer of open-graded asphalt with no storage underneath
Pervious concrete	PC	Open graded concrete (no fines)
Permeable interlocking concrete pavers	PICP	Interlocking concrete blocks that are not permeable, but the spaces between them allow water to infiltrate
Permeable interlocking concrete pavers with a swale	PICP w/swale	PICP whose effluent stream feeds a vegetated swale
Concrete block	CB	Concrete blocks that are not interlocking, but the spaces between them allow water to infiltrate
Grassy paver	Grassy paver	Flexible plastic grid paver with sand backfill and planted with grass
Gravel paver	Gravel paver	Flexible plastic grid paver with gravel backfill
Turf paver	Turf paver	Concrete grid paver with soil backfill and planted with grass

Pervious pavement is not a new concept and both pervious asphalt and pervious concrete have been in use for several decades. The first open-graded asphalt friction course in the US was installed in 1947 near Red Bluff, CA, on US Highway 99 (Diniz 1980). Open-graded friction courses reduced road spray, thereby increasing visibility in wet conditions, and reduced hydroplaning by removing water from the road surface. Runoff rates were not investigated. In the early 1970s, the Franklin Institute constructed a pervious asphalt friction course with a pervious, less-compacted gravel subbase specifically to reduce runoff and improve runoff water quality through infiltration (Thelen et al., 1972; Field et al., 1982). By 1980, several pervious asphalt installations across the United States (DE, FL, MD, NY, PA, TX), in Australia, and in Switzerland were monitored (Diniz, 1980). Pervious concrete has been in use for more than 50 years (Brown 2003). The early pervious concrete systems were installed for drainage and traction improvements and the use of pervious concrete for environmental effects did not occur until the 1970s (Ferguson 2005). Early studies on pervious concrete infiltration rates were summarized by Wingerter and Paine (1989). Concrete paving blocks were used to rebuild European cities after World War II and designs have evolved since then (Shackel, 1997). Concrete block paving arrived relatively late to the United States due to the preferential installation of asphalt roadways. Concrete block use started in the 1970s when open-jointed blocks were developed (Shackel, 2003; Ferguson, 2005). Early studies on concrete block pervious pavement were done by William James' research group at the University of Guelph in Ontario, Canada (Kresin et al., 1996; James and Thompson, 1996; James and Shahin, 1997). A seminal study examining pervious asphalt, concrete grid pavers backfilled with sand, a gravel trench, a grass lot, conventional asphalt, and conventional concrete in Austin, Texas, compared observed data to simulated data from a pervious pavement modeling program (Goforth et al., 1983). Since these early installations, pervious pavement has grown in popularity, especially as a stormwater management control. Pervious pavements are being installed in parking lots, pedestrian walkways, driveways, overflow parking areas, and some roadways. In general, pervious pavements have not been used for roads and streets due to a concern over high traffic volumes and heavier vehicles leading to compaction of the subgrade layers, which could lead to reduced infiltration rates and shortening of the pavement lifetime (Ferguson, 2005).

The approach here is to provide a holistic assessment for pervious pavement technology from both quantitative and qualitative points of view. Qualitatively, the potential for stormwater volume reduction and groundwater recharge through the utilization of pervious pavement systems will be examined. Quantitatively, the pollutant removal capability of pervious pavement systems for solids, nutrients, metals, and organic pollutants will be presented. The variable nature of the studies led to comparison difficulties and it appeared that the best way to normalize study data was to examine pollutant removal efficiencies as percent removals compared either to conventional surface runoff or to influent values.

13.2 QUANTITATIVE ASSESSMENT

13.2.1 Volume Reduction

The main tenet of stormwater management is to reduce the runoff effects on receiving waters. From a volumetric standpoint, this can be accomplished by reducing runoff volumes and decreasing and delaying peak flows. Pervious pavement systems are infiltration-based BMPs specifically designed to reduce runoff. Success can be limited by poor site conditions (i.e., accumulation of debris and solids), lack of maintenance, and over-compaction of the system layers during construction. Pervious pavement is sensitive to surface clogging, so a proper maintenance regime must be employed to retain high infiltration rates. Careful consideration should also be given to the site soil. Successful volume reduction will require a soil with a high infiltration rate, unless a partial exfiltration pavement system is installed. Application at sites with less permeable soils is feasible if an adequate reservoir course is included or when the rain interevent period is large. Like all stormwater controls, pervious pavement systems should be designed with enough storage for the amount of runoff that will be treated and consideration needs to be given for storms exceeding these volumes. Pervious pavement systems serve two low-impact development purposes as they both reduce impervious area and infiltrate stormwater runoff.

A number of studies have shown the successful stormwater runoff volume reduction capabilities of pervious pavement systems. As early as 1980, it was reported that pervious pavements could reduce runoff between 70-80% (Gburek and Urban, 1980). Various studies have shown complete stormwater infiltration and report no runoff for grassy concrete grid pavers, plastic grid with gravel, plastic grid with grass, concrete grid with sand, pervious concrete, pervious asphalt, and PICP (Brattebo and Booth 2003, Kwiatkowski et al., 2007, Valavala et al., 2006, Bean et al., 2007a, Van Seters et al., 2007, Collins et al., 2008, Post et al., 2008). Some studies that reported zero runoff from the pervious pavement systems benefited from small storm events (Van Seters et al., 2007, and Post et al., 2008) or sandy underlying soils (Bean et al., 2007). Dreelin et al. (2006) observed 93% runoff reduction for a plastic grid grass paver system compared to the runoff volume produced by conventional asphalt during small rain events and dry antecedent conditions prior to monitoring. Other studies yielded smaller, but still substantial, reductions. PICP with a swale reduced runoff by 50% compared to that produced by conventional asphalt without a swale and removed 30% more runoff than conventional concrete with a swale and asphalt with a swale at a Florida site with average soil permeability (Rushton 2001). Pratt et al. (1995) reported average runoff reductions between 53-66% for four concrete block systems with various sub-base media. A pervious concrete partial exfiltration reactor reduced runoff 55-70% (clayey glacial till soil) (Sansalone and Teng, 2004). Bean et al. demonstrated that site maintenance is important for ensuring that infiltration rates remain high as pervious concrete infiltration rates showed mean rates as low as 16 cm/h at disturbed sites with lots of fines, while undisturbed sites had mean surface infiltration rates of 2000 cm/h (2007b).

Pervious pavement systems are often designed and installed specifically to reduce runoff. Studies have undisputedly shown that pervious pavement systems can be used to reduce stormwater runoff volumes, with reductions of 60% or more commonly seen. Runoff storage is dependent on the depth of the reservoir layer and typical rain events in specific locales and should be examined prior to pervious pavement design. Runoff was reduced most successfully for pervious pavements installed in high infiltration soils and for small rain events. Proper maintenance of pervious pavements and keeping the site free from disturbed soils will ensure that initially high infiltration rates and runoff volume reduction will continue with time.

13.2.2 Groundwater Recharge

There are two types of groundwater recharge: near-surface and deep. Near-surface groundwater recharge improves base flows of rivers and streams, while deep groundwater recharge potentially increases water level in the groundwater table. Runoff that enters pervious pavement systems can evaporate or infiltrate. Infiltrated water can then either add to the near-surface groundwater or it can percolate into the deep groundwater.

Most studies in the literature have examined near-surface groundwater recharge. Soil type is especially important for a pervious pavement system's ability to recharge near-surface groundwater. Andersen et al. (1999) reported that, for a concrete block pervious system, 55% of a 15 mm/hour 1-hour rain event could be retained in a previously dry bench-scale system with no soil underneath the storage reservoir. Legret and Colandini (1999) found that a pervious asphalt pavement system infiltrated 96.7% of stormwater volume below the pervious pavement reservoir structure in a soil with many fines (below 125 μm). In a study examining pervious block pavement, Hou et al. (2008) measured moisture depth under each pavement cell and water was found 19 mm below the cells. It took more than 10 days for the subgrade to dry out after infiltration through soil with a low permeability (2.28×10^{-4} mm/s or 0.032 in/hr) (Hou et al., 2008). The authors stress that the high infiltration rates seen in their systems with low soil permeability is most likely due to less-intense rainfall and their results should not be indiscriminately applied to other sites (Hou et al., 2008). Bean et al. (2007) reported that a PICP system stored a mean of 335 mm (13.2 inches) of runoff in the system, a concrete grid paver system backfilled with sand stored a mean of 109 mm (4.29 inches) of runoff, and a pervious concrete system retained a mean of 74 mm (2.91 inches) of runoff. All of these systems were unlined to allow for runoff infiltration into the underlying soil.

These studies imply that runoff infiltrated through pervious pavement systems may eventually reach the groundwater, although it has been difficult to prove. No studies have directly shown deep groundwater recharge by infiltration from pervious pavement systems, as this would be difficult to measure. Despite this lack of proof, there is some concern over the potential for groundwater contamination from the infiltration of stormwater runoff. The likelihood of contamination from stormwater infiltration is dependent on the specific stressor, the soil subbase, and environmental conditions. The partitioning of a particular pollutant in stormwater runoff is important for determining its transport. Many toxic organics and metals in stormwater are found in the particulate fraction and they can be removed by BMPs that rely on sedimentation and filtration principles, such as pervious pavement (Pitt et al., 1993). Some stressors that are more often found in the dissolved phase and are highly mobile in the environment are zinc, nitrates, salts (from deicing), and several PAHs (Pitt et al., 1993). Despite the high mobility of these stressors, zinc, nitrates, and PAHs are usually not found in high enough concentrations to have high groundwater contamination potential. Deicing salts, however, are highly mobile, have high abundance in stormwater during winter, and are mostly present in the dissolved phase, and these characteristics make salts recalcitrant to removal by any means and, therefore, they have a high groundwater contamination potential (Pitt et al., 1993). In order to avoid salt contamination when using pervious pavement, care should be taken to avoid the use of deicing salts, especially as icing often does not occur due to the runoff removal ability of the systems. Pervious pavement systems are not recommended for installation near contamination hotspots due to the potential for deep groundwater contamination. Examples of hotspots pertaining to pervious pavement include gas stations, vehicle salvage yards, vehicle fleet storage areas (buses, trucks, etc.), and industrial sites (Clar et al., 2004). Another way to avoid groundwater contamination is to install infiltration-based BMPs with the floor of the BMP between 2 and 4 feet above the seasonally high groundwater table elevation depending on site conditions (Clar et al., 2004).

13.3 QUALITATIVE ASSESSMENT

13.3.1 Solids

The biennial EPA water quality reports identify siltation/sedimentation as the primary cause of impairment to assessed rivers and streams with more than 100,000 stream miles adversely affected (U.S. EPA, 2007). Sedimentation can lead to reduced aquatic plant growth, loss of aquatic habitat, and drinking water quality issues. Total suspended solids (TSS) concentration in runoff is a documented stormwater pollutant and has been examined in studies around the world. Runoff TSS concentrations vary widely with reported concentrations ranging from 3-129,000 mg/L (U.S. EPA, 1983; Mudgway et al., 1997; Lee and Bang, 2000; Line et al., 2002). The National Stormwater Quality Database (NSQD), listing the analytical results of many Phase I cities, reported a median TSS concentration of 58 mg/L (n = 3390) (Maestre and Pitt, 2005). This is similar to the 101 mg/L median concentration determined in EPA's National Urban Runoff Program study (U.S. EPA, 1983).

Multiple studies of pervious pavement systems have documented the capability to remove solids. Some studies compare pervious pavement effluent to runoff from conventional pavement surfaces, while others compare influent and effluent TSS concentrations. Figure 13.2 shows the results of some of these studies. The reported effluent TSS concentrations from pervious pavement were typically 59 to 94% smaller than effluent concentration from conventional asphalt. The studies that compared influent to effluent report TSS reductions of 72 to 83%.

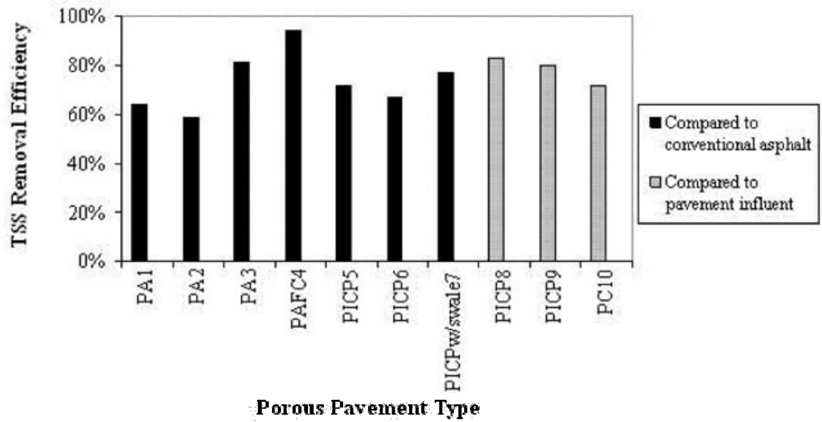

Figure 13.2: TSS removal efficiency of pervious pavement systems compared to either conventional asphalt or influent. Sources: [1]Legret et al., 1996; [2]Legret and Colandini, 1999; [3]Pagotto et al., 2000; [4]Barrett et al., 2006; [5]Bean 2005; [6]Gilbert and Clausen, 2006; [7]Rushton 2001, [8]Jayasuriya et al., 2007; [9]Nanbakhsh et al., 2007; [10]Sansalone and Teng, 2004

Many pervious pavement systems have geotextile liners separating the sub-base layer from the native soil. Studies of these pervious pavement systems often do not specify the type of geotextile installed. To examine performance differences among geotextile types, the EPA's Urban Watershed Management Branch undertook a bench-scale study. The solids removal of block paver systems was investigated using locally-collected urban stormwater runoff as the water source. The study monitored and compared three systems: those installed with a woven geotextile liner, those installed with a nonwoven geotextile liner, and those with no liner (controls). The two geotextile types were physically different from each other, with the woven having a grid-like pattern and a slippery feel, while the nonwoven was spun with no pattern. The geotextiles also had different specifications with the woven having an apparent opening size twice that of the nonwoven, while the nonwoven flow rate was 10 times that of the woven in constant head tests (ASTM, 2004). It was expected that these differences would result in different pollutant removal efficiencies. The TSS removal for both the woven and nonwoven pervious pavement systems was 89% ($R^2 > 0.96$) (Figure 13.3). The unlined systems removed 62% of solids indicating that the gravel and block paver system, with no geotextile, provides some TSS reduction. The performance of the unlined control systems was more variable than the systems with geotextiles. The presence of a geotextile in pervious block systems increased TSS removal efficiency ($p = 3.6 \times 10^{-5}$), but the two geotextile types are not statistically different ($p = 0.059$).

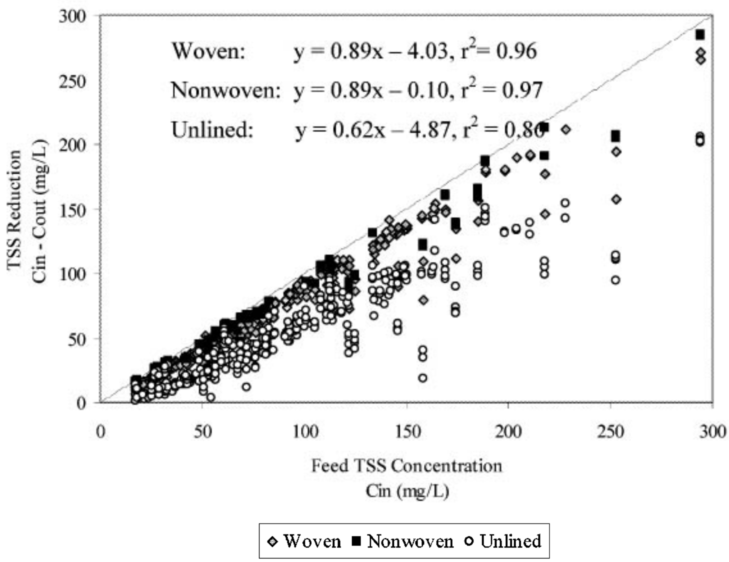

Figure 13.3: TSS concentration reductions compared to feed concentrations for woven, nonwoven, and unlined systems

The effluent flow rates from the woven and nonwoven systems were statistically different ($p = 0.035$) and both differed from the unlined systems ($p = 0.0091$, $p = 4.7 \times 10^{-8}$ woven and nonwoven, respectively). The unlined bins had the largest effluent flow rates. The nonwoven bins had, on average, larger flow rates than the bins with the woven liner, which was to be expected from the manufacturer's geotextile flow rate differences (constant head test). The experiment simulated the rainfall from an equivalent of six years of New Jersey rainfall. The effluent flow rates from the lined and unlined bins at the start of the experiments showed no statistical difference from the flows at the end of the experiment. This suggests that the systems had no measurable clogging during the experimental period.

Demonstrations have shown that pervious pavement systems reduce runoff TSS concentrations by 60% or more compared to conventional asphalt, and TSS reductions are larger (> 70%) when simply comparing influent concentrations to effluent concentrations from these systems. Solids are removed from runoff via filtration or sedimentation in the pervious pavement system and a buildup of these solids will reduce infiltration rates with time. A timely maintenance schedule of sweeping and vacuuming should be followed in order to ensure the pervious pavement remains pervious.

13.3.2 Nutrients

The EPA identified nutrients as one of the top five causes of impairment to rivers and streams in the United States (U.S. EPA, 2007). Excessive nutrient loads affect more than 15% of the impaired streams (52,000 river miles). Agricultural runoff is the primary source of stream nutrient impairment, with more than 35% of assessed streams impacted (U.S. EPA, 2007). Under normal conditions, nutrients enhance plant growth and lead to healthy ecosystems, but excessive nutrients can lead to receiving water eutrophication by stimulating algal blooms. Algal blooms destabilize dissolved oxygen levels in aquatic habitats, change the food web, and can lead to organism die-off. Phosphorous and nitrogen in their many speciated forms are the most common nutrients and sources include: fertilizers, manure, septic systems, and atmospheric deposition. Stormwater runoff often contains nutrients from these sources and can be an important pathway for nutrients to enter aquatic ecosystems (Line et al., 1996; Mudgway et al., 1997; Wu et al., 1998; O'Shea and Brosnan, 2000; Brezonik and Stadlemann 2002; Brown and Peake, 2006).

Pervious pavement systems have been investigated for their nutrient removal. Pervious pavement research often compares effluent concentrations to either influent concentrations or effluent concentrations from conventional surfaces. Figure 13.4 shows total phosphorous and total nitrogen removals for four pervious pavement systems compared to conventional asphalt. Total phosphorous removals range from 11-65% and total nitrogen removals are around 35%. The exception here is the grassy paver system that shows a 43% increase of total nitrogen compared to the conventional asphalt surface. Table 13.2 summarizes other studies that examined nutrient removals of pervious pavement systems. Total Kjeldahl Nitrogen (TKN) removals ranged from 43-91% for PICP and pervious asphalt friction courses compared to conventional asphalt. Nitrogen associated with ammonia (NH_4-N) reductions varied between 51% and 86% for several types of systems. Nitrogen associated with nitrate (NO_3^--N) reductions ranged from 32 % to 73% for pervious asphalt systems.

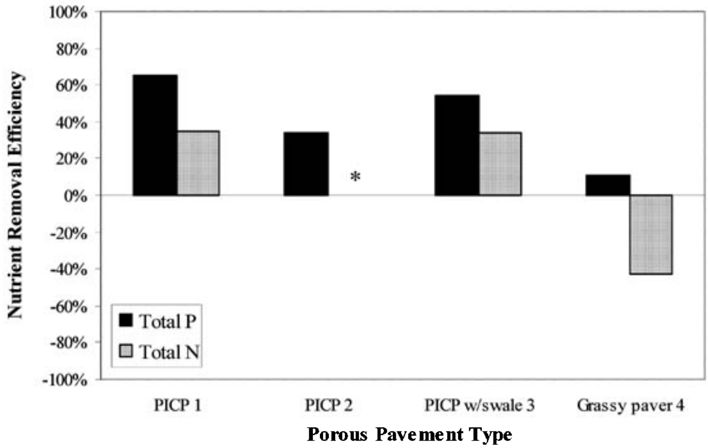

Figure 13.4: Nutrient removal efficiencies of several pervious pavement systems compared to conventional asphalt. The asterisk denotes that total nitrogen was not measured for the system. [1]Bean 2005; [2]Gilbert and Clausen, 2006; [3]Rushton 2001, [4]Dreelin et al., 2006

Table 13.2: Nutrient removal efficiencies of various pervious pavement systems

Authors	Pavement Type	Comparison	Analyte	Result
Barrett et al., 2006	PAFC	Asphalt	TKN	43% reduction
Bean 2005	PICP	Asphalt	TKN	55% reduction
Gilbert and Clausen, 2006	PICP	Asphalt	TKN	91% reduction
Rushton 2001	PICP w/ swale	Asphalt w/ swale (Asphalt w/o swale)	NH_4-N	51% reduction (83%) reduction
Bean 2005	PICP	Asphalt	NH_4-N	86% reduction
Nanbaksh et al., 2007	Block paving	Influent	NH_4-N	86% reduction
Rushton 2001	PICP w/ swale	Asphalt w/ swale (Asphalt w/o swale)	NO_3^--N	51% reduction (83%) reduction
Gilbert and Clausen, 2006	PICP	Asphalt	NO_3^--N	50% reduction

Agricultural practices have long been associated with excessive nutrient loads due to manure production and fertilizer use. About 12 million tons of nitrogen and 2 million tons of phosphorus are applied annually as commercial fertilizer, while another 7 million tons of nitrogen and 2 million tons of phosphorus are applied as manure (USGS, 1999). It would be beneficial to watershed managers and planners to have proven methods for reducing excessive nutrient loads at the source (livestock feeding areas, barns, etc.). Luck et al. (2008) conducted a study of nutrient removals comparing pervious concrete to a wire mesh screen for compost consisting of wood shavings and cow manure. A column of the compost was placed on each of the surfaces and clean water was poured through the systems for 2 days. Compared to the effluent from the wire screen, the pervious concrete removed 48% more total (organic + inorganic) nitrogen and 54% more total (organic + inorganic) phosphorous. Several inorganic nitrogen species (ammonium (NH_4^+), nitrate (NO_3^-), nitrite (NO_2^-)) were all exported by the pervious concrete systems, rather than reduced. Solids removal ranged between 92-97% for pervious concrete, with much of the compost remaining on the pervious surface. The use of pervious concrete in livestock shelters and feeding areas could reduce some nutrient concentrations in runoff from agricultural operations.

Pervious pavement systems can reduce runoff nutrient concentrations compared to influent concentrations or conventional surface runoff concentrations. There may be nutrient export, however, if fertilizers, decaying leaf matter, or grass clippings are near the pervious pavement or when the system uses plants, e.g. grassy pavers. More study on the nutrient export of pervious pavement systems is needed in order to better understand this phenomenon, as it appears that there may be a more complicated reason for higher speciated nutrient concentrations in the effluent of these systems. Pervious pavement systems seem well-suited to removing total phosphorous and total nitrogen from stormwater runoff.

13.3.3 Metals

The EPA has determined that metals are one of the top five causes of stream impairment, and more than 52,000 assessed river miles are impacted (U.S. EPA, 2007). Metals contamination in aquatic ecosystems can lead to changes in species distribution, which can upset the habitat's balance among microbes, plants, and animals (U.S. EPA, 1999). Copper (Cu), zinc (Zn), and lead (Pb) were found in more than 90% of the stormwater samples taken in the Nationwide Urban Runoff Program (U.S. EPA, 1983). The National Stormwater Quality Database reported copper in more than 85% of samples, while zinc was found in more than 95%. Lead was seen in only 77% of samples, and this decrease since 1983 may be related to the increased use of unleaded vehicle fuel (Masetre and Pitt, 2005). Metals are found in both the dissolved and particulate phase in stormwater, and those found mostly in the particulate phase generally have high removal efficiencies through BMPs that rely on settling or filtration, such as pervious pavement (Pitt et al., 1993). Copper, zinc, and lead are measured most frequently in pervious pavement studies. Figure 13.5 presents the lead, copper, and zinc removal efficiencies of a variety of pervious pavement systems compared to conventional asphalt runoff. Copper removal ranges from 35-

75%, zinc reduction percentages are between 58-88%, while lead removals are between 56-93%.

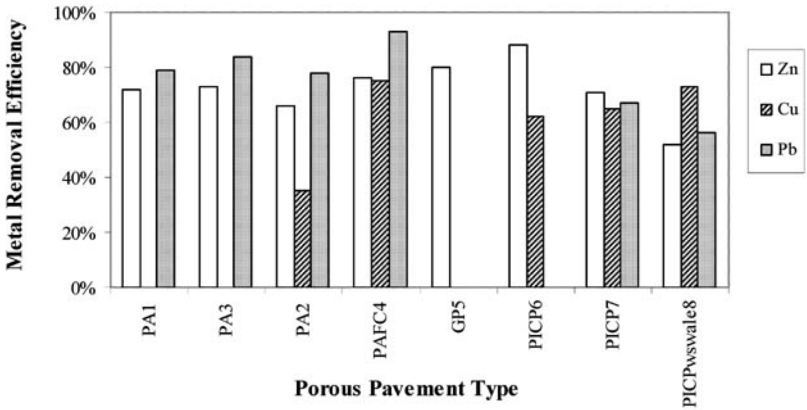

Figure 13.5: Metal removal efficiency for Zn, Cu, and Pb compared to conventional asphalt for a number of pervious pavement systems. The asterisks indicate that that metal was not measured. [1]Legret et al., 1996; [2]Legret and Colandini, 1999; [3]Pagotto et al., 2000; [4]Barrett et al., 2006; [5]Dreelin et al., 2006; [6]Bean 2005; [7]Gilbert and Clausen, 2006; [8]Rushton 2001

In studying several side-by-side pervious pavement systems, Booth and Leavitt (1999) examined the effluent water quality from these systems compared to conventional asphalt shortly after installation. A companion study was undertaken six years after the construction of the parking lot, to examine performance changes with time and use (Brattebo and Booth, 2003). The research investigated four pervious pavement systems: a flexible plastic grid planted with grass, a flexible plastic grid filled with gravel, a concrete grid planted with grass, and a concrete block system with the spaces in between filled with gravel. Figure 13.6 summarizes the results for zinc, while Figure 13.7 shows those for copper.

For zinc, removal efficiencies compared to asphalt decreased after 6 years for all systems except the concrete blocks backfilled with gravel. For copper, removal efficiencies compared to asphalt improved with time or remained at similar levels. Large differences were seen in results for the flexible plastic grid planted with grass and the concrete blocks backfilled with gravel. For samples that were below method detection limits, the graphs show 100% metal removal. Overall, it appears that the installed pavements remained durable and retained their pollutant removal capabilities for at least a six-year period. Brattebo and Booth (2003) did not discuss site maintenance practices, but noted that the site in Washington state did not have winter weather problems that are often seen at other locations (snow removal, salting, etc.).

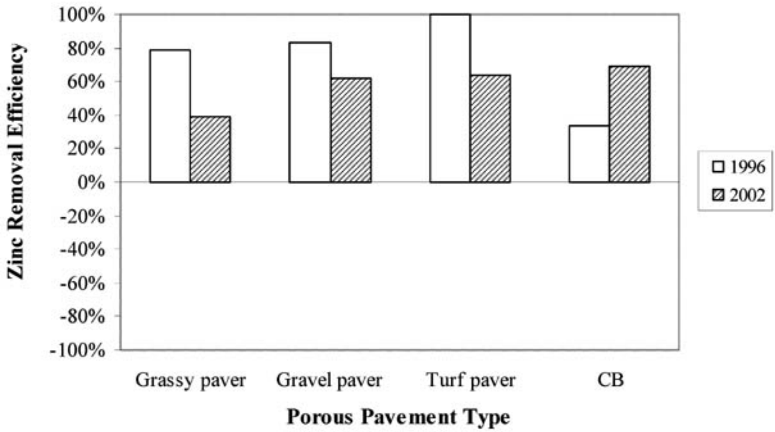

Figure 13.6: Removal efficiency of zinc by various pervious pavement systems (Booth and Leavitt, 1999; Brattebo and Booth, 2003)

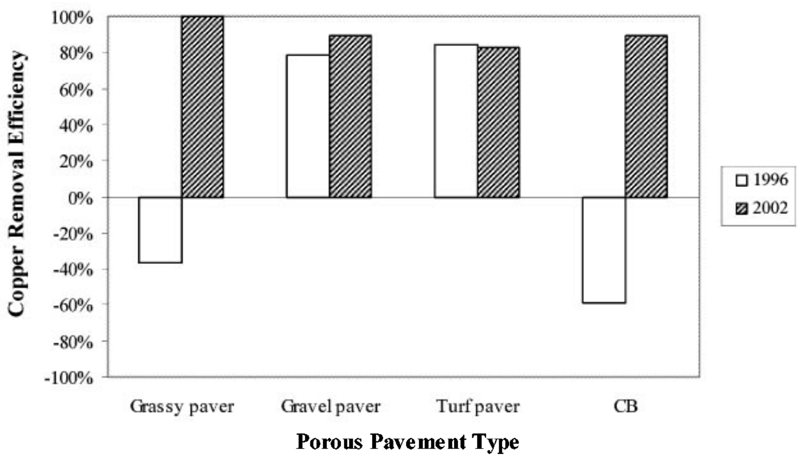

Figure 13.7: Removal efficiency of copper by various pervious pavement systems (Booth and Leavitt, 1999; Brattebo and Booth, 2003)

Sansalone and Teng (2004) examined the metal removal efficiency of a partial exfiltration reactor with a pervious concrete surface and an oxide-coated sand subbase (to augment sorption and filtration). Figure 13.8 summarizes the removal efficiencies of studied metals in the particulate and dissolved fractions, as well as totals, compared with influent concentrations. The export of magnesium (Mg) and calcium (Ca) is most likely associated with the increase in influent hardness after passing through the pervious concrete. The removal efficiency of the pervious concrete and the export of Mg and Ca through increased hardness approximately offset each other, leading to a value of about 0% for total (dissolved + particle) removals (Sansalone and Teng, 2004). Zinc and manganese (Mn) had the highest removal efficiencies, while lead had the lowest dissolved and total removal percentages, which may have been due to low influent values (Sansalone and Teng, 2004).

In general, pervious pavement systems have high capture efficiencies for zinc, copper, and lead, which are the metals most often found in stormwater runoff (U.S. EPA, 1983). These favorable removal capabilities are seen when comparing pervious pavement effluents both to influent concentrations and those found in conventional asphalt runoff.

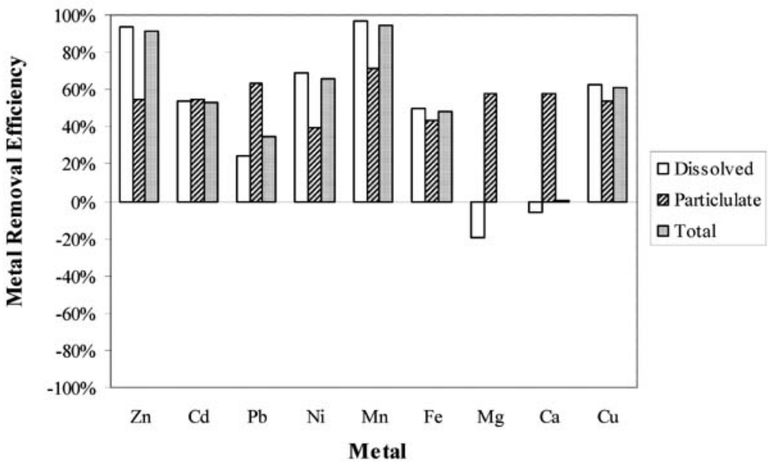

Figure 13.8: Removal efficiencies of various metals for a partial exfiltration reactor with a pervious concrete surface compared to influent (Sansalone and Teng, 2004)

13.3.4 Organic Pollutants

Certain organic compounds are known environmental pollutants. These include oils, polycyclic aromatic hydrocarbons (PAHs), polychlorinated biphenyls (PCBs), and pesticides. PCBs and pesticides are both hydrophobic, chemically-stable classes of compounds that persist in the environment. Oils and PAHs are commonly associated with automobiles (fossil fuels, incomplete combustion) and are, therefore, often found in stormwater runoff (Ngabe et al., 2000; VanMetre and Mahler, 2003; Kamalakkannan et al., 2004; Brown and Peake, 2006). According to the NSQD, the median concentration of oil and grease in stormwater runoff is 4.3 mg/L (n = 1834) (Maestre and Pitt, 2005). Despite the ubiquitous presence of organic compounds in the environment, there is a lack of data for their removal by pervious pavement systems. Of the available data, most research focuses on oils. In Renton, WA, motor oil was found in 89% of runoff samples taken coming off conventional asphalt parking spaces, but motor oil was not detected in any samples taken from 4 types of pervious pavement parking spaces (Brattebo and Booth, 2003).

There are several studies from a research group at Coventry University in the United Kingdom whose main interest is in oil degradation/removal by permeable interlocking concrete paver systems. These systems have nonwoven geotextiles installed at both the soil-reservoir interface and the filter-reservoir interface. The pavement systems remove some oil from stormwater, but microbial biofilms degrade most of the oils. The group conducted bench-scale long-term studies on the permeable interlocking concrete paving systems and found that 98% of oil added at a rate 100 times greater than that expected at an urban site was retained, even four years after the study began (Bond et al., 1999). Adding a specially designed oil trap to the PICP system (located in the sub-base layer) retained 99% more clean oil than the typical PICP system from the group's earlier studies, which were removing 98% of influent oil (Newman et al., 2004). The research group also investigated the oil removal of PICP systems compared to conventional concrete and conventional asphalt, at the laboratory scale. (Newman et al., 2006). Overall, the PICP system retained 97% of the influent oil, conventional asphalt retained 50%, and conventional concrete retained 70%. Figure 13.9 illustrates the performance of asphalt and concrete systems decreased rapidly as higher oil concentrations were added, while the oil retention of the PICP remained steady. At the higher oil concentrations (150-183 mg/L), the asphalt and concrete released nearly 75% of the influent. Microbial degradation of oil in PICP systems can be enhanced by adding phosphorous to the system. A geotextile imbedded with slow-release fertilizer beads can gradually supply the phosphorous to the system (Newman et al., 2006).

Boving et al. (2008) monitored an 800-space pervious asphalt parking lot in Rhode Island and measured total PAHs directly. Unfortunately, there was not enough data (low sample volume, etc.) to determine PAH removal efficiency for the pervious asphalt system. A single tracer study was performed on a smaller portion of the pervious asphalt with a variety of dissolved pollutants. PAHs were not examined directly, but a known degradation product of the commonly-found naphthalene (sodium-salicylate) was used as a tracer. The tracer solution was infiltrated into the pervious surface before a rain event. Less than half (47.8%) of the PAH tracer was recovered, which indicates moderate removal by the pervious asphalt system.

Several studies have indicated that pervious pavement systems may remove organic pollutants from stormwater runoff. Oil and grease removals look especially promising. Unfortunately, direct investigations of the removal capabilities of pervious pavement for specific contaminants (PAHs, PCBs, volatile organic compounds) are not available at this time.

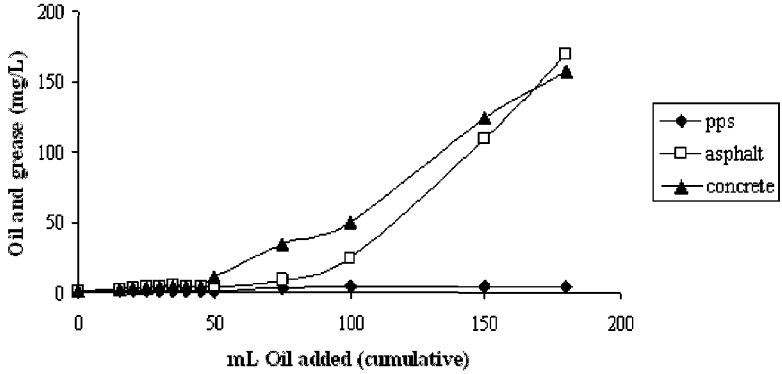

Figure 13.9: Oil and grease concentrations in effluent compared to cumulative oil volume added. Pps is pervious pavement system. Reprinted from 8th International Conference on Concrete Block Paving (2006) by Newman et al. with permission of International Concrete Pavement Institute

13.4 RECOMMENDATIONS

Careful consideration should be taken before comparing various pervious pavement studies. The results of the reviewed studies are specific to the pavement types chosen, the climatic conditions, underlying soil types, etc. When a geotextile is installed in a pervious pavement system, the specifications of that geotextile (apparent opening size, constant head flow rate, etc.) should be noted in the report so that comparisons can be made. Care should also be taken when comparing laboratory studies and large-scale, outdoor studies. Controlled conditions may yield dissimilar results compared to the unpredictable nature of the environment.

There are a variety of ways that the scope of the current literature could be improved. It has been shown that pervious pavement systems have high solids removal efficiencies, but it would be informative to see the particle-size distribution of solids in the effluent compared to the influent in order to assess the capture characteristics of these systems. Nutrient removals of pervious pavement systems should be investigated further due to the export of several nitrogen species in various studies. Chromium and arsenic are found in more than 50% of reported stormwater studies, but these contaminants are seldom monitored at pervious pavement sites (U.S. EPA, 1983). Hwang and Foster (2006, 2008) have suggested that persistent organic contaminants (PAHs, PCBs) should have high removal efficiencies in pervious systems because of high solids removal, but more direct study of organic pollutant removals by pervious pavement systems is necessary to demonstrate this hypothesis. Several studies have shown that pervious pavement systems can store runoff, but more quantified data is needed to better address the question of groundwater recharge.

In general, there is a lack of full-scale, outdoor, real-world pervious pavement studies with system replicates. More studies of pervious pavement operating in its intended use (parking lot, roadway, etc.) with climatic events, daily usage, and maintenance effects are necessary. In the hope of adding this type of study to the literature, the Urban Watershed Management Branch is currently preparing to install a full-scale 110-space pervious pavement parking lot that will be instrumented and monitored for a number of environmental stressors. This parking lot will investigate differences among side-by-side pervious asphalt, pervious concrete, and permeable interlocking concrete paver systems. There will be three parking islands, each one a different pervious pavement type, and the driveways of the lot will be conventional asphalt. The pervious pavement parking areas of the lot will have sections lined with an impermeable liner in order to collect the pervious pavement effluent and sections with a permeable geotextile liner to allow the effluent to infiltrate to the underlying soil. There will be three impermeable and three permeable sections for each pervious pavement type, which will allow for statistical analyses of collected data. The stressors that are going to be investigated include: volume, solids, pathogens, nutrients, metals, and semi-volatile organic compounds. This project is being undertaken because there has not been a definitive, long-term, multi-stressor, statistically-valid investigation on multiple pervious pavement systems at the same site.

13.5 CONCLUSIONS

Pervious pavement systems are being installed in many regions of the country with monitoring at several sites. Despite the number of projects, there is a surprisingly small amount of data on the performance of these systems in several areas. Pervious pavement systems have been shown to be effective installations for stormwater volume control and solids removal from runoff. The pollutant removal efficiency for nutrients, metals, and organic contaminants is yet to be determined due to either a lack of data or inconsistent results.

There is no clear overall method to normalize different studies, which makes conclusions difficult to draw. It is understood that percent pollutant removal efficiency is not the ideal parameter for comparing studies as important data are left out (extreme weather events, outlying analyte values, etc.), but it was a way to normalize data here. It would be most useful to designers, planners, and practitioners to have data reported as pollutant reductions in concentrations or loads and then informed decisions can be made from these data. The installation of pervious pavement as an appropriate best management practice at a specific site should be determined only after the examination of site conditions, underlying soil characteristics, and local climate.

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Notice

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CHAPTER 14

Hydrological and Environmental Modeling Analyses of Pervious Pavement Impact in a Coastal City

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ABSTRACT: Urbanization is one of the outcomes associated with economic development and population growth all over the world. However, unpleasant environmental consequences have begun to appear recently. The changes or impacts in the nexus of the terrestrial hydrological cycle in urban areas have caused significant public concern. Flooding and water logging occur frequently as the groundwater level varies continuously with time and space. The use of pervious pavements is one of the sustainable development strategies in dealing with the negative impacts of urbanization. This chapter, based on an experimental project in the pervious pavement region of a coastal city in China, uniquely discusses the hydrological and environmental effects of the pervious pavement. Given a suit of natural conditions of concern, the pro and con effects of pervious pavement in urban areas are evaluated thoroughly with a practical approach. Three modes of reclamation are simulated for a four-year time frame using the MIKE SHE® software package. It is found that stormwater infiltration through the pervious pavement raises the water head in the saturated zone, thereby reducing its salinity levels. These research findings are helpful to the decision how to use pervious pavement methods in urban regions.

14.1 INTRODUCTION

Cities, due to the development of social, economy, and culture, are centers of economy, politics, science, technology, culture, and education of their vicinity. However, with the high-speed urbanization of these areas, urban environment problems such as shortage of water resources, bad water quality, low groundwater level, lack of flow in urban rivers, degenerated ecological condition and etcetera appear gradually. The main reason of these phenomena is the increase of urban impervious surfaces. (Booth and Jackson, 1997)

The main materials used for ordinary urban pavement are impervious bitumen, concrete, and brick, etc. Their infiltration capability is virtually zero. The performance of these types of land surface greatly damage the process of the natural hydrological cycle, and exclude the subsoil from rainfall-infiltration process. In addition, the ordinary pavement increases the velocity of overland flow and reduces the time of concentration. (Teng and Sansalone, 2004) The low impact development (LID) approach has been recommended as an alternative to traditional stormwater design. Pervious pavements, a LID practices, have been researched in recent years (Dietz, 2007). Due to the especial infiltration capacity and water retention, pervious pavements recover the healthy urban hydrological cycle, particularly the processes of rainfall-infiltration, runoff, and evaporation (Dreelin and Fowler, 2006; Bean and Hunt, 2007; Kuang and Sansalone, 2005; Asaeda and Ca, 2000 etc.).

Furthermore, pervious pavement brings other environmental benefits, such as resistance to seawater intrusion in coastal cities. Figure 14.1 shows the image of interaction between fresh water and salt water in coastal regions with pervious pavement. In order to recover the environmental condition of a coastal area, it is essential to use fresh water to push sea water out. The more fresh water is accumulated, the better the environmental condition will become. Naturally, stormwater is the main water resource and pervious pavement is a useful measure to support the process.

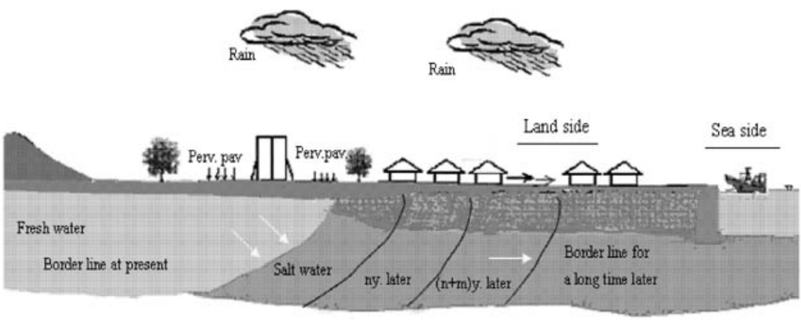

Figure 14.1: Variation tendency of the border line between fresh and salt water

14.2 MODEL DEVELOPMENT

The calculation model of pervious pavements should include two parts. One is the model of pervious pavement cells and the other is the model of resistance of seawater intrusion. The following sections show the introductory detail and analysis of both models.

14.2.1 The Model of Pervious Pavement Cells

The hydrological cycle in an urban area with pervious pavement is different from a natural one. Figure 14.2 shows the structure and the hydrological process in two layers: one for pervious pavement and the other for subsoil. The pervious pavement, with big infiltration capacity and large water retention, drives more stormwater to infiltrate into the subsoil. The hydrological effect of pervious pavement is analyzed, based on Horton theory.

The nomenclature with the Figure 14.2 and the analysis is as follows: t is the time [T], i is the rainfall intensity [L/T], W_m is the saturated water volume [L], ΔW --the change of water volume [L], f_p -- infiltration capacity of the pervious pavement [L/T], f_s -- infiltration capacity of the subsoil [L/T], r_{sub} --interflow intensity [L/T].

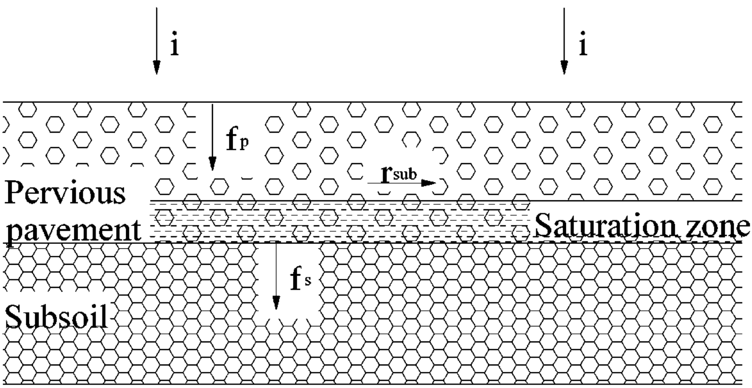

Figure 14.2: The image of hydrological process in two layer structure

Let's take an event of rainfall occurring in an urban region with pervious pavement as an example. Suppose that prophase precipitation is not large and the water content of the entire aeration zone (including pervious pavement) is less than field capacity (i.e., $W_0 < W_f$, W_0 being the initial water content of the pervious pavement and W_f being the water content when the pervious pavement reaches field capacity). At the beginning of rainfall (t_0) , the infiltration capacity of the pervious pavement is so strong (NDRC, 2005) that it is unlikely that the rainfall intensity exceeds the steady infiltration capacity of the pervious pavement ($i < f_p$) . So the probability of producing overland flow is little ($R_s \to 0$). Because of the low initial water content, it is not possible to reach the upper limit of water capacity of the pervious pavement in a short period of time. Therefore, the water amount infiltrating into the subsoil is approximately zero in this period. The increment of water volume of the pervious pavement is $\Delta W = \int_{t_0}^{t} i(t)dt$.

With the rain lasting, precipitation augments gradually. The accumulative water infiltrating through the surface of pervious pavement exceeds its water capacity ($W_0 + \Delta W \geq W_f$). In this period, the water begins to infiltrate into subsoil. Because of the great difference of infiltration capacity between pervious pavement and subsoil

($f_p >> f_s$), there are obvious changes of infiltration. If the rainfall intensity is less than or equal to the infiltration capacity of the subsoil ($i \le f_s$), all water going through pervious pavement will continually infiltrate into subsoil. Practical infiltration rate is equal to rainfall intensity ($f_a = i$). The infiltration water amount is $\int_{t_1}^{t} i(t)dt$. If the rainfall intensity exceeds the infiltration rate of subsoil ($i > f_s$), water will cumulate on the interfaces above subsoil and form a temporary saturation zone. Accumulation of water is $\int_{t_1}^{t} [i(t) - f_s(t)]dt$, the water amount infiltrated into the subsoil is $\int_{t_1}^{t} f_s\,dt$. If the rainfall intensity is continuously larger than the infiltration rate of the subsoil, water will accumulate at the saturation zone ($i > f_s + r_p$), and interflow in the pervious pavement (R_{sub}, amount of the interflow) will come into being. During the rainfall process, it is possible that rainfall intensity is larger than the summation of subsoil infiltration capacity and interflow runoff capacity ($i > f_s + r_{sub}$), because the infiltration capacity of the subsoil is descending. As a result, the thickness of temporary saturation zone on the interface (the thickness of interflow) increases. In the condition of enough rainfall intensity and rainfall amount, saturation zone can ultimately reach the surface of ground ($\int_{t_1}^{t} i(t)dt - \int_{t_1}^{t} f(t)dt - \int_{t_1}^{t} r_{sub}\,dt > W_m - W_f$). Then the saturation surface runoff is produced. The analysis of infiltration and runoff process of the pervious pavement above is based on the circumstances that rainfall intensity increases gradually, so that a basic theoretical calculation model for this hydrological effect can be built. Figure 14.3 is the detailed illustration of this model. Every step is corresponding with the change of the rainfall infiltration condition and the change of water amount. (*I*--infiltration amount of subsoil, *P*--amount of rainfall).

$$W_0 < W_f, t = t_0, i = 0$$

$$\downarrow$$

$$\Delta W_p = \int_{t_0}^{t} i(t)dt, t > t_0, i < f_p$$

$$\downarrow$$

$$\Delta W_p = \int_{t_0}^{t} f_p(t)dt, R = P - I_0 - E, t > t_0, i > f_p$$

$$\downarrow$$

$$W_f = W_0 + \Delta W, t = t_1$$

$$\downarrow$$

$$I_1 = \int_{t_1}^{t} i(t)dt, t > t_1, i < f_s$$

$$\downarrow$$

$$I_2 = \int_{t_1}^{t} f_s(t)dt, R_{sub} = \int_{t_1}^{t} [i(t) - f_s(t)]dt, t > t_1, i > f_s$$

$$\downarrow$$

$$W_m - W_f = \int_{t_1}^{t_2} i(t)dt - \int_{t_1}^{t_2} f(t)dt - \int_{t_1}^{t_2} r_{sub}dt, t = t_2, i > f_s + r_{sub}$$

$$\downarrow$$

$$I_3 = \int_{t_2}^{t} f(t)dt, t > t_2, i > f_s + r_{sub}$$

Figure 14.3: Flowchart of Pervious Pavement Cell Simulation Model

14.2.2 Modeling the Resistance to Seawater Intrusion

In aquifers of coastal areas, fresh water and sea water maintain an approximate static balance and form a comparatively stable interface (transition zone) under natural condition. However, with the reduction of ground water caused by the urbanization in coastal zones, the water head of fresh water is reduced, and the balance is broken. As a result, the interface moves from sea side to land side, and the seawater intrusion occurs. From previous section, it is known the large area of pervious pavement in coastal cities plays a favorable role in increasing the infiltration of stormwater. With the stormwater recharging into the groundwater, the head of groundwater increased. As a result of continuous rainfall, freshwater pushes the interface back from landside to sea side gradually. Of course, it is foreseeable that the interface likely varies with the alternative of rain and dry seasons.

From above, the interface is actually the focus of the issue. In fact, the sea water and fresh water are easily mixed, so under the effect of hydraulics dispersion, they form a saturated transitional zone with the salinity from high to low. The transition zone is sometimes only a few meters to tens of meters wide, and at times the width could reach several kilometers. Therefore, based on the relationship between the scale of the research region and the breadth of transitional zone, as well as the need for studying this problem, the transitional zone can be regarded as immiscibility mutation interface or miscible convection - diffusion interface. Therefore two different models are introduced in the followings.

The Model of Immiscibility Mutation Interface

Pinder (1976), Bear (1979) and Xue (2007) have studied the calculation model of seawater intrusion with immiscibility mutation interface hypothesis. The main equations of the model are shown as followings. Continuity equation of freshwater section and seawater section of both sides of the interface respectively are:

In the freshwater section,

$$\nabla \cdot v_f + S_s^f \frac{\partial H_f}{\partial t} = 0 \tag{1a}$$

$$v_f = -K_f \cdot \nabla H_f \tag{1b}$$

In the seawater section,

$$\nabla \cdot v_s + S_s^s \frac{\partial H_s}{\partial t} = 0 \tag{2a}$$

$$v_s = -K_s \cdot \nabla H_s \tag{2b}$$

Where K_f and K_s, S_s^f and S_s^s, H_f and H_s, v_f and v_s respectively are hydraulic conductivity, specific storage, water head, seepage velocity, respectively of freshwater section and seawater section. The water heads are given as follows:

$$H_f = Z + \frac{p}{\gamma_f}, \quad H_s = Z + \frac{p}{\gamma_s} \tag{3}$$

Where γ_f, γ_s respectively are specific weight of freshwater and seawater, respectively.

Supposed the equation of the break interface between freshwater and seawater is

$$F(x, y, z, t) = 0 \tag{4}$$

At the interface of z = b, the pressure must be continuous resulting in the following equations

$$\gamma_f (H_f - b) = \gamma_s (H_s - b) \tag{5}$$

$$b(x,y,t)=\frac{\gamma_s H_s-\gamma_f H_f}{\gamma_s-\gamma_f}=(1+\delta)H_s-\delta H_f \tag{6}$$

$$\delta=\frac{\gamma_f}{\gamma_s-\gamma_f} \tag{7}$$

Combining equations (1) (2), if H_f and H_s can be obtained, the position of the interface can be known by equation (6). Moving speed of the interface can be obtained by differentiating (6) with respect to t. The equation of interface is

$$F(x,y,z,t)\equiv z-b(x,y,t)=0 \tag{8}$$

Due to the fact that the corporate boundary (namely interface) of freshwater section and seawater section is unknown, equations (1) and (2) are not a closed system. The location of the interface must be coupled with Equations (1) and (2).

The interface is a substance face and consists of ilk water molecule. Relative to the interface, water molecule can be considered as immovable, so water dynamical differential coefficient can be expressed by equation (9).

$$\frac{DF}{Dt}=\frac{\partial F}{\partial t}+u\cdot\nabla F=0 \tag{9}$$

Where, $u=(u_x,u_y,u_z)$ is the surface speed of the substance. u_s is for seawater particle, u_f is for freshwater particle, they are expressed by equation (10a) (10b), respectively.

$$\frac{\partial F}{\partial t}+u_s\cdot\nabla F=\frac{\partial F}{\partial t}+\frac{v_s}{\phi}\cdot\nabla F=0 \tag{10a}$$

$$\frac{\partial F}{\partial t}+u_f\cdot\nabla F=\frac{\partial F}{\partial t}+\frac{v_f}{\phi}\cdot\nabla F=0 \tag{10b}$$

Where, ϕ is the effective porosity. $\frac{\partial F}{\partial t}=-\frac{\partial b}{\partial t}$ be concluded by equation (6). Substitution $\frac{\partial F}{\partial t}=-\frac{\partial b}{\partial t}$ into equation (10a) (10b), then

$$\phi\frac{\partial b}{\partial t}=v_s\,|_{z=b}\cdot\nabla(Z-b)=v_{sz}\,|_{z=b}-\left[v_{sx}\frac{\partial b}{\partial x}+v_{sy}\frac{\partial b}{\partial y}\right]_{z=b} \tag{11a}$$

$$\phi \frac{\partial b}{\partial t} = v_f \,|_{z=b} \cdot \nabla (Z - b) = v_{fz} \,|_{z=b} - \left[v_{fx} \frac{\partial b}{\partial x} + v_{fy} \frac{\partial b}{\partial y} \right]_{z=b} \tag{11b}$$

The Model of Miscible Convection - diffusion

Due to the changing of solution concentration, liquid density will alter, which has an effect on the velocity field. So the water-flow equation under such circumstances differs from the traditional flow equation. Huyakorn, etc. (1987) and Xue Yuqun (2007) have conducted systemic research. Ignoring local acceleration, the motion equation of heterogeneous fluid in anisotropic medium is

$$v_i = -\frac{k_{i,j}}{\mu} \left(\frac{\partial p}{\partial x_j} + \rho g \frac{\partial x_3}{\partial x_j} \right) = -\frac{k_{i,j}}{\mu} \left(\frac{\partial p}{\partial x_j} + \rho g e_j \right) \tag{12}$$

Where, v_i is the seepage velocity; $k_{i,j}$ is the permeability tensor; μ is the dynamic viscosity coefficient; ρ is the liquid density; g is the acceleration of gravity; e_j is the j component of unit vector of gravity orientation ($e_1 = e_2 = 0$, $e_3 = 1$).

Continuity equation of seepage is

$$\left[-\frac{\partial}{\partial x_i} (\rho v_i) + \rho q \right] \Delta x_1 \Delta x_2 \Delta x_3 = \frac{\partial}{\partial t} (\rho \phi \Delta x_1 \Delta x_2 \Delta x_3) \quad (i = 1,2,3) \tag{13}$$

Where, Δx_1 , Δx_2 and Δx_3 are the length of the sides of the infinitesimal equilibrium unit body (parallelepiped), each side of the parallelepiped is parallel to the coordinate; ϕ is the effective porosity; q is the flux of unit volume of porous medium sources (or sinks). If one pump well is located at $P(x_0)$ and its flux is q, q will become $q\delta(x - x_0)$, δ is Delta function. Substituting (12) into (13), we obtain

$$\frac{\partial}{\partial x_i} \left[\frac{\rho k_{i,j}}{\mu} \left(\frac{\partial p}{\partial x_j} + \rho g e_i \right) \right] \Delta x_1 \Delta x_2 \Delta x_3 = \frac{\partial}{\partial t} (\rho \phi \Delta x_1 \Delta x_2 \Delta x_3) - \rho q \Delta x_1 \Delta x_2 \Delta x_3 \tag{14}$$

Consider $\rho = \rho(p, c)$, c is the concentration of certain component of liquid. Suppose lateral compression and lateral expansion of aquifer are restricted, Δx_1 and Δx_2 are constant. Considering vertical deformation only, expanding right part of equation (14), we obtain

$$\frac{\partial}{\partial t}\left(\rho\phi\Delta x_1\Delta x_2\Delta x_3\right)=\left[\phi\rho\frac{\partial\left(\Delta x_3\right)}{\partial t}+\left(\rho\Delta x_3\frac{\partial\phi}{\partial p}+\phi\Delta x_3\frac{\partial\rho}{\partial p}\right)\frac{\partial p}{\partial t}+\phi\Delta x_3\frac{\partial\rho}{\partial c}\frac{\partial c}{\partial t}\right]\Delta x_1\Delta x_2$$

Substituting related formula of ground water dynamics

$$d\left(\Delta x_3\right)=\Delta x_3\alpha dp,$$

$$\frac{\partial\phi}{\partial p}=(1-\phi)\alpha,$$

$$\frac{\partial\rho}{\partial p}=\rho\beta$$

And the definition of water holding capacity $S_S=\rho g(\alpha+\phi\beta)$ into above equation, we obtain

$$\frac{\partial}{\partial t}(\rho\phi\Delta x_1\Delta x_2\Delta x_3)=\left\{\phi\beta\Delta x_3\alpha\frac{\partial p}{\partial t}+\left[\rho\Delta x_3\left(1-\phi\right)\alpha+\phi\Delta x_3\rho\beta\right]\frac{\partial p}{\partial t}+\phi\Delta x_3\frac{\partial\rho}{\partial c}\frac{\partial c}{\partial t}\right\}\Delta x_1\Delta x_2$$
$$=\left(\frac{S_s}{g}\frac{\partial p}{\partial t}+\phi\frac{\partial\rho}{\partial c}\frac{\partial c}{\partial t}\right)\Delta x_1\Delta x_2\Delta x_3 \qquad (15)$$

Where, α is the volume compressibility of porous medium; β is the volume compressibility of liquid. Substituting equation (15) into equation (14), and considering that the volume of unit body $\Delta x_1\Delta x_2\Delta x_3$ is infinitesimal, $\Delta x_1\Delta x_2\Delta x_3$ can be divided out of the equation. So

$$\frac{\partial}{\partial x_i}\left[\frac{\rho k_{i,j}}{\mu}\left(\frac{\partial p}{\partial x_j}+\rho g e_i\right)\right]=\frac{S_s}{g}\frac{\partial p}{\partial t}+\phi\frac{\partial\rho}{\partial c}\frac{\partial c}{\partial t}-\rho q \qquad (16)$$

In the range of concentration changing when sea water and fresh water mix, μ, which is changed very little, can be regarded as a constant, and is equal to viscosity coefficient of fresh water μ_0; considering that the density ρ exhibits linear change mainly along with concentration in the range of above, so we can suppose ρ is a linear function of concentration (Huyakorn, 1987), viz.

$$\rho=\rho_0\left(1+\varepsilon\frac{c}{c_s}\right) \qquad (17)$$

Where, ρ_0 is the reference density, namely freshwater density; c_s is the component concentration corresponding to the maximal density ρ_s; density difference rate is

$$\varepsilon = \frac{\rho_s - \rho_0}{\rho_0}$$

So,

$$\frac{\partial \rho}{\partial c} = \rho_0 \frac{\varepsilon}{c_s} = \rho_0 \eta \tag{18}$$

Where, $\eta = \varepsilon / c_s$ is the density coupling coefficient. Reference head, viz. freshwater head, is defined as

$$H = \frac{p}{\rho_0 g} + x_3 \tag{19}$$

Then,

$$\frac{\partial p}{\partial x_j} = \rho_0 g \frac{\partial H}{\partial x_j} - \rho_0 g e_j \tag{20}$$

$$\frac{\partial p}{\partial t} = \rho_0 g \frac{\partial H}{\partial t} \tag{21}$$

Substituting (18) 、 (20) 、 (21) into (16), we can obtain

$$\frac{\partial}{\partial x_i}\left\{\frac{\rho k_{i,j}}{\mu}\rho_0 g\left[\frac{\partial H}{\partial x_j}+\left(\frac{\rho}{\rho_0}-1\right)e_j\right]\right\} = \rho_0\left(S_s\frac{\partial H}{\partial t}+\phi\eta\frac{\partial c}{\partial t}-\frac{\rho}{\rho_0}q\right)$$

Then substituting (17) into above equation, define the relation of permeability coefficient tensor $K_{i,j}$ and permeability tensor $k_{i,j}$ as

$$K_{i,j} = \frac{\rho g k_{i,j}}{\mu} \tag{22}$$

Divided out ρ_0, we can obtain the flow equation (21) for describing the liquid of continuous changing density.

$$\frac{\partial}{\partial x_i}\left[K_{i,j}\left(\frac{\partial H}{\partial x_j}+\eta c e_j\right)\right]=S_s\frac{\partial H}{\partial t}+\phi\eta\frac{\partial c}{\partial t}-\frac{\rho}{\rho_0}q \tag{23}$$

14.3 CASE STUDY

14.3.1 Infiltration through Pervious Pavement

Condition and Basic Data

Dalian is located in northeast China and on the northwest shore of Pacific, which is surrounded by the seas on three sides of east, west and south. Dalian belongs to the East Asia monsoon climatic zone and a variety of temperature and drought is evident in four seasons. The precipitation is uneven for each year. The annual average is 643mm. Most of the precipitation in a year occurs from June to September, which accounts for about 80% of the total annual precipitation. Except for evaporation, most of the rainfall enters into the sea in the form of surface runoff. Owing to the severe water shortage in Dalian, the collection and utilization of rain water make an obvious profit on society, environment, and economy.

Dalian has a character of continental coast climate. Rainfall centralizes mainly in summer and short-time storm occurs frequently in rainy seasons. Figure 14.4 shows the rainfall intensity recorded in June, 2005. It lasted 75min with 29.4mm precipitation. Taking this rain as an example, the proposed model was used to analyze the hydrological effect of pervious pavement cell. The property of the pervious pavement is shown in Table 14.1. Infiltration capacity curve of pervious pavement is based on Horton infiltration curve $f_p(t)=f_c+(f_0-f_c)\times e^{-\beta t}$ (f_0 =0.4mm/min, f_c=0.1mm/min, β=0.060min^{-1}). Infiltration Capacity Curve of subsoil is showed in Figure 14.5. Incidentally, the runoff from the surrounding impervious district into the pervious pavement district is assumed to be zero.

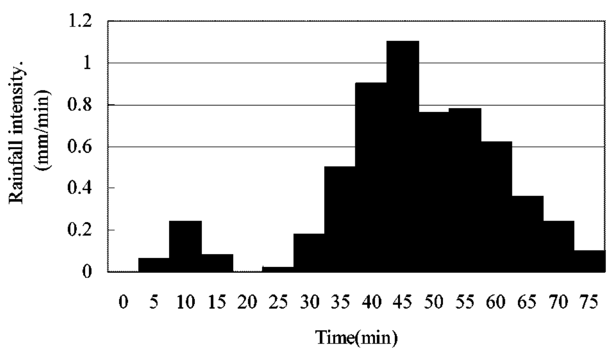

Figure 14.4: Rainfall process

Table 14.1: Property of pervious pavement

	f_p (mm/min)	W_0 (mm)	W_f (mm)	W_m(mm)	H_p(mm)
Pervious pavement	6	2	6	50	150

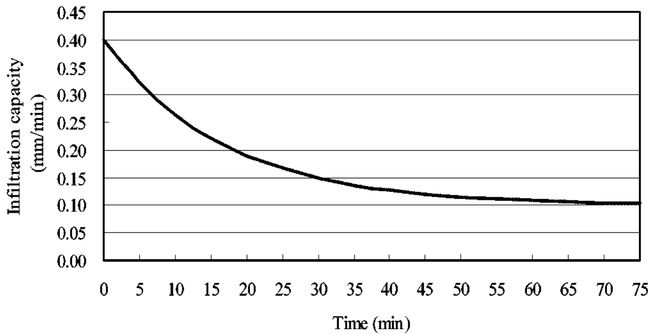

Figure 14.5: Infiltration capacity curve

Result and Analysis

In the process of rainfall, 4mm precipitation complemented the water holding capacity of pervious pavement, and the process lasted 32.7 minutes. From then on, rain water began to infiltrate into the subsoil. At the end of the rain, the amount of infiltration into subsoil is 8.9 mm and the amount storing in the pervious pavement reached 16.5 mm. During another 161.2 minutes' infiltration, almost all saturation water held in the pervious pavement has infiltrated into the subsoil. In the whole simulation period, surface runoff did not happen. The result shows that pervious pavement woks well. The infiltration rate curve for the whole rainfall period is shown in Figure 14.6.

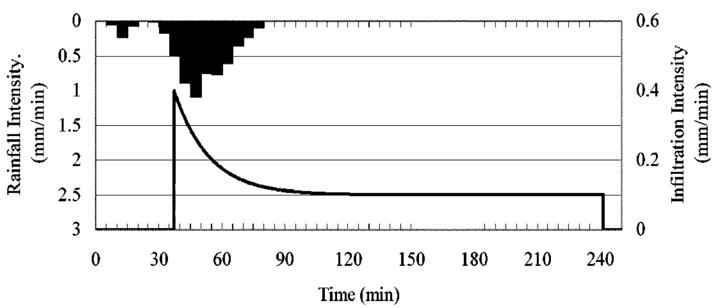

Figure 14.6: Infiltration intensity of subsoil

Due to the short duration of rainfall, the evaporation is not considered. Based on the investigation of the red soil (Wang and Chen, 2007), under 2mm/min rainfall intensity, subsurface flow rate is about 10-5mm/min. The total amount of subsurface flow is so small and the delay time is so long that it is not considered in the calculation. Through the calculation and analysis above, an interesting phenomenon is found. During the rain, pervious pavement held up some rain water, which does not infiltrate into the subsoil immediately as the infiltration ability of subsoil is not large enough. When rainfall stopped, the infiltration process lasted about 160minutes, and almost all water held in the pervious pavement (except evaporation) infiltrated into the subsoil. Therefore, it is important that there is enough water storage capacity of the pervious pavement. As a result, some types of pervious pavement are built with thick underlayer in order to obtain more water retention capacity. In addition, when the lower subsurface flow intensity blocks the transfer of water, penetration pipes are installed in pervious pavement. This measure can make the subsurface flow intensity larger and the infiltration of water held in pervious pavements faster.

14.3.2 Resistance to Seawater Intrusion

A MIKE SHE computer model was applied for an area in the City of Dalian, China. This is a reclaimed land with the information of topography, rainfall, evapotranspiration, etc., available.

Introduction of Research Region

Dalian tariff free zone with 5 km^2 reclaimed land was selected as a research region. East of the region is a freight port with pile foundation structure. The impact of the foundation to the groundwater exchange is negligible. Figure 14.7 depicts the location and the map of the region.

About MIKE SHE

MIKE SHE developed by Denmark hydraulic Institute（DHI）is a distributed hydrological model. The function of it for describing hydrological process and water quality is robust. It has been widely used in the engineering practices. MIKE SHE consists of many modules with different functions for different hydrological process. The modules of overland flow (OL), unsaturated flow (UZ), evapotranspiration (ET), saturated flow (SZ) and advection-dispersion (AD) are used in this study. Following is the main equations used in the model. (DHI, 2006)

The governing equation for the three-dimensional saturated flow in a saturated porous media is

$$\frac{\partial}{\partial x}\left(K_{xx}\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_{yy}\frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(K_{zz}\frac{\partial h}{\partial z}\right)-Q=S\frac{\partial h}{\partial t} \tag{24}$$

Figure 14.7 The location and the map of research region

where K_{xx}, K_{yy}, K_{zz} are the hydraulic conductivity along the x, y and z axes of the model, which are assumed to be parallel to the principle axes of hydraulic conductivity tensor, h is the hydraulic head, Q represents the source/sink terms, and S_s is the specific storage coefficient.

The transport of solutes in the saturated zone is governed by the advection-dispersion equation, which for a porous medium with uniform porosity is

$$\frac{\partial c}{\partial t} = -\frac{\partial}{\partial x_i}(cv_i) + \frac{\partial}{\partial x_i}\left(D_{ij}\frac{\partial c}{\partial x_j}\right) + R_c \quad i,j = 1,2,3 \tag{25}$$

Where c is the concentration of the solute, R_c is the sum of the sources and sinks, D_{ij} is the dispersion coefficient tensor and v_i is the velocity tensor.

The advection transport is determined by the water fluxes (Darcy velocities) dynamically calculated during the simulation. To determine the groundwater velocity, the Darcy velocity is divided by the effective porosity.

$$v_i = \frac{q_i}{\theta} \tag{26}$$

Where q_i is the Darcy velocity vector and θ is the effective porosity of the medium..

Theoretical Model and Computer Model

Theoretical analytical model of pervious pavement cell is established by the module of unsaturated zone (UZ) of MIKE SHE. Pervious pavement is defined with the soil profile definition tool of UZ. The Modules of saturated flow are used to set up model of resistance seawater intrusion. The simulation of miscible convection-diffusion is carried out by the Modules of advection-dispersion. Figure 8 is the flowchart of this calculation procedure.

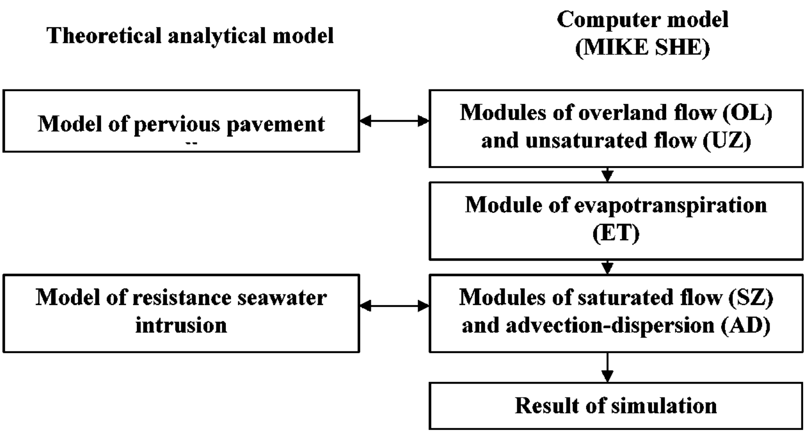

Figure 14.8 Flowchart of calculation procedure

Setting of Model Condition and Adjusting of Simulation Process

Most of the parameters, such as initial condition, rainfall, evaporation, etc., were directly selected based on field observation conducted from January 1, 2002 to December 31, 2005. The other parameters are set as following. This analysis has grid size of 100 m × 100 m and time step of 1-day. The initial head elevation is 1m, and the initial salinity in SZ (saturated zone) is equal to seawater. Three simulations of different states of ground surface - pervious state, paved state, and natural state, are used to compare the water movement and quality.

Result and Analysis

Water head elevation and water quality in saturated zone of three cases with obviously different settings are simulated by MIKE SHE. The findings show that: 1) Head elevation, nature case > pervious case > paved case; and 2) Salinity, paved case > pervious case > nature case. In the pervious case, after 60 time steps adjustment, the salinity of the model comes into the stable state. Then, with the rainfall infiltration process, the water head elevation rises and the salinity reduces gradually. In the paved case with similar adjusted process as above, the water level of groundwater rises slightly; the salinity of groundwater is a little bit lower than the initial value. However, as a result of small rainfall supply to groundwater, the water head of grids close to sea side reduces. Figure 14.9 shows the simulated water head elevation in SZ at the end of every year with the ground surface condition of pervious state. Points A and B are two check elements which is 2,500m and 400m far from the coastline respectively.

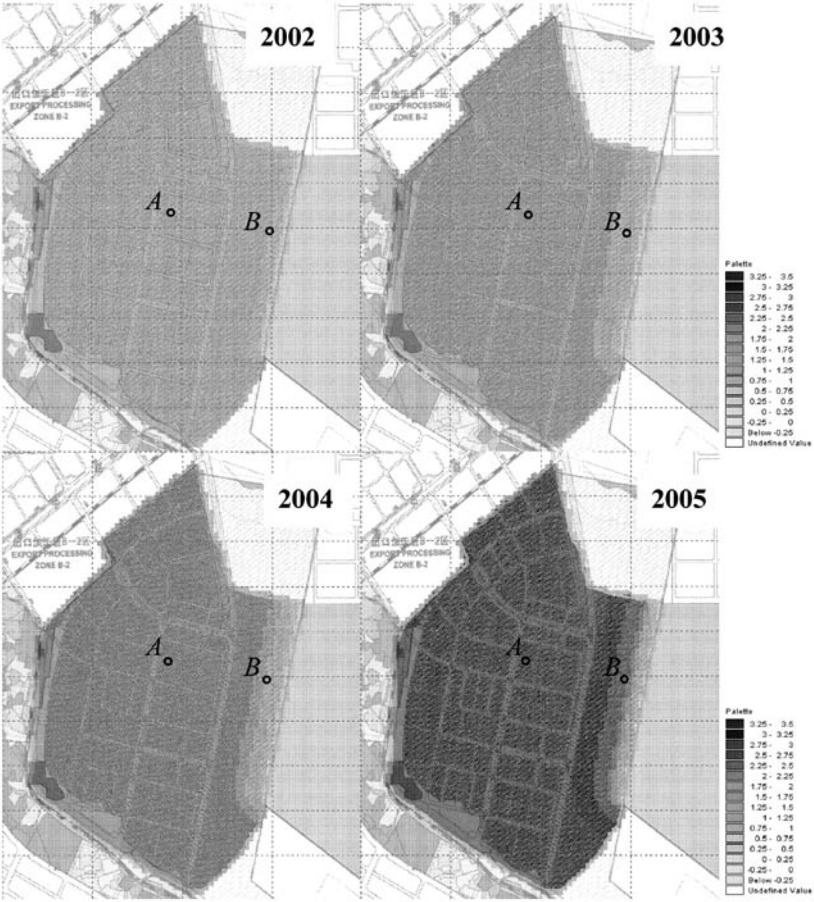

Figure 14.9: Head elevation in SZ at the end of every year in simulation

The influences of rainfall infiltration at the points A and B are different. Figure 14.10 shows the process of water head elevation at point A and point B from January 1, 2002 to December 31, 2005 respectively. The three lines are water head elevation processes for the ground surface conditions of pervious state, paved state and natural state. The results indicate that the groundwater level rises gradually at both point A and B. However, and the raising range of groundwater level at point A is larger than point B. This is corresponding with the flow tendency of fresh water from land side to sea side.

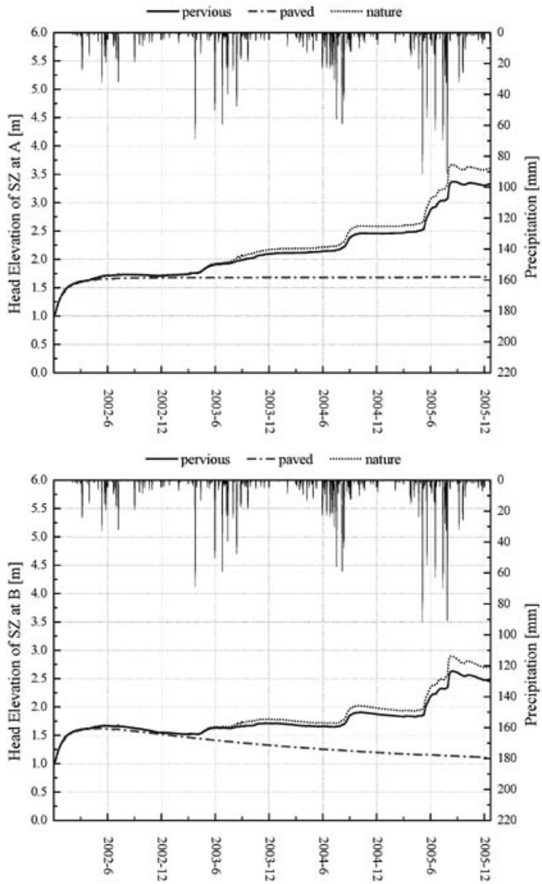

Figure 14.10: Water Head elevation processes at Points A and B in SZ.

In the pervious case, through every rainy season, salinity will decline substantially; the grids near coastline will be a concentration rebound after each decline process, which is brought by the diffusion of seawater. As the distance from the coastline increases, the rebound becomes inconspicuous until no rebound. It can be inferred that the salinity of the grids far away from the coastline will be further reduced as a result of the durative infiltration of rainfall, making the seawater intrusion recede gradually.

In the paved case with similar adjusted process as above, the salinity of groundwater is a little bit lower than the initial value. For the grids away from the coastline, a little change is observed during the simulated time. But the tendency shows that the seawater invasion will continue to extend to the side of the continent.

Figure 14.11 shows the simulation results of salinity in SZ at every end of the year. The ground surface condition is the pervious state.

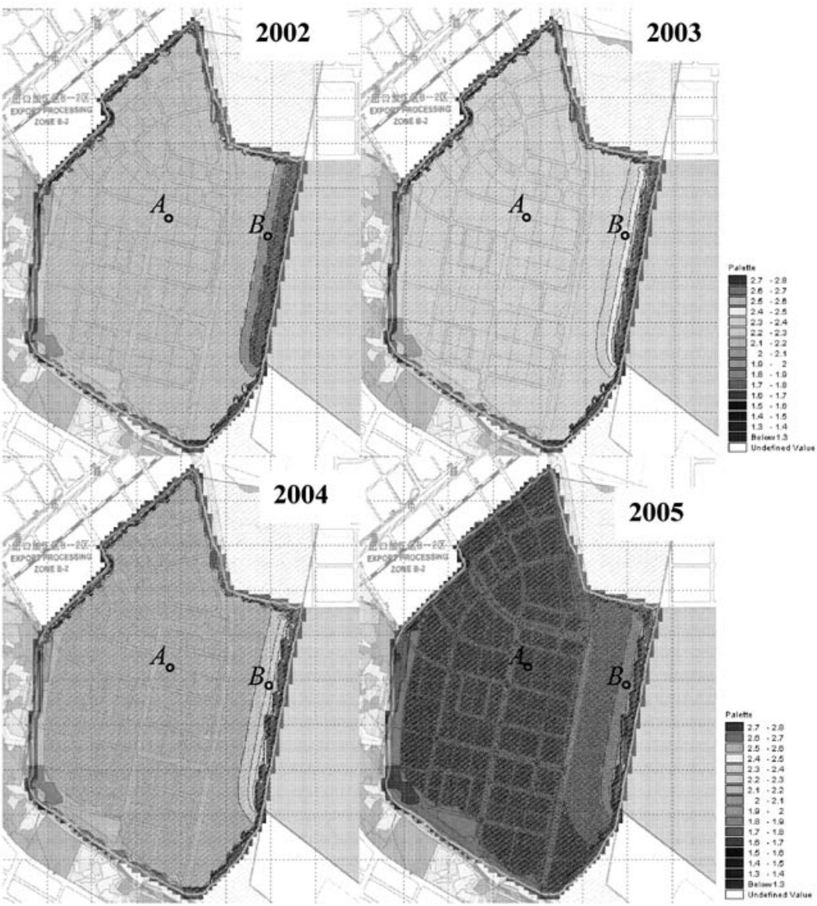

Figure 14.11: Salinity at the end of every year in simulation

Figure 14.12 shows the salinity in SZ at point A and point B. The salinity reduces gradually as the accumulation of rainfall in pervious case. Paved case completely deviates to the natural state. While the water level of groundwater falls down, the salinity increases gradually. This means that paved ground surface has a negative impact on the environment of the coastal regions.

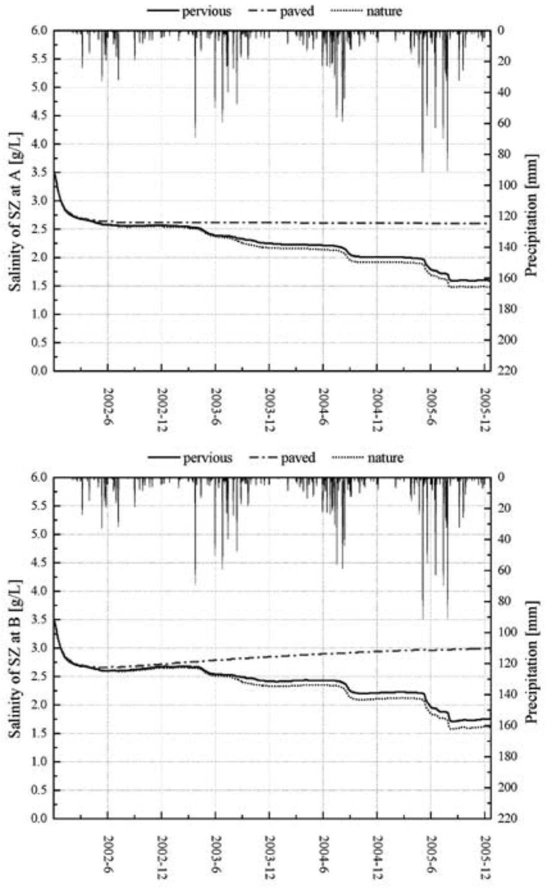

Figure 14.12 Salinity processes in SZ at point A and B

14.4 CONCLUSIONS

Urbanization changes the state of ground surface seriously and increases the rate of impervious area in the whole region. As a result, under the same climatic and hydrological condition the state of surface water and groundwater can be affected. The situation is more serious in coastal cities due to seawater intrusion. In order to improve the water environment and water quality in urban areas, especially in the coastal fill built land, it is suggested to use all kinds of fresh water sources, such as rainwater, to recharge groundwater. The pervious pavement is one useful measure to store and infiltrate rainwater into ground efficiently. Basing on Horton theory, a calculation model of infiltration process in pervious pavement is established. According to the structure and the material features of pervious pavement, for a certain rain event, the infiltration process in the pervious pavement and subsoil can be calculated out. The model is useful tool to design pervious pavement structures. Combing the infiltration model of pervious pavement and the MIKE SHE, which can be used to make hydrological calculation, an approach is established for analyzing the infiltration process and its affection of pervious pavement. Using a coastal fill built land in Dalian as an example, the study completed the calculation of the amount of water storing in pervious pavement and infiltrating into subsoil. For the impact analysis of pervious pavement, three states of ground surface, such as pervious pavement, impervious pavement and natural ground, are simulated by MIKE SHE. The results show that the water head elevations of groundwater for the different kind of land surfaces are: nature case > pervious case > paved case. The salinity values at a same elevation are: paved case > pervious case > nature case. That is, rainwater infiltrating through pervious pavement can effectively resists seawater intrusion, and the interface between fresh water and seawater can be pushed back to coastline.

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Index